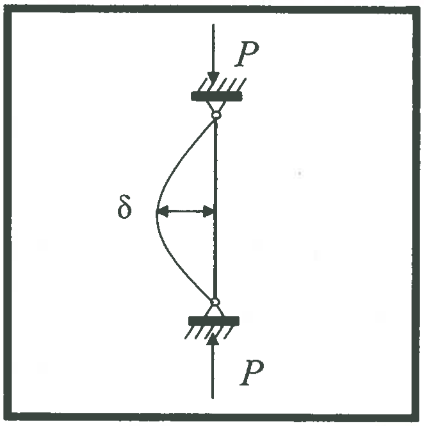
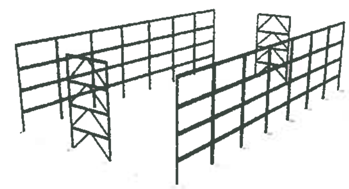
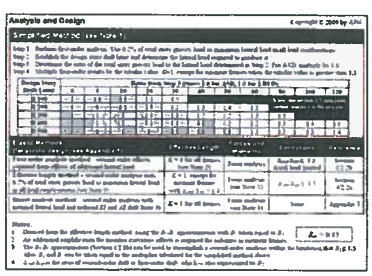
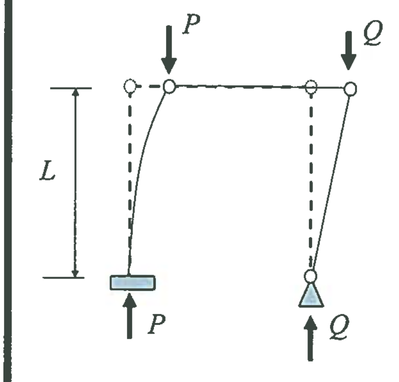
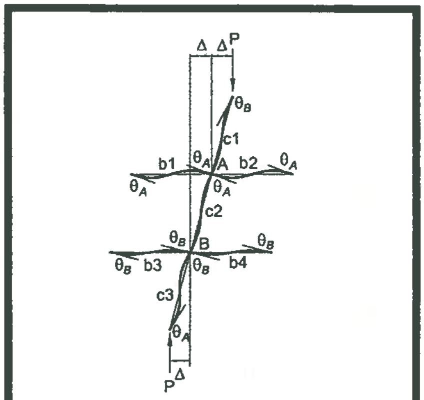
Design Steel Your Way II: Efficient Analysis for Steel Design Using the 2005 AISC Specification

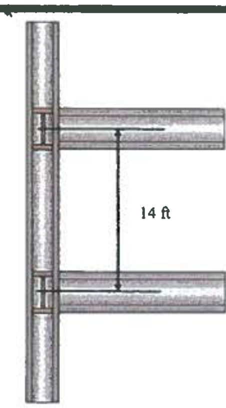












Design Steel Your Way II: Efficient Analysis for Steel Design Using the 2005 AISC Specification

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  - Structural analysis
  - Second-order effects
- Part II – How you go about doing it!
  - Determination of required strength
  - Applications
  - Examples

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I-2

Part I – What you have to consider!

- Design Basis
- Analysis myths and the AISC Specification
- Types of analysis
- Second-order analysis
- Stability
- Geometric imperfections
- Residual stresses

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I-3

Design Basis

Steel is Steel:
It doesn't know how it was designed.

- The unifying factors
  - The same limit states must be considered for all design philosophies
  - The nominal strength is the same for all design philosophies
  - There can be a direct relationship between resistance factors and safety factors

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I-4

Design Basis

There is no longer
ASD vs. LRFD

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I-5

Design Basis

- Important Definitions
  - Required Strength, R_r
    - ASD, R_a
    - LRFD, R_u
  - Nominal Strength, R_n
  - Available Strength, R_c
    - Allowable Strength, R_n/Ω
    - Design Strength, ϕR_n

ANSI/AISC 360-05

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I-6

Design Basis

B3.3 For LRFD, design shall be performed in accordance with:

$$R_u \leq \phi R_n$$

B3.4 For ASD, design shall be performed in accordance with:

$$R_a \leq R_n / \Omega$$

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I-7

Safety Factors

- Calibration of LRFD with ASD leads to

V/D = 3/

$$\Omega = \frac{1.5}{\phi}$$

This relationship is used throughout the Specification

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I-8

Design Basis

Analysis vs. Design

$$R_r \leq R_c$$

Required strength < Available Strength

The two sides of this equation must be balanced
One side should not be determined with more precision than the other

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I-9

Compatibility of Analysis and Design

- Must be properly matched
- Must assure adequate level of safety
- Must provide an efficient process

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I-10

Myth about AISC Specifications

- Past AISC Specifications have never said anything about analysis!

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I-11

Fact

1989 ASD

- A5.3. Structural Analysis

  "Selection of the method of analysis is the prerogative of the responsible engineer."

- B4.Stability

  "General stability shall be provided for the structure as a whole and for each compression element."

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I-12

Fact

1989 ASD

- C1. General

"In addition to meeting the requirements of member strength and stiffness, frames and other continuous structures shall be designed to provide the needed deformation capacity and to assure over-all frame stability."

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I-13

Fact

1999 LRFD

A5. Design Basis

"The required strength of structural members and connections shall be determined by structural analysis for the appropriate factored load combinations as stipulated..."

"Design by either elastic or plastic analysis is permitted,..."

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I-14

Fact

1999 LRFD

C1. Second Order Effects

"Second order effects shall be considered in the design of frames"

C1.2 Design by Elastic Analysis

"In structures designed on the basis of elastic analysis, M_u for beam-columns, connections, and connected members shall be determined from a second-order elastic analysis…"

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I-15

Fact

AISC 360-05 (the 2005 Specification)

B3.1 Required Strength

"The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations as stipulated…"

"Design by elastic, inelastic or plastic analysis is permitted."

B3.5 Design for Stability

"Stability of the structure and its elements shall be determined in accordance with Chapter C."

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I-16

Fact

AISC 360-05 (the 2005 Specification)

C1.1 Stability Design Requirements

"Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of second-order effects, flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses on the stability of the structure and its elements is permitted."

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I-17

Fact

AISC 360-05 (the 2005 Specification)

"The methods prescribed in this chapter and Appendix 7, Direct Analysis Method, satisfy these requirements."

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I-18

Analysis Methods in AISC 360

C2.2a. Design by Second-Order Analysis
C2.2b. Design by First-Order Analysis
Appendix 7. Direct Analysis Method

or

Any method that gets the correct answer.

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I-19

Analysis

- Process used to determine how a structure responds to specific loads or actions
- Measured by establishing forces and deformations throughout the structure

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I-20

Analysis

- A mathematical model used to predict behavior of a real structure based on:
  - Engineering mechanics theory
  - Laboratory research
  - Model and field experimentation
  - Experience
  - Engineering judgment
- In the end, the results must satisfy equilibrium

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I-21

Analysis

- Determinate vs. Indeterminate
- Linear vs. Nonlinear
- Small vs. Large Displacement
- 1st Order vs. 2nd Order

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I-22

Determinate vs. Indeterminate

- Determinate
  - Number of unknowns = number of equations
    - Equations of equilibrium
    - Condition equations
  - Force and moments independent of member properties
- Indeterminate
  - Number of unknowns > number of equations
    - Equilibrium, condition, other equations
  - Forces and moments dependent on relative member properties

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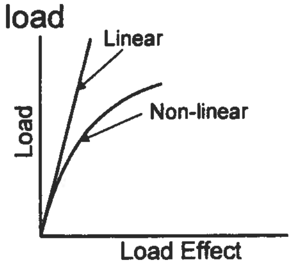
I-23

Linear vs. Non-linear

- Linear
  - Effects of load proportional to load
  - Elastic material
  - Superposition applicable
- Non-linear
  - Effects of load not proportional to load
  - Geometric
  - Elastic-plastic material
  - Inelastic material
  - Superposition not valid



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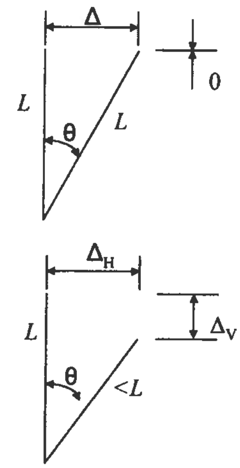
I-24

Small Deflection vs. Large Deflection

- Small
  - 1^{st} order
  - $\sin\theta = \tan\theta = \theta$
- Large
  - 2^{nd} order
  - $\sin\theta \neq \tan\theta \neq \theta$
  - Cable structures



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I-25

1^{st} Order vs. 2^{nd} Order

- 1^{st} order
  - Equilibrium formulated about undeformed geometry
  - Beam-column ignores impact of axial load on moment
- 2^{nd} order
  - Equilibrium formulated about final displaced geometry
  - Beam-column includes impact of axial load on moment

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I-26

Structural Engineering

"The art of modeling materials that we do not wholly understand, into shapes that we cannot precisely analyze, so as to withstand forces we cannot really assess, in such a way that the community at large has no reason to suspect the extent of our ignorance!"

A. R. Dykes, IStructE

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I-27

MODEL - ACTUAL
- CONSTRUCTION
DIFFERENCE

Structural Engineering

"Is the exact analysis of an approximate model good enough to qualify as an approximate analysis of the exact structure?"

M. Sozen

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I-28

Categories of Analysis

- First-Order Elastic Analysis
- Linear Buckling Analysis
- Second-Order Elastic Analysis
- First-Order Plastic Hinge Analysis
- Second-Order Inelastic Analysis

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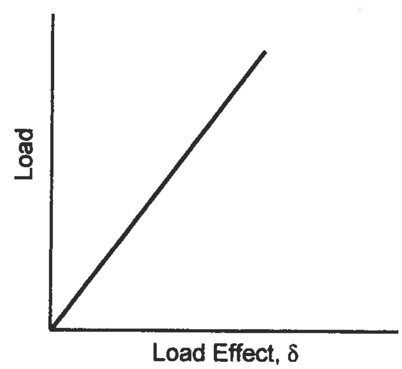
I-29

First-Order Elastic Analysis

- Linear
  - Elastic Materials
  - Equilibrium about original geometry
- Typical Methods
  - Moment Distribution
  - Slope Deflection
  - Stiffness (Matrix) Method
  - Most Commercial Computer Programs



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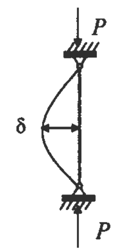
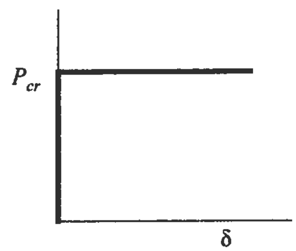
I-30

Linear Buckling Analysis

- Bifurcation Analysis
  - From no displacement to an infinite displacement
  - Eigenvalue analysis
- Typical Column Analysis
  - Column Effective Length
- Linear-Elastic





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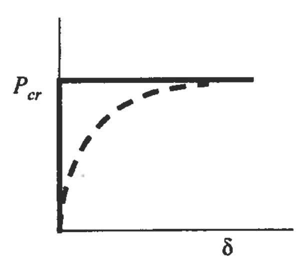
I-31

Second-Order Elastic Analysis

- Linear-Elastic Material
- Equilibrium about displaced configuration
  - $P\Delta$
  - $P\delta$
- Exact, Iterative or Approximate Solution
- Accounts for Stability
- Approaches Buckling Load



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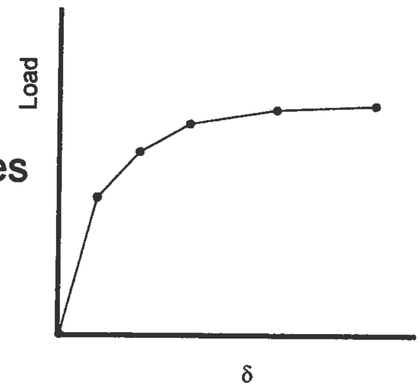
I-32

First-Order Plastic Hinge Analysis

- Plastic hinges form
  - Elastic/Fully Plastic
  - M_y followed by M_p
- Redistribution of Forces
- Methods
  - Rigid-Plastic
  - Elastic-Plastic

Not doing anymore



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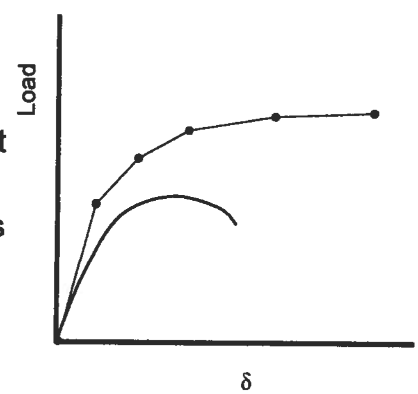
I-33

Second-Order Inelastic Analysis

- Accounts for Material Yielding
- Plastic-hinged based
  - One beam-column element for each member
  - Zero-Length Plastic Hinges
  - Efficient analysis of large-scale buildings
  - Still only approximate



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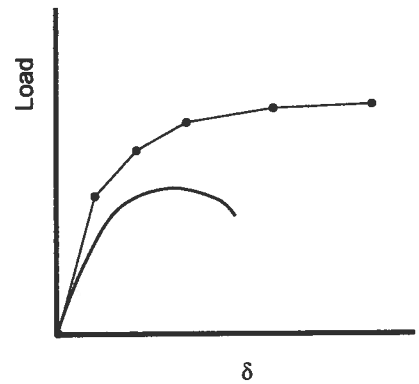
I-34

Second-Order Inelastic Analysis

- Plastic-Zone Analysis
  - Distributed plasticity
  - Discretized members
  - Many finite elements
  - Most complex method



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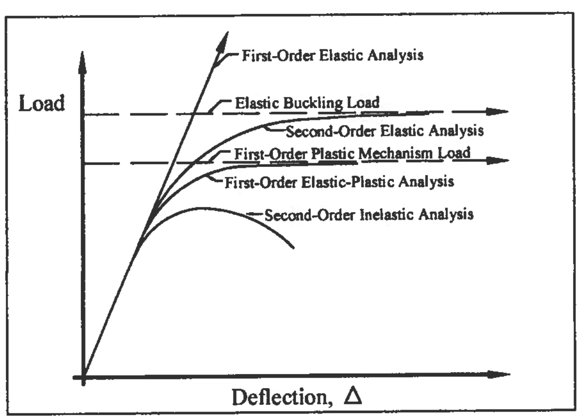
I-35

Comparison of Analysis Results



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I-36

AISC 360

B3.1 Required Strength

"The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations as stipulated..."

"Design by elastic, inelastic or plastic analysis is permitted."

Elastic analysis is the most common approach in practice.

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I-37

AISC 360

C1.1 Stability Design Requirements

"Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of:

1. second-order effects,
2. flexural, shear and axial deformations,
3. geometric imperfections, and
4. member stiffness reduction due to residual stresses

on the stability of the structure and its elements is permitted."

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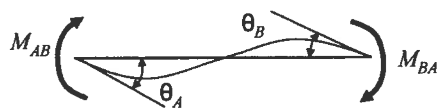
I-38

Elastic Analysis

- Member Slope Deflection Equations

$$M_{AB} = \frac{4EI}{L}\theta_A + \frac{2EI}{L}\theta_B - \frac{6EI}{L^2}\Delta_{AB} + FEM_{AB}$$

Flexural deformations only



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I-39

Elastic Analysis

- Slope Deflection in Matrix Form

$$\begin{bmatrix} V_A \\ M_A \\ V_B \\ M_B \end{bmatrix} = \begin{bmatrix} \frac{12EI}{L^3} & \frac{-6EI}{L^2} & \frac{12EI}{L^3} & \frac{-6EI}{L^2} \\ \frac{6EI}{L^2} & \frac{4EI}{L} & \frac{-6EI}{L^2} & \frac{2EI}{L} \\ \frac{12EI}{L^3} & \frac{-6EI}{L^2} & \frac{12EI}{L^3} & \frac{-6EI}{L^2} \\ \frac{6EI}{L^2} & \frac{2EI}{L} & \frac{-6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \begin{bmatrix} \delta_{A2} \\ \theta_A \\ \delta_{B2} \\ \theta_B \end{bmatrix}$$

Includes only flexural deformations

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I-40

Elastic Analysis

- Member Stiffness Matrix

$$\begin{Bmatrix} P_A \\ V_A \\ M_A \\ P_B \\ V_B \\ M_B \end{Bmatrix} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} & 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} \\ 0 & \frac{-6EI}{L^2} & \frac{4EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{2EI}{L} \\ \frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} & 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} \\ 0 & \frac{-6EI}{L^2} & \frac{2EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{4EI}{L} \end{bmatrix} \begin{Bmatrix} \delta_{A1} \\ \delta_{A2} \\ \theta_A \\ \delta_{B1} \\ \delta_{B2} \\ \theta_B \end{Bmatrix}$$

Includes axial and flexural deformations

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I-41

Elastic Analysis

- Member Stiffness Matrix

$$\begin{Bmatrix} P_A \\ V_A \\ M_A \\ P_B \\ V_B \\ M_B \end{Bmatrix} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3(1+\beta_s)} & \frac{-6EI}{L^2(1+\beta_s)} & 0 & \frac{12EI}{L^3(1+\beta_s)} & \frac{-6EI}{L^2(1+\beta_s)} \\ 0 & \frac{-6EI}{L^2(1+\beta_s)} & \frac{4EI(4+\beta_s)}{L(1+\beta_s)} & 0 & \frac{-6EI}{L^2(1+\beta_s)} & \frac{2EI(2-\beta_s)}{L(1+\beta_s)} \\ \frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3(1+\beta_s)} & \frac{-6EI}{L^2(1+\beta_s)} & 0 & \frac{12EI}{L^3(1+\beta_s)} & \frac{-6EI}{L^2(1+\beta_s)} \\ 0 & \frac{-6EI}{L^2(1+\beta_s)} & \frac{2EI(2-\beta_s)}{L(1+\beta_s)} & 0 & \frac{-6EI}{L^2(1+\beta_s)} & \frac{4EI(4+\beta_s)}{L(1+\beta_s)} \end{bmatrix} \begin{Bmatrix} \delta_{A1} \\ \delta_{A2} \\ \theta_A \\ \delta_{B1} \\ \delta_{B2} \\ \theta_B \end{Bmatrix}$$

$$\beta_s = \frac{12EIf_s}{GAL^2}$$

Includes axial, flexural, and shearing deformations

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I-42

Elastic Analysis

- Member Stiffness Matrix – 3-Dimensional

$$\begin{Bmatrix} F_1 \\ F_2 \\ F_3 \\ F_4 \\ F_5 \\ F_6 \end{Bmatrix}_{ij} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & \frac{-6EI_z}{L^2} \\ 0 & 0 & \frac{12EI_y}{L^3} & 0 & \frac{6EI_y}{L^2} & 0 \\ 0 & 0 & 0 & \frac{GJ}{L} & 0 & 0 \\ 0 & 0 & \frac{6EI_y}{L^2} & 0 & \frac{4EI_y}{L} & 0 \\ 0 & \frac{-6EI_z}{L^2} & 0 & 0 & 0 & \frac{4EI_z}{L} \end{bmatrix} \begin{Bmatrix} \delta_1 \\ \delta_2 \\ \delta_3 \\ \delta_4 \\ \delta_5 \\ \delta_6 \end{Bmatrix}_{ij}$$

Includes axial, flexural, and torsional deformations

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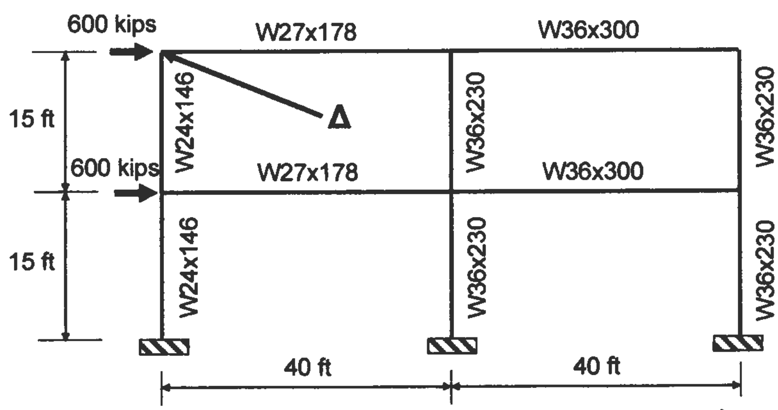
I-43

Elastic Analysis



$$\Delta_{flex+axial+shear} = 2.94 \text{ in.}, \ \Delta_{flex+axial} = 2.42 \text{ in.},$$

$$\Delta_{flex} = 2.24 \text{ in.}, \ \Delta_{flex+rigid\ ends} = 1.79 \text{ in.}$$

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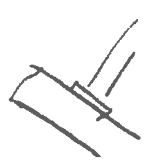
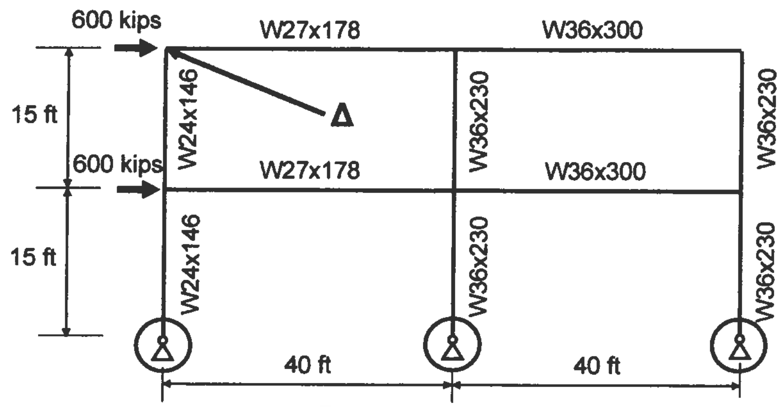
I-44



Elastic Analysis



With pin supports $\Delta_{flex,axial,shear} = 7.11$ in.

compared to $\Delta_{flex,axial,shear} = 2.94$ in. with fixed supports

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I-45

Elastic Analysis

- What approach does your software use?

?

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I-46

RISA ??

Elastic Analysis

- What approach does your software use?
  1. Includes axial deformations_____
  2. Includes flexural deformations_____
  3. Includes shearing deformations_____
  4. Includes torsional deformations_____
  5. Includes component deformations
     a. panel zone_____
     b. connection components_____

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I-47

AISC 360

C1.1 Stability Design Requirements

"Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of:

1. second-order effects,
2. flexural, shear and axial deformations,
3. geometric imperfections, and
4. member stiffness reduction due to residual stresses

on the stability of the structure and its elements is permitted."

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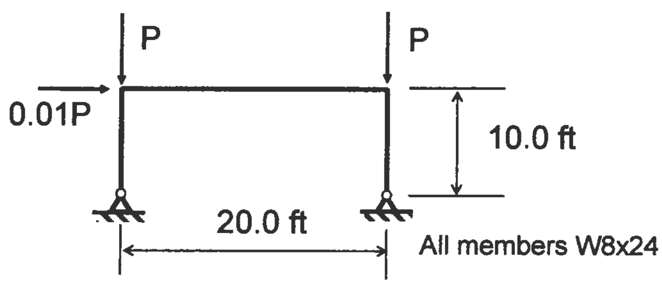
I-48

Second-Order Elastic Analysis vs. First-Order Elastic Analysis



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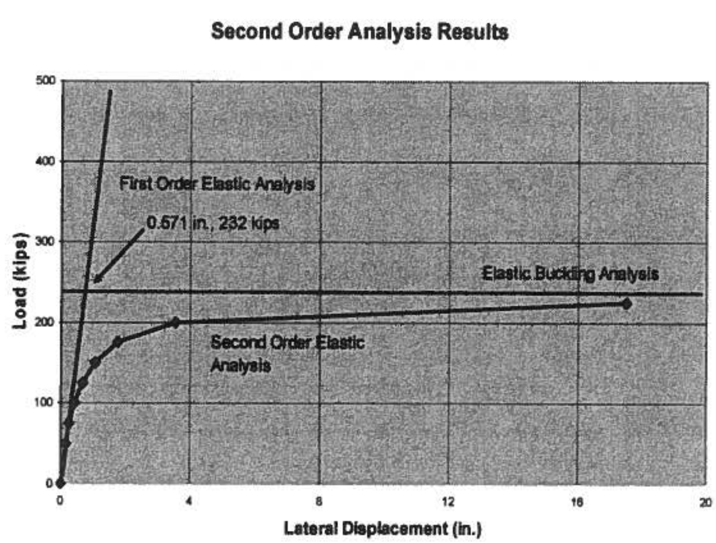
I-49



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I-50

Second-Order Analysis

- Impact of second-order effects
  - Moments in beams and columns
  - Shear and axial forces in beams and columns
  - Forces on connections and foundations
- Second-order moments may have a different distribution than first-order moments
- Superposition does not apply

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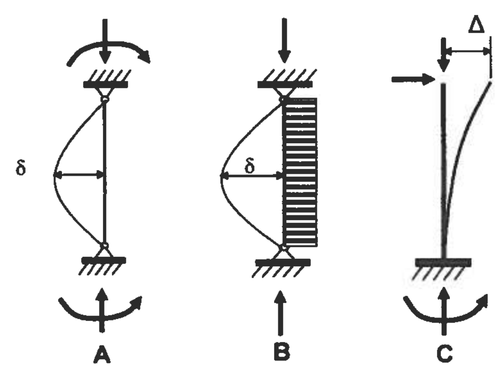
I-51

Second-Order Analysis

- Approximate step-by-step second-order analysis
- 3 examples



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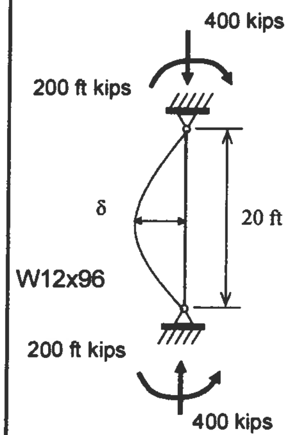
I-52

Second-Order Analysis

A

First Iteration on member effect



$$\delta_{1st} = \frac{Ml^2}{8EI} = \frac{200(20)^2(1728)}{8(29000)(833)} = 0.715 \text{ in.}$$

$$M_{2nd} = \frac{(400(0.715))}{12} = 23.8 \text{ ft-kips}$$

$$M_r = 200 + 23.8 = 224 \text{ ft-kips}$$

$$\text{Amplification Factor } = \frac{(224)}{200} = 1.12$$

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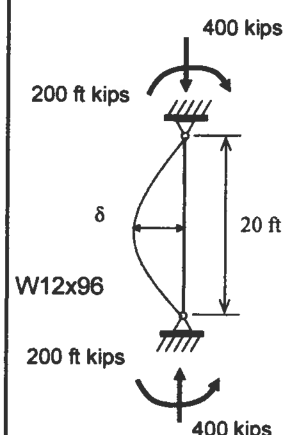
I-53

Second-Order Analysis

A

First Iteration on member effect



First-order moment = 200 ft-kips

Using the rectangular moment diagram will yield more deflection than actually there.

First cycle second-order moment = 224 ft-kips

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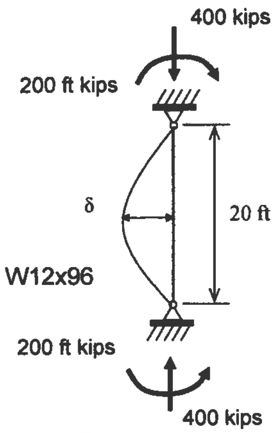
I-54

Second-Order Analysis

A

Second Iteration on member effect (Approximation)



$$\delta_{1st} = \frac{Ml^2}{8EI} = \frac{23.8(20)^2(1728)}{8(29000)(833)} = 0.0851 \text{ in.}$$

$$M_{2nd} = \frac{(400(0.0851))}{12} = 2.84 \text{ ft-kips}$$

$$M_r = 200 + 23.8 + 2.84 = 227 \text{ ft-kips}$$

$$\text{Amplification Factor } = \frac{(227)}{200} = 1.14$$

This might be expected to over estimate the amplification

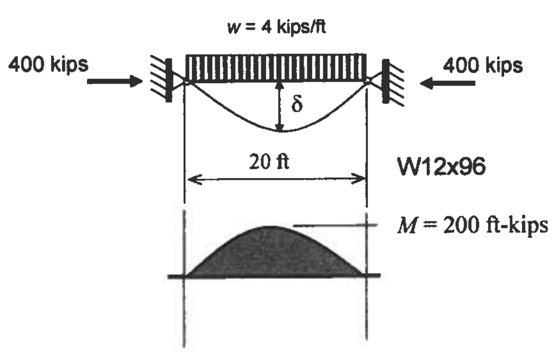
There's always a solution in steel

I-55

Second-Order Analysis

B

Beam with axial force



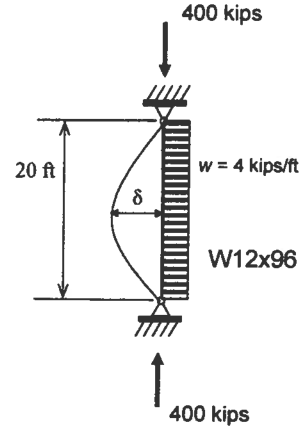
There's always a solution in steel

I-56

Second-Order Analysis

B

First Iteration on member effect



$$\delta_{1st} = \frac{5wl^4}{384EI} = \frac{5(4.0)(20)^4(1728)}{384(29000)(833)} = 0.596 \text{ in.}$$

$$M_{2nd} = \frac{(400(0.596))}{12} = 19.9 \text{ ft-kips}$$

$$M_r = 200 + 19.9 = 220 \text{ ft-kips}$$

$$\text{Amplification Factor } = \frac{(220)}{200} = 1.10$$

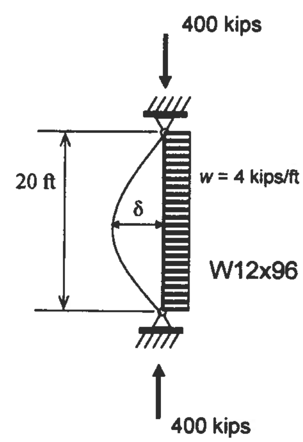
There's always a solution in steel

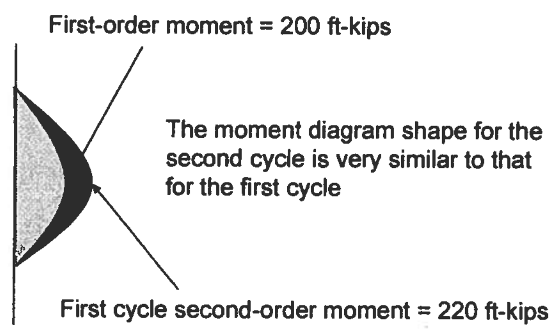
I-57

Second-Order Analysis

B

First Iteration on member effect



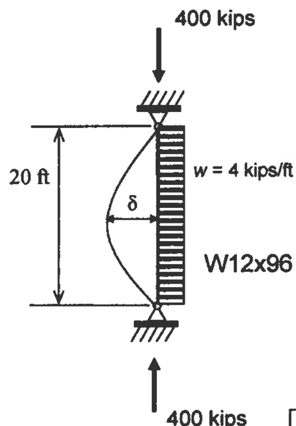


There's always a solution in steel

I-58

Second-Order Analysis B

Second Iteration on member effect (Approximation)



$$\delta_{1st} = \frac{5Ml^2}{48EI} = \frac{5(19.9)(20)^2(1728)}{48(29000)(833)} = 0.0593 \text{ in.}$$

$$M_{2nd} = \frac{(400(0.0593))}{12} = 1.98 \text{ ft-kips}$$

$$M_r = 200 + 19.9 + 1.98 = 222 \text{ ft-kips}$$

$$\text{Amplification Factor } = \frac{(222)}{200} = 1.11$$

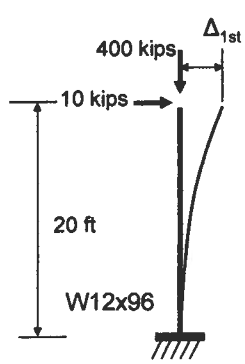
This can be expected to accurately estimate the amplification

There's always a solution in steel

I-59

Second-Order Analysis C

First Iteration on sidesway effect



$$\Delta_{1st} = \frac{Hl^3}{3EI} = \frac{10(20)^3(1728)}{3(29000)(833)} = 1.91 \text{ in.}$$

$$M_{2nd} = \frac{1.91 \text{ in. } (400 \text{ kips})}{12} = 63.7 \text{ ft-kips}$$

$$M_r = 10(20) + 63.7 = 264 \text{ ft-kips}$$

$$\text{Amplification Factor } = \frac{(264)}{200} = 1.32$$

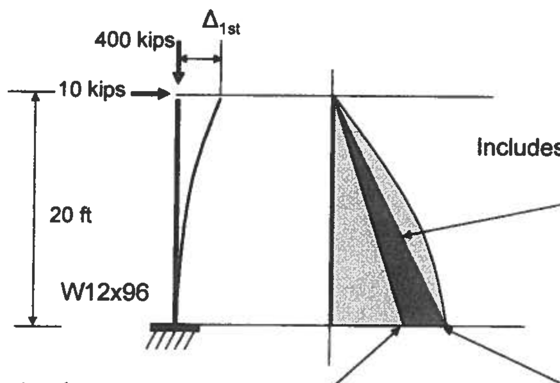
There's always a solution in steel

I-60

Second-Order Analysis

C

First Iteration on sidesway effect



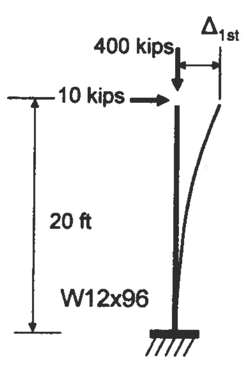
There's always a solution in steel

I-61

Second-Order Analysis

C

Second Iteration on sidesway effect (Approximation)



$$H_2 = \frac{63.7}{20} = 3.19 \text{ kips}$$

$$\Delta_{1st} = \frac{Hl^3}{3EI} = \frac{3.19(20)^3(1728)}{3(29000)(833)} = 0.608 \text{ in.}$$

$$M_{2nd} = \frac{0.608 \text{ in. } (400 \text{ kips})}{12} = 20.3 \text{ ft-kips}$$

$$M_r = 200 + 63.7 + 20.3 = 284 \text{ ft-kips}$$

$$\text{Amplification Factor } = \frac{(284)}{200} = 1.42$$

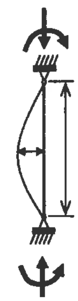
This might be expected to under estimate the amplification

There's always a solution in steel

I-62

Second-Order Analysis

A



- Exact Theoretical Solution

$$\mu = \sqrt{\frac{PL^2}{4EI}} = \sqrt{\frac{400(20)^2(144)}{4(29000)(833)}} = 0.488$$

$$M_{MAX} = M_o \sec\mu = M_o \sec(0.488) = 1.13M_o$$

$$\delta = \frac{M_oL^2}{8EI}\left(\frac{2(1-\cos\mu)}{\mu^2\cos\mu}\right) = \frac{M_oL^2}{8EI}\left(\frac{2(1-\cos(0.488))}{(0.488)^2\cos(0.488)}\right) = 1.11\frac{M_oL^2}{8EI}$$

$\boxed{\text{Compared to 1.14}}$

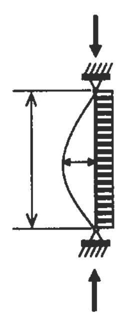
There's always a solution in steel

I-63

Second-Order Analysis

B

- Exact Theoretical Solution
  Commentary Benchmark Problem Case 1.



$$\mu = \sqrt{\frac{PL^2}{4EI}} = \sqrt{\frac{400(20)^2 144}{4(29000)(833)}} = 0.488$$

$$M_{MAX} = \frac{wL^2}{8}\left[\frac{2(\sec\mu-1)}{\mu^2}\right] = \frac{wL^2}{8}\left[\frac{2(\sec(0.488)-1)}{(0.488)^2}\right] = 1.11\frac{wL^2}{8}$$

$$\delta_{MAX} = \frac{5wL^4}{384EI}\left[\frac{12(2\sec\mu-\mu^2-2)}{5\mu^4}\right] = \frac{5wL^4}{384EI}\left[\frac{12\left(2\sec(0.488)-(0.488)^2-2\right)}{5(0.488)^4}\right] = 1.11\frac{5wL^4}{384EI}$$

$\boxed{\text{Compared to 1.11}}$

There's always a solution in steel

I-64

Second-Order Analysis

C

- Exact Theoretical Solution

Commentary Benchmark Problem Case 2.

$$\alpha = \sqrt{\frac{PL^2}{EI}} = \sqrt{\frac{400(20)^2(144)}{29000(833)}} = 0.977$$

$$M_{MAX} = HL\left(\frac{\tan\alpha}{\alpha}\right) = HL\left(\frac{\tan(0.977)}{0.977}\right) = 1.52HL$$

$$\delta_{MAX} = \frac{HL^3}{3EI}\left(\frac{3(\tan\alpha-\alpha)}{\alpha^3}\right) = \frac{HL^3}{3EI}\left(\frac{3(\tan(0.977)-0.977)}{(0.977)^3}\right) = 1.62\left(\frac{HL^3}{3EI}\right)$$

Compared to 1.42

There's always a solution in steel

I-65

Rigorous Second-Order Analysis

- Beam-column approach using modified slope-deflection equations (Stability Functions)
- Finite Element approach based on energy theorems
- Pseudo load approach using fictitious loads.
- And others

There's always a solution in steel

I-66

Rigorous Second-Order Analysis

- Modified Slope-Deflection Equations

$$M_A = \frac{EI}{L}\left[s_{ii}\theta_A + s_{ij}\theta_B - \left(s_{ii} + s_{ij}\right)\frac{\Delta_{AB}}{L} \right]$$

$$M_B = \frac{EI}{L}\left[s_{ij}\theta_A + s_{jj}\theta_B - \left(s_{ji} + s_{jj}\right)\frac{\Delta_{AB}}{L} \right]$$

- Stability Functions

$$s_{ii} = s_{jj} = \frac{kL\sin kl - (kl)^2\cos kL}{2 - 2\cos kL - kL\sin kL}$$

$$s_{ij} = s_{ji} = \frac{(kL)^2 - kL\sin kL}{2 - 2\cos kL - kL\sin kL}$$

$$k = \sqrt{\frac{P}{EI}}$$

There's always a solution in steel

I-67

Rigorous Second-Order Analysis

- Advantages of modified slope-deflection equations
  - In addition to P-Δ, the P-δ effects are exactly represented for small deformations
  - Accounts for inclusion of axial forces on fixed end forces
  - Approximate yet “rigorous” analysis

There's always a solution in steel

I-68

Rigorous Second-Order Analysis

- Finite Element Method
  - Geometric Stiffness
  - Assumes small strains, large rotations, large displacements
  - Based on a cubic polynomial of transverse displacements
  - Uses an incremental stiffness matrix

$$k = k_o + k_p$$

There's always a solution in steel

I-69

Rigorous Second-Order Analysis

$$k = \frac{EI}{L}\begin{bmatrix} \frac{A}{I} & 0 & 0 & -\frac{A}{I} & 0 & 0 \\ 0 & \frac{12}{L^2} & \frac{6}{L} & 0 & -\frac{12}{L^2} & \frac{6}{L} \\ \cdots \\ \cdots \\ \cdots \\ 0 & \frac{6}{L} & 2 & 0 & -\frac{6}{L} & 4 \end{bmatrix} + \frac{P}{L}\begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{6}{5} & \frac{L}{10} & 0 & -\frac{6}{5} & \frac{L}{10} \\ \cdots \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -\frac{6}{5} & -\frac{L}{10} & 0 & \frac{6}{5} & -\frac{L}{10} \\ 0 & \frac{L}{10} & -\frac{L^2}{30} & 0 & -\frac{L}{10} & \frac{2L^2}{15} \end{bmatrix}$$

Elastic Stiffness or Small-Displacement Stiffness Matrix

Geometric Stiffness Matrix

There's always a solution in steel

I-70

Rigorous Second-Order Analysis

$$k=\frac{EI}{L}\begin{bmatrix} \frac{A}{I} & 0 & 0 & -\frac{A}{I} & 0 & 0 \\ 0 & \frac{12}{L^2} & \frac{6}{L} & 0 & -\frac{12}{L^2} & \frac{6}{L} \\ 0 & \frac{6}{L} & 4 & 0 & -\frac{6}{L} & 2 \\ -\frac{A}{I} & 0 & 0 & \frac{A}{I} & 0 & 0 \\ 0 & -\frac{12}{L^2} & -\frac{6}{L} & 0 & \frac{12}{L^2} & -\frac{6}{L} \\ 0 & \frac{6}{L} & 2 & 0 & -\frac{6}{L} & 4 \end{bmatrix}+\frac{P}{L}\begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{6}{5} & \frac{L}{10} & 0 & -\frac{6}{5} & \frac{L}{10} \\ 0 & \frac{L}{10} & \frac{2L^2}{15} & 0 & -\frac{L}{10} & -\frac{L^2}{30} \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -\frac{6}{5} & -\frac{L}{10} & 0 & \frac{6}{5} & -\frac{L}{10} \\ 0 & \frac{L}{10} & -\frac{L^2}{30} & 0 & -\frac{L}{10} & \frac{2L^2}{15} \end{bmatrix}$$

There's always a solution in steel

I-71

Rigorous Second-Order Analysis

- Finite Element Method advantages
  - One function in each matrix element for tension or compression
  - More easily extended to three dimensional analysis
  - Very general approach

There's always a solution in steel

I-72

Rigorous Second-Order Analysis

- Pseudo Load Approach
  - This is intended to refer to all other methods that might be used where loads are added to account for second-order effects.
  - The main advantage is that the stiffness matrix is only evaluated once.
  - As with the other approaches, it is still an iterative method.

There's always a solution in steel

I-73

Rigorous Second-Order Analysis

- What approach does your software use?

?

There's always a solution in steel

I-74

Rigorous Second-Order Analysis

- What approach does your software use?
  1. What method ______________________
  2. Reference__________________________
     ____
  3. Does it include second-order effects_____
     a. P-δ_____
     b. P-Δ_____

There's always a solution in steel

I-75

First-Order vs. Second-Order

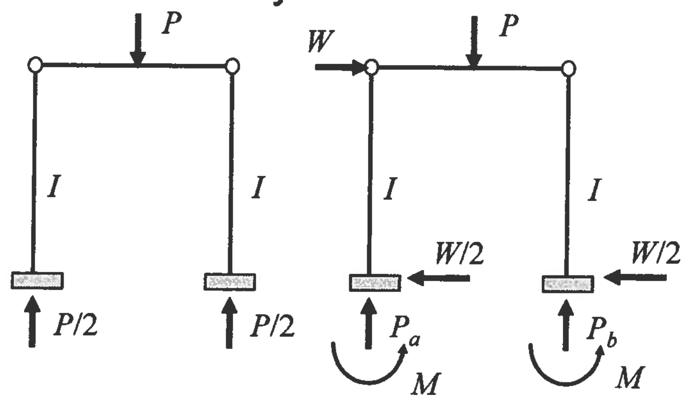
- Compare the results of a first-order analysis with those of a second-order analysis for:
  - Gravity load only
  - Gravity plus lateral

There's always a solution in steel

I-76

Symmetric Portal Frame

- First-Order Analysis

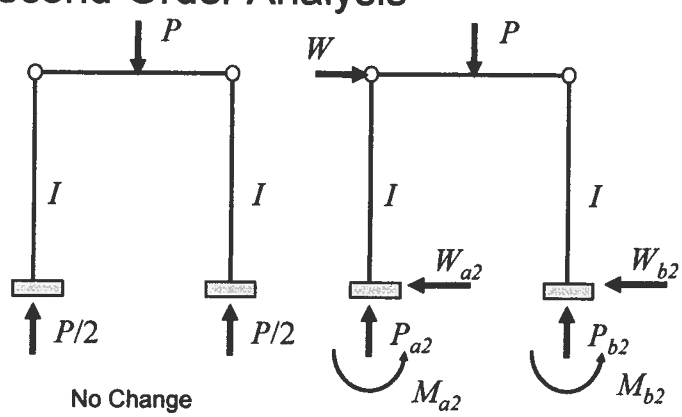


There's always a solution in steel

I-77

Symmetric Portal Frame

- Second-Order Analysis



No Change

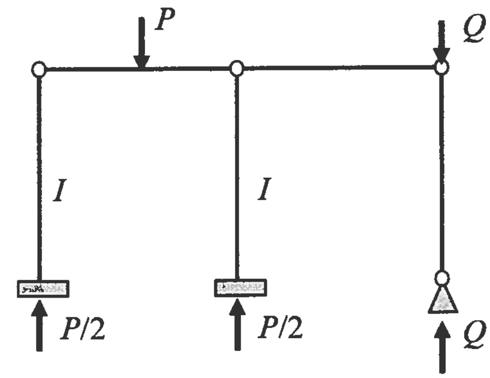
There's always a solution in steel

Results change

I-78

Symmetric Frame with Leaning Column

First-Order Analysis

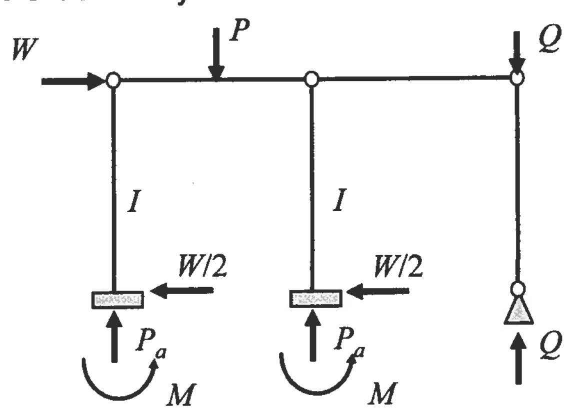


There's always a solution in steel

I-79

Leaning Column and Lateral Load

First-Order Analysis



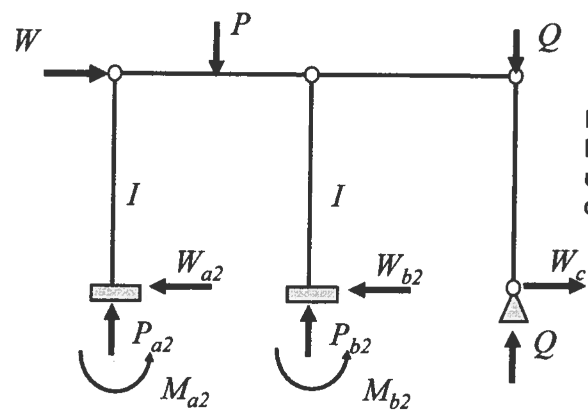
Leaning column has **no** influence under a first-order analysis

There's always a solution in steel

I-80

Leaning Column and Lateral Load

Second-Order Analysis

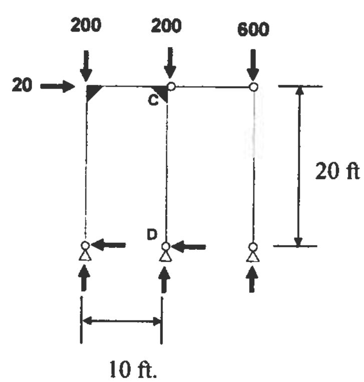


Leaning column **has** influence under a second-order analysis

There's always a solution in steel

I-81

Rigorous Second-Order Analysis



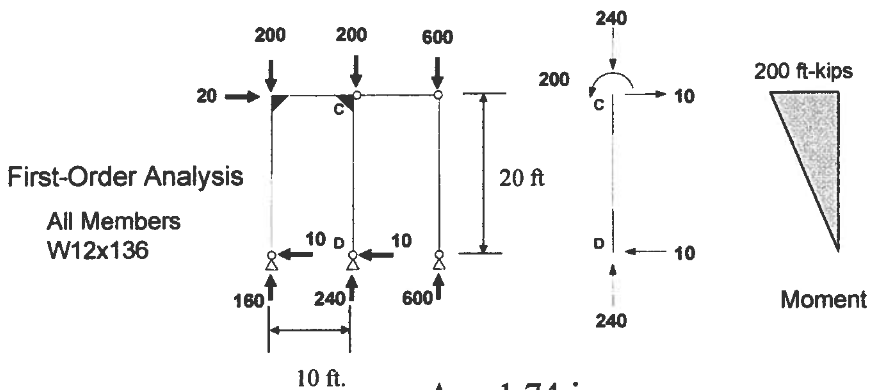
All Members W12x136

Consider a simple example structure

There's always a solution in steel

I-82

Rigorous Second-Order Analysis

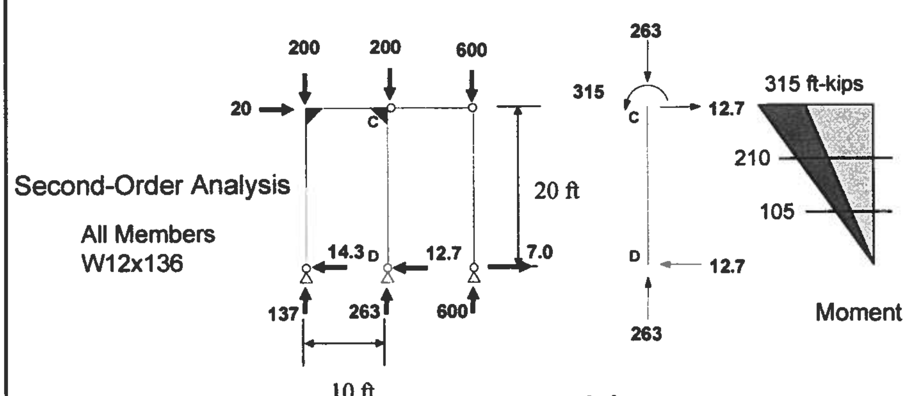


$\Delta_c = 1.74$ in.

There's always a solution in steel

I-83

Rigorous Second-Order Analysis



$\Delta_c = 2.79$ in.

Neglects P-δ

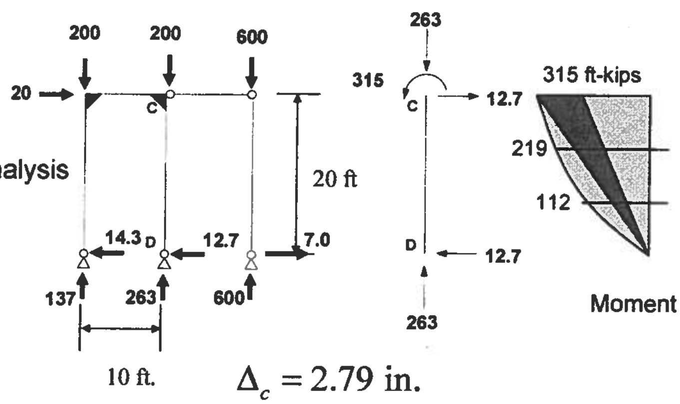
There's always a solution in steel

I-84

Rigorous Second-Order Analysis

Second-Order Analysis

All Members
W12x136



$\Delta_c = 2.79$ in.

Modeled with nodes at third points of columns

There's always a solution in steel

I-85

Second-Order Analysis by Amplified First-Order Analysis

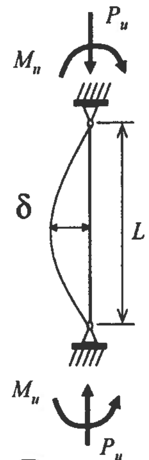
- The Goal:

  Develop a relationship between the first- order moment and the second-order moment that will permit a simple amplification of the results of a first-order analysis to determine the results of a second-order analysis

There's always a solution in steel

I-86

Second-Order Analysis by Amplified First-Order Analysis



Write the equilibrium equation at mid-height

$$M_{2nd} = M_u + P_u\delta$$

Define the amplification factor

$$AF(M_u) = M_{2nd} = M_u + P_u\delta$$

Solve for AF

$$AF = \frac{M_u + P_u\delta}{M_u}$$

There's always a solution in steel

I-87

Second-Order Analysis by Amplified First-Order Analysis

- Add to the denominator and simplify

$$(P_u\delta - P_u\delta)$$

$$AF = \frac{M_u + P_u\delta}{M_u + (P_u\delta - P_u\delta)} = \frac{1}{1 - \dfrac{P_u\delta}{M_u + P_u\delta}}$$

There's always a solution in steel

I-88

Second-Order Analysis by Amplified First-Order Analysis

- Simplifying Assumptions

$$\frac{\delta}{M_u + P_u\delta} \approx \frac{\delta}{M_u}$$

- LESS CONSER

LARGER

$$\frac{M_u}{\delta} = \frac{8EI}{L^2} \approx \frac{\pi^2 EI}{L^2} = P_e$$

- Substituting into the equation for AF

There's always a solution in steel

I-89

Second-Order Analysis by Amplified First-Order Analysis

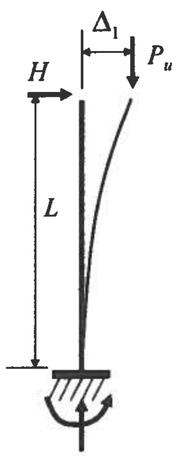
- The amplification factor for the "member effect" is thus

$$AF = \frac{1}{1 - \dfrac{P_u}{P_e}}$$

There's always a solution in steel

I-90

Second-Order Analysis by Amplified First-Order Analysis



- For a first-order analysis

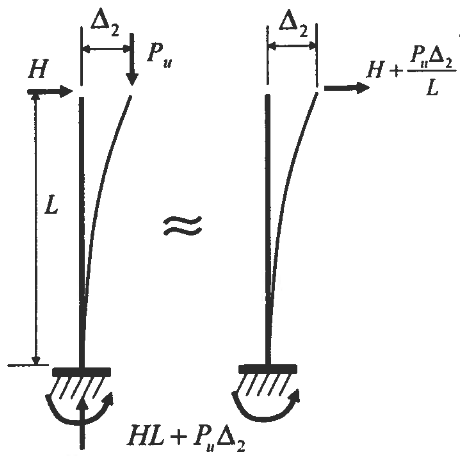
$$M = HL$$

$$\Delta_1 = \frac{HL^3}{3EI}$$

There's always a solution in steel

I-91

Second-Order Analysis by Amplified First-Order Analysis



- For second-order analysis assume these two models are equal

$$\Delta_2 = \frac{(H + P_u\Delta_2)L^3}{3EI}$$

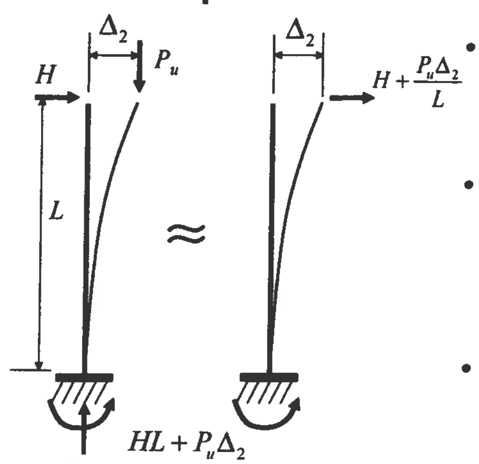
$$= \frac{HL^3}{3EI}\left(1 + \frac{P_u\Delta_2}{HL}\right)$$

$$\Delta_2 = \Delta_1 + \frac{P_u\Delta_1\Delta_2}{HL}$$

There's always a solution in steel

I-92

Second-Order Analysis by Amplified First-Order Analysis



- Solve for Δ_2

$$\Delta_1 = \left(1 - \frac{P_u \Delta_1}{HL}\right)\Delta_2$$

- Thus,

$$\Delta_2 = \frac{\Delta_1}{\left(1 - \frac{P_u \Delta_1}{HL}\right)} = (AF)\Delta_1$$

- So for the "sway effect"

$$AF = \frac{1}{1 - \frac{P_u \Delta_1}{HL}}$$

There's always a solution in steel

I-93

Second-Order Analysis by Amplified First-Order Analysis

- Amplification for member effect

$$AF = \frac{1}{1 - \frac{P_u}{P_e}}$$

- Amplification for sway effect

$$AF = \frac{1}{1 - \frac{P_u \Delta_1}{HL}}$$

There's always a solution in steel

I-94

Amplified First-Order Analysis

- C2.1a. Second-Order Analysis by Amplified First-Order Elastic Analysis

  Any second-order analysis method that considers both P-Δ and P-δ effects may be used.

  The Amplified First-Order Elastic Analysis Method defined in Section C2.1b is an accepted method for second-order elastic analysis of braced, moment, and combined framing systems.

There's always a solution in steel

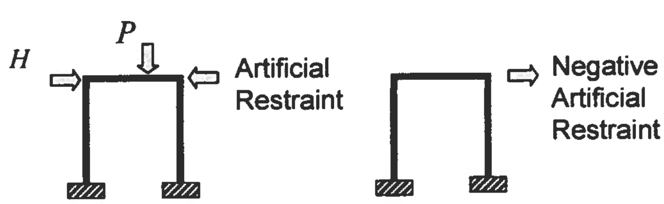
I-95

Amplified First-Order Analysis

- Required second-order flexural and axial strength AMP

$$M_r = B_1 M_{nt} + B_2 M_{lt} \quad \text{(C2-1a)}$$

$$P_r = P_{nt} + B_2 P_{lt} \quad \text{(C2-1b)}$$



No translation Lateral translation

There's always a solution in steel

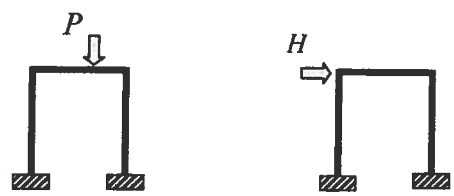
I-96

Amplified First-Order Analysis

- Common design office approximation

$$M_{nt} = \text{gravity load moments}$$

$$M_{lt} = \text{lateral load moments}$$



No translation ? Lateral translation

There's always a solution in steel

I-97

Amplified First-Order Analysis

- Member effect $$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1 \quad \text{(C2-2)}$$

Gravity

$$C_m = 0.6 - 0.4\left(M_1/M_2\right) \quad \text{(C2-4)}$$

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

$$\alpha P_r = 1.6 P_a \text{ or } P_u$$

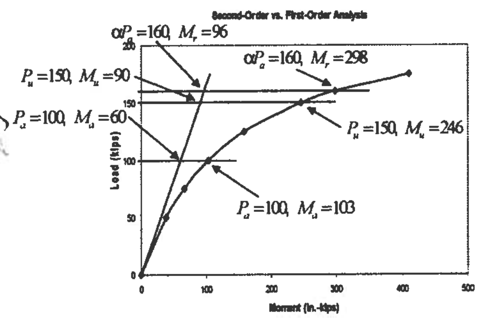
$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \quad \text{(C2-5)}$$

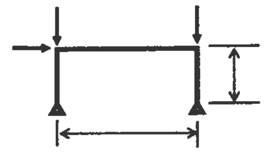
There's always a solution in steel

I-98

Amplified First-Order Analysis

- The constant α (1.0 for LRFD and 1.6 for ASD)





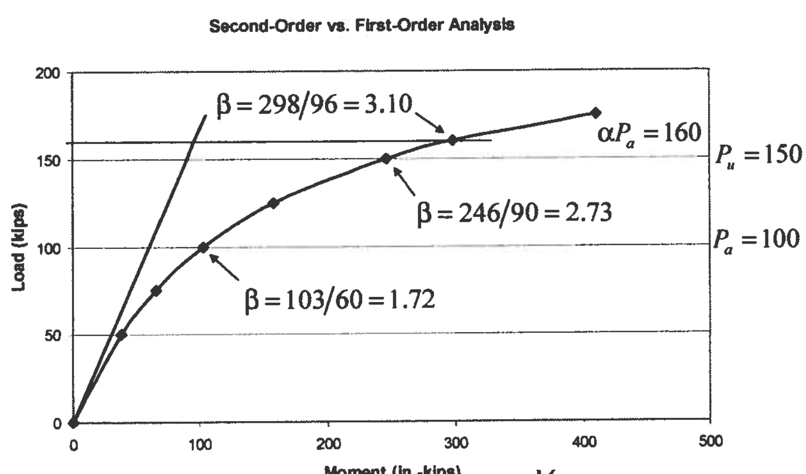
Use α to be sure that the analysis captures the nonlinear aspects at the ultimate strength

Live/DL = 3.0 Assum At

There's always a solution in steel

I-99

Amplified First-Order Analysis



$\beta = \dfrac{M_{2nd}}{M_{1st}}$ = amplification factor

There's always a solution in steel

I-100

Amplified First-Order Analysis

- Sway effect

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1 \quad \text{(C2-3)}$$

$$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta_H} \quad \text{(C2-6b)}$$

$R_m = 0.85$ for moment frames

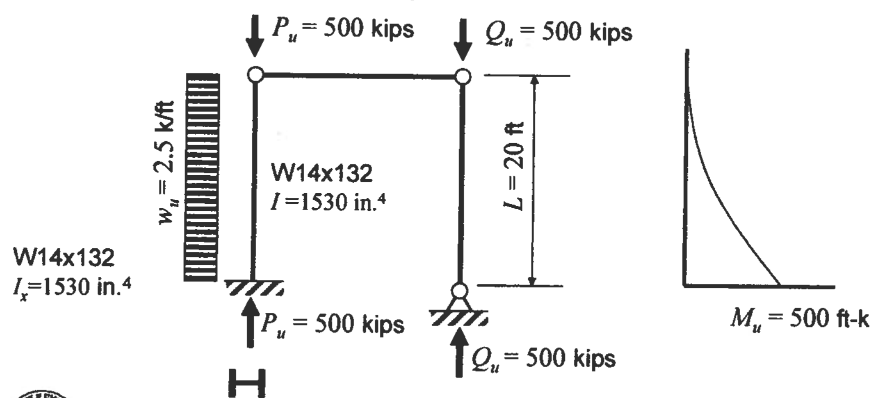
$= 1.0$ for braced frames

There's always a solution in steel

I-101

Amplified First-Order Analysis

- Application (LRFD)

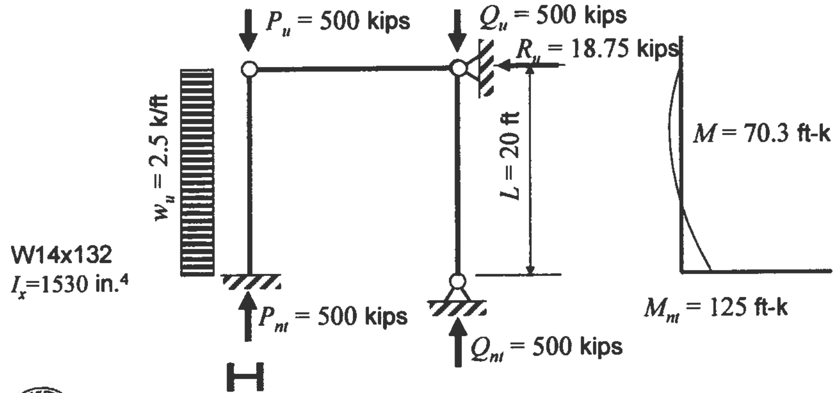


There's always a solution in steel

I-102

Amplified First-Order Analysis

- No translation

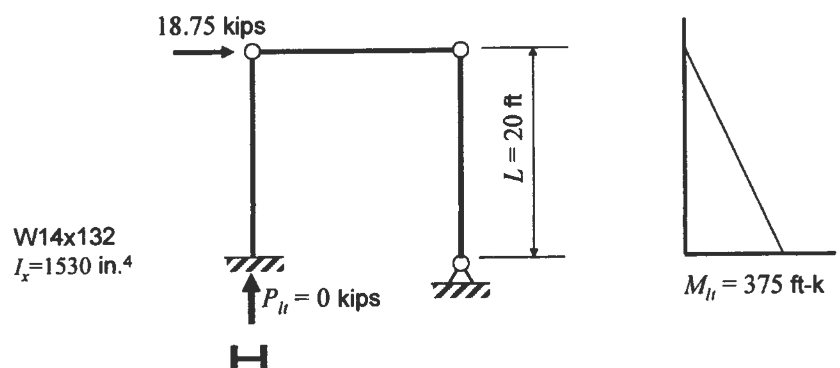


There's always a solution in steel

I-103

Amplified First-Order Analysis

- Translation $M_u = M_{nt} + M_{lt} = 125 + 375 = 500$ ft-kips



There's always a solution in steel

I-104

Amplified First-Order Analysis

- Summary of analysis:

$$M_{nt} = 125 \text{ ft-kips}$$
$$M_{lt} = 375 \text{ ft-kips}$$
$$P_{nt} = 500 \text{ kips}$$
$$P_{lt} = 0 \text{ kips}$$

There's always a solution in steel

I-105

Amplified First-Order Analysis

- Member Amplification

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

$$C_m = 1.0 \text{ (transverse load)}$$
$$\alpha P_r = 500 + 0 = 500 \text{ kips}$$
$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{\pi^2 (29{,}000)(1530)}{(20(12))^2} = 7{,}600 \text{ kips}$$

There's always a solution in steel

I-106

Amplified First-Order Analysis

- Member Amplification

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} = \frac{1.0}{1 - \frac{500}{7,600}} = 1.07 \geq 1$$

There's always a solution in steel

I-107

Amplified First-Order Analysis

- Sway Amplification

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1$$

$$\alpha \Sigma P_{nt} = \alpha(P + Q) = 1.0(500 + 500) = 1,000 \text{ kips}$$

$$\Delta = \frac{HL^3}{3EI} = \frac{1.0(20)^3(1,728)}{3(29,000)(1530)} = 0.104 \text{ in.}$$

$$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta_H} = 0.85 \frac{1.0(20(12))}{0.104} = 1,960 \text{ kips}$$

There's always a solution in steel

I-108

Amplified First-Order Analysis

- Sway Amplification

$$B_2 = \frac{1}{1-\frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} = \frac{1}{1-\frac{1,000}{1,960}} = 2.04 \geq 1$$

There's always a solution in steel

I-109

Amplified First-Order Analysis

- Second-order results

$$M_r = B_1 M_{nt} + B_2 M_{lt} = 1.07(125) + 2.04(375) = 899 \text{ ft–kips}$$
$$P_r = P_{nt} + B_2 P_{lt} = 500 + 2.04(0) = 500 \text{ kips}$$

- If there had been no load on the leaning column

$$B_2 = \frac{1}{1-\frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} = \frac{1}{1-\frac{500}{1,960}} = 1.34 \geq 1$$

There's always a solution in steel

I-110

Example 1 (LRFD)

Determine the 2nd-order forces and moments for a W14x120 in a moment frame when the results of a 1st-order elastic analysis yields:

$$P_{nt} = 408 \text{ kips}$$
$$P_{lt} = 98 \text{ kips}$$
$$M_{ntA} = 47.3 \text{ ft-kips}$$
$$M_{ntB} = 94.5 \text{ ft-kips}$$
$$M_{ltA} = 77.5 \text{ ft-kips}$$
$$M_{ltB} = 155 \text{ ft-kips}$$

$$\boxed{\text{Load Case} = 1.2D + 0.5L + 1.6W}$$

There's always a solution in steel

I-111

Example 1 (LRFD)

Determine the member effect amplification:

No translation, M_{nt}

$$C_m = 0.6 - 0.4\left(M_1/M_2\right)$$
$$C_m = 0.6 - 0.4\left(47.3/94.5\right) = 0.4$$

There's always a solution in steel

I-112

Example 1 (LRFD)

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

$$P_{e1} = \frac{\pi^2(29,000)(1,380)}{\left(1.0(12.5)(12)\right)^2} = 17,600 \text{ kips}$$

There's always a solution in steel

I-113

Example 1 (LRFD)

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

$$B_1 = \frac{0.4}{1 - \frac{(408+98)}{17,600}} = 0.41 \ngeq 1.0 \therefore\ B_1 = 1.0$$

There's always a solution in steel

I-114

Example 1 (LRFD)

Determine the translation amplification factors

Translation, M_{lt},

Assume the frame deflection will be limited, in the final design, to

$$\Delta_H = L/400$$

There's always a solution in steel

I-115

Example 1 (LRFD)

For the entire frame at this story

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

$\sum P_{nt} = 2,450$ kips (total gravity load)

thus,

$$\Sigma P_{e2} = 0.85 \frac{\Sigma HL}{\Delta_H} = 0.85(150)(400) = 51,000 \text{ kips}$$

This is a measure of the frame stiffness

There's always a solution in steel

I-116

Example 1 (LRFD)

- Sway amplification factor

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$$B_2 = \frac{1}{1 - \frac{1.0(2,450)}{51,000}} = 1.05$$

There's always a solution in steel

I-117

Example 1 (LRFD)

Second-order force

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

$$P_u = (408) + 1.05(98) = 511 \text{ kips}$$

Second-order moment

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

$$M_u = 1.0(94.5) + 1.05(155) = 257 \text{ ft-kips}$$

There's always a solution in steel

I-118

Example 1 (ASD)

Determine the 2nd-order forces and moments for a W14x120 in a moment frame when the results of a 1st-order elastic analysis yields:

$$P_{nt} = 378 \text{ kips}$$
$$P_{lt} = 46 \text{ kips}$$
$$M_{ntA} = 43.9 \text{ ft-kips}$$
$$M_{ntB} = 87.8 \text{ ft-kips}$$
$$M_{ltA} = 36.0 \text{ ft-kips}$$
$$M_{ltB} = 72.0 \text{ ft-kips}$$

$$\boxed{\text{Load Case} = D + 0.75L + 0.75W}$$

There's always a solution in steel

I-119

Example 1 (ASD)

Determine the amplification factors for the no translation, $M_{nt,}$ member effect;

$$C_m = 0.6 - 0.4(M_1/M_2)$$
$$C_m = 0.6 - 0.4(43.9/87.8) = 0.4$$

There's always a solution in steel

I-120

Example 1 (ASD)

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

$$P_{e1} = \frac{\pi^2 (29{,}000)(1{,}380)}{\left(1.0(12.5)(12)\right)^2} = 17{,}600 \text{ kips}$$

There's always a solution in steel

I-121

Example 1 (ASD)

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

$$B_1 = \frac{0.4}{1 - \dfrac{1.6(378+46)}{17{,}600}} = 0.42 \not> 1.0 \therefore\ B_1 = 1.0$$

There's always a solution in steel

I-122

Example 1 (ASD)

Determine the amplification factors for translation, M_{lt}, the sway effect;

Assume the frame deflection will be limited, in the final design, to

$$\Delta_H = L/400$$

There's always a solution in steel

I-123

Example 1 (ASD)

For the entire frame at this story

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

$\sum P_{nt} = 2,270$ kips (total gravity load)

thus,

$$\Sigma P_{e2} = 0.85 \frac{\Sigma HL}{\Delta_H} = 0.85(150)(400) = 51,000 \text{ kips}$$

There's always a solution in steel

I-124

Example 1 (ASD)

- Sway amplification factor

$$B_2 = \frac{1}{1 - \dfrac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$$B_2 = \frac{1}{1 - \dfrac{1.6(2,270)}{51,000}} = 1.08$$

There's always a solution in steel

I-125

Example 1 (ASD)

Second-order force

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

$$P_a = (378) + 1.08(46.0) = 428 \text{ kips}$$

Second-order moment

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

$$M_a = 1.0(87.8) + 1.08(72.0) = 166 \text{ ft-kips}$$

There's always a solution in steel

I-126

Second-Order Analysis for the Design Office

- Exact solutions
  - Limited to fairly simple problems/elements.
- Rigorous Analysis
  - Many methods implemented in the wide variety of software packages available.

- SOME SOFTWARE

- Amplified 1st-Order Analysis
  - Equally acceptable and advantageous in some applications.

There's always a solution in steel

I-127

AISC 360

- Stability

B3.5 Design for Stability

"Stability of the structure and its elements shall be determined in accordance with Chapter C."

C1.1 Stability Design Requirements

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

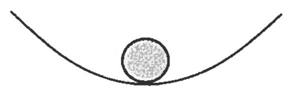
There's always a solution in steel

I-128

Stability

- *Stable Equilibrium*: ..."within certain limits, any slight change of the loading condition does not produce disproportionate increase of the stresses or the elastic distortions of the system."

H. Bleich
Buckling Strength of Metal Structures



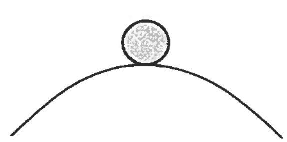
There's always a solution in steel

I-129

Stability

- *Unstable Equilibrium*:..."disproportionately large increases, indeterminate as to magnitude, to which deformations and stresses are subject at slight increases in load."

H. Bleich
Buckling Strength of Metal Structures

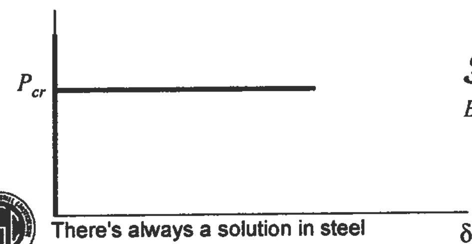


There's always a solution in steel

I-130

Stability

- *Bifurcation*: "Upon reaching the critical load there are two equilibrium positions possible, the straight form and a deflected form infinitesimally near it, both under the same axial load.



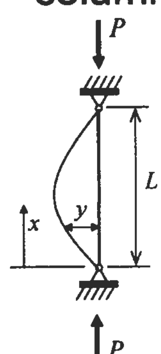
H. Bleich
Buckling Strength of Metal Structures

There's always a solution in steel

I-131

Stability

- Determine the elastic buckling load for a column.



•Assumptions
  •Perfectly elastic
  •Perfectly straight
  •Constant cross section
  •Pin ends
•Equilibrium at a point on a free body in the displaced configuration

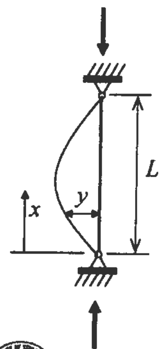
$$M_x = Py$$

There's always a solution in steel

I-132

Stability

- Determine the elastic buckling load for a column.



•From the principles of mechanics using small displacement theory

$$\frac{d^2y}{dx^2} = -\frac{M_x}{EI}$$

•Combining and rearranging terms

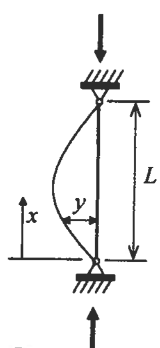
$$\frac{d^2y}{dx^2} + \frac{P}{EI}y = 0$$

There's always a solution in steel

I-133

Stability

- Determine the elastic buckling load for a column.



•Define

$$k^2 = \frac{P}{EI}$$

•The result is the differential equation of the column

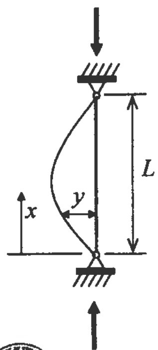
$$\frac{d^2y}{dx^2} + k^2y = 0$$

There's always a solution in steel

I-134

Stability

- Determine the elastic buckling load for a column.



•Solution

$$y = A \sin kx + B \cos kx$$

•From boundary conditions

$$B = 0$$

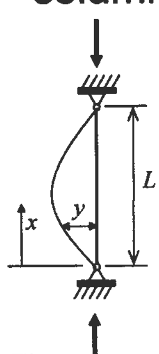
$$A \sin kL = 0$$

There's always a solution in steel

I-135

Stability

- Determine the elastic buckling load for a column.



•Thus,

$$\sin kL = 0$$

and

$$kL = n\pi$$

•The shape of the deflected column is

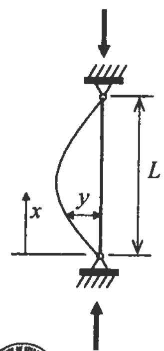
$$y = A \sin \frac{n\pi x}{L}$$

There's always a solution in steel

I-136

Stability

- Determine the elastic buckling load for a column.



•Remembering that

$$k^2 = \frac{P}{EI} \quad \text{and} \quad kL = n\pi$$

thus,

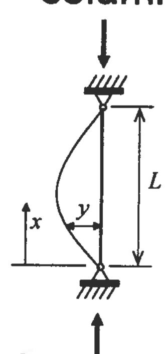
$$k^2 = \frac{P}{EI} = \frac{n^2\pi^2}{L^2}$$

There's always a solution in steel

I-137

Stability

- Determine the elastic buckling load for a column.



•The solution for this differential equation is

$$P = P_{cr} = \frac{n^2\pi^2 EI}{L^2}$$

which has a minimum value when

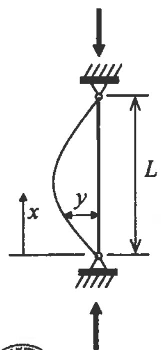
$$n = 1$$

There's always a solution in steel

I-138

Stability

- Determine the elastic buckling load for a column.



- Thus, we have the well known Euler Equation for the elastic buckling load:

$$P_{cr} = \frac{\pi^2 EI}{L^2}$$

There's always a solution in steel

I-139

Elastic Buckling Load

- The Perfect Column - The Euler Buckling Equation:

$$P_e = \frac{\pi^2 EI}{L^2}$$

- The real column, as part of a structure and with imperfections, etc.:

$$P_{cr} = P_e \times \text{(reduction factor)}$$

There's always a solution in steel

I-140

Definition of Effective Length Factor

- The reduction factor is defined as:

$$\text{reduction factor} = \frac{1}{K_{exact}^2}$$

- Thus, the elastic buckling load for a real column is given by:

$$P_{cr} = \frac{\pi^2 EI}{(K_{exact}L)^2}$$

There's always a solution in steel

I-141

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.

There's always a solution in steel

I-142

Elastic Buckling Analysis

- Eigenvalue Analysis
  - General form of the eigenvalue problem

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

$[K_o]=$ linear stiffness matrix

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

There's always a solution in steel

I-143

Elastic Buckling Analysis

- Eigenvalue Analysis
  - Reduction to standard form

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

$\{Y\}=$ eigenvector

$\omega=$ eigenvalue

$\lambda=\dfrac{1}{\omega}=$ load ratio

There's always a solution in steel

I-144

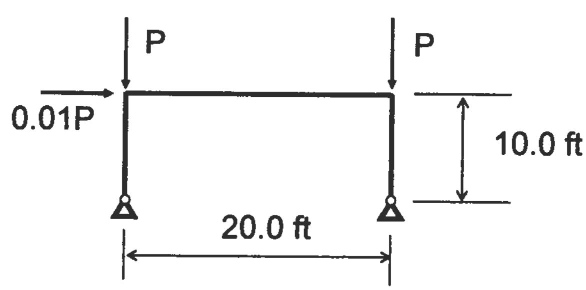
Elastic Buckling Analysis

- Eigenvalue Analysis
  - Solution techniques
    - Polynomial expansion
    - Power method
    - Iteration

There's always a solution in steel

I-145

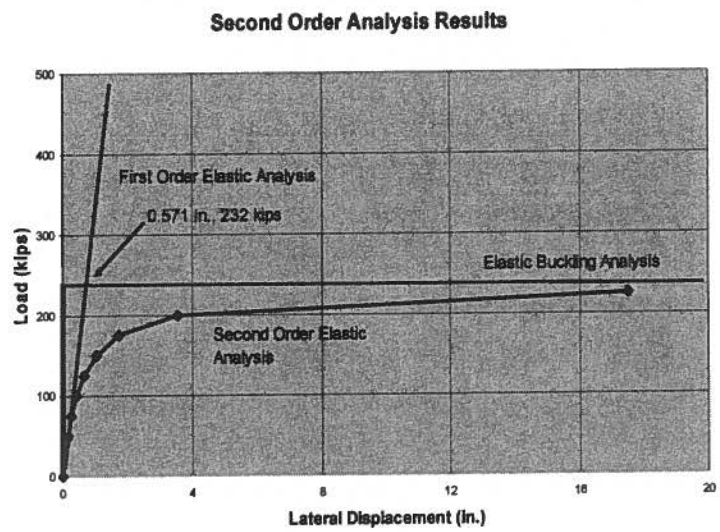
Elastic Buckling Analysis



All members W8x24

There's always a solution in steel

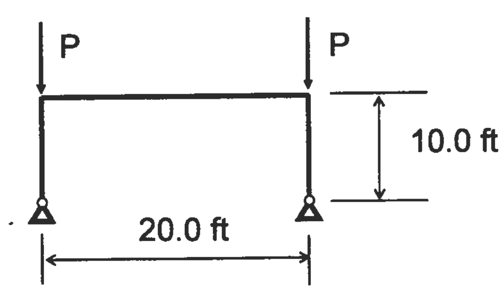
I-146



There's always a solution in steel

I-147

Elastic Buckling Analysis



All members W8x24

$I_x = 82.7$ in.^{4}

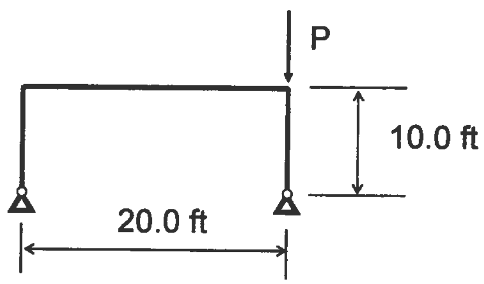
Elastic Buckling
P_{cr} =232 kips

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

There's always a solution in steel

I-148

Elastic Buckling Analysis



All members W8x24

$I_x = 82.7 \text{ in.}^4$

Elastic Buckling

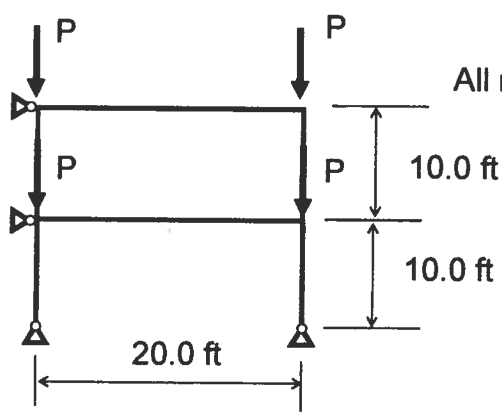
P_{cr} =460 kips

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

There's always a solution in steel

I-149

Elastic Buckling Analysis



All members W8x24

$I_x = 82.7 \text{ in.}^4$

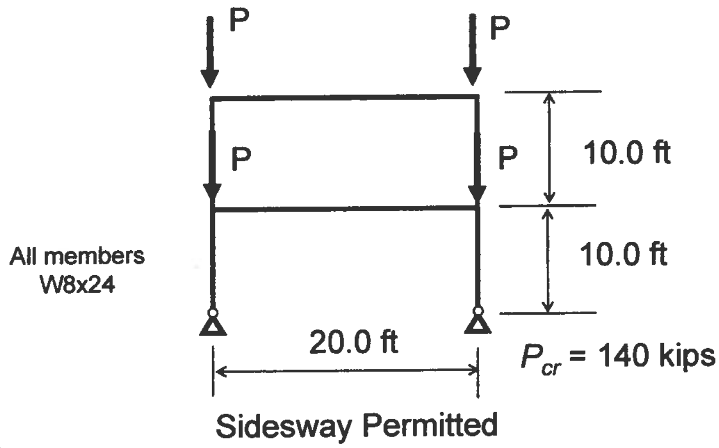
P_{cr}= 1378 kips

Sidesway Prevented

There's always a solution in steel

I-150

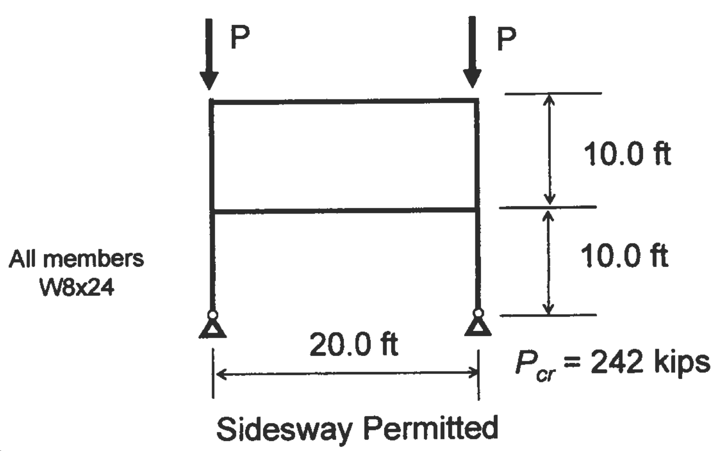
Elastic Buckling Analysis



There's always a solution in steel

I-151

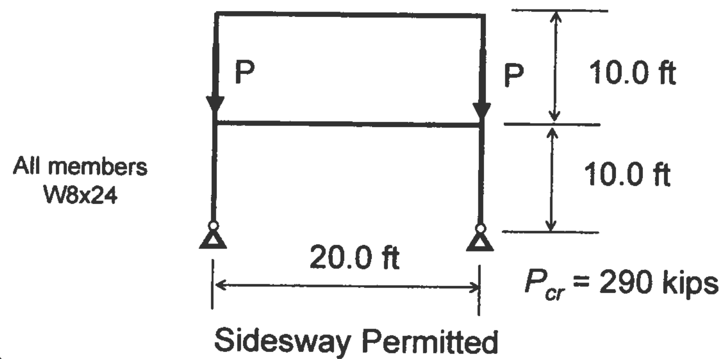
Elastic Buckling Analysis



There's always a solution in steel

I-152

Elastic Buckling Analysis



There's always a solution in steel

I-153

Effective Length Factor

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

There's always a solution in steel

I-154

Effective Length Factor

- Assumptions:
  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.

There's always a solution in steel

I-155

Effective Length Factor

- Assumptions:
  5. In sidesway permitted frames (moment frames), rotations at opposite ends of restraining beams are equal producing reverse curvature
  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

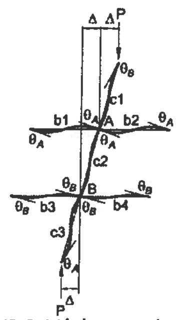
There's always a solution in steel

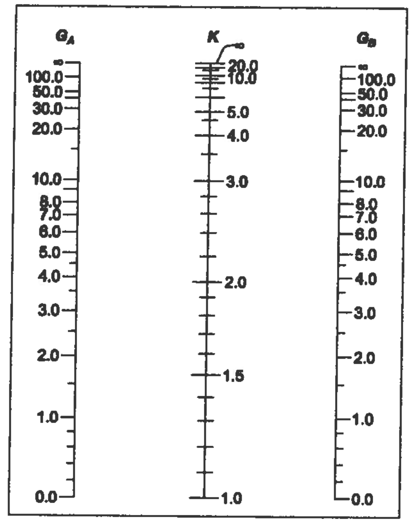
I-156

Effective Length

Nomograph or Alignment Chart for Moment Frame

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





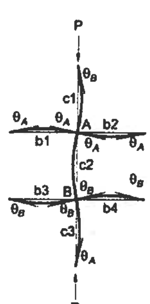
There's always a solution in steel

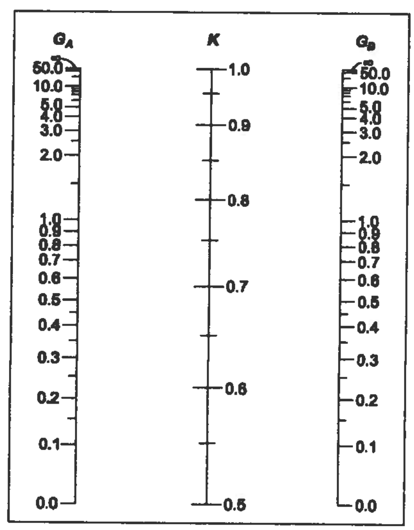
I-157

Effective Length

Nomograph or Alignment Chart for Braced Frame

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





There's always a solution in steel

I-158

Effective Length Factor

- Sidesway Inhibited (Braced Frames)

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

- Sidesway Permitted (Moment Frames)

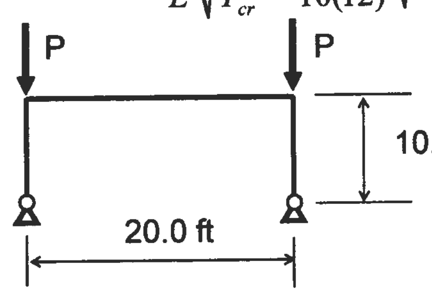
$$\frac{G_A G_B (\pi/K)^2 - 36}{6(G_A + G_B)} = \frac{\pi/K}{\tan \pi/K}$$

There's always a solution in steel

I-159

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

Nomograph
$K = 2.63$

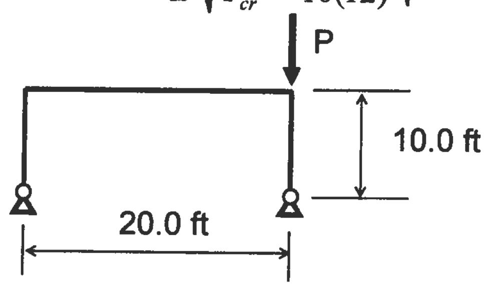
The structure and loading are close to satisfying the derivation assumptions. Thus, the results are close.

There's always a solution in steel

I-160

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
$K = 2.63$

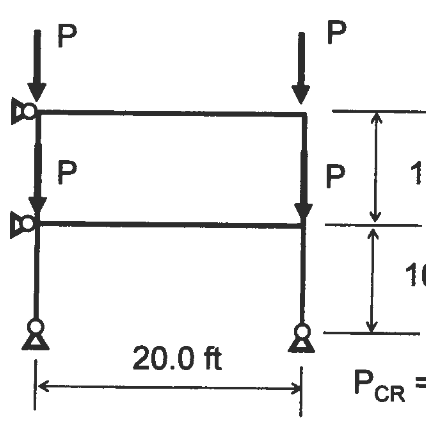
All members
W8x24

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

There's always a solution in steel

I-161

Buckling of two story frame



All members
W8x24

P_{CR} = 1378 kips

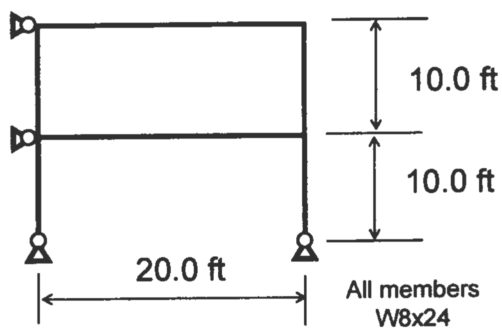
Sidesway Prevented

There's always a solution in steel

I-162

Buckling of two story frame

All members W8 x 24



Nomograph

$K_{\text{upper}} = 0.88$

$K_{\text{lower}} = 0.95$

Elastic Buckling

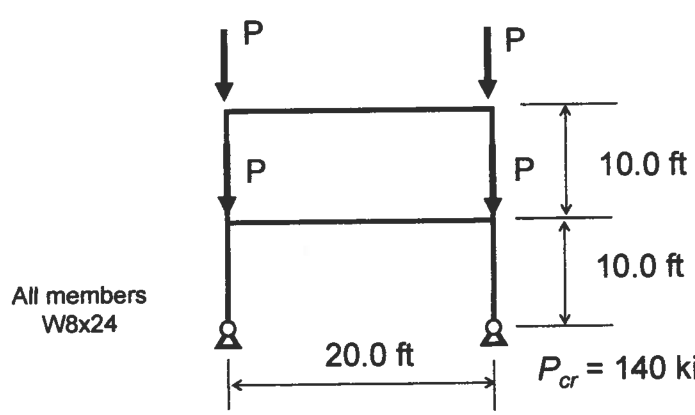
$K_{\text{upper}} = 1.09$

$K_{\text{lower}} = 0.77$

There's always a solution in steel

I-163

Buckling of two story frame



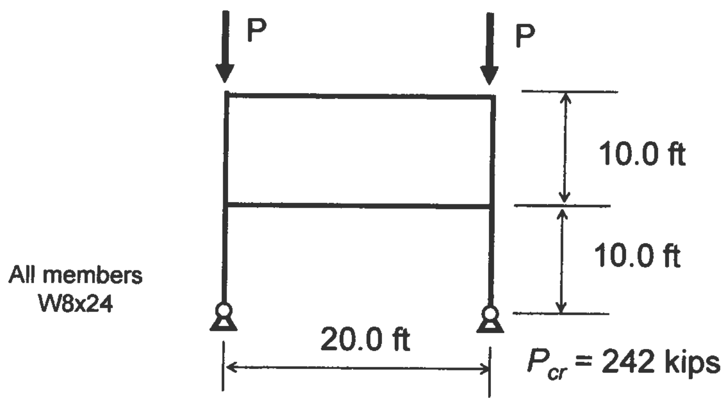
P_{cr} = 140 kips

Sidesway Permitted

There's always a solution in steel

I-164

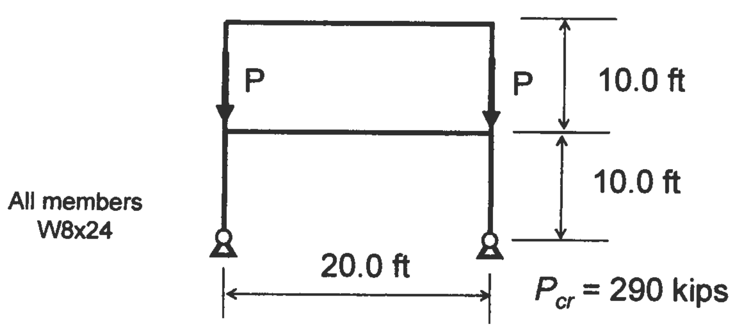
Buckling of two story frame



There's always a solution in steel

I-165

Buckling of two story frame

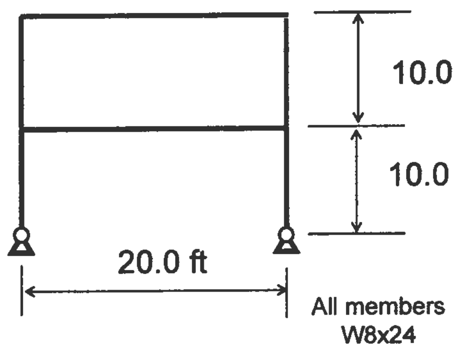


There's always a solution in steel

I-166

Buckling of two story frame

All members W8 x 24



Nomograph

$K_{upper} = 1.79$

$K_{lower} = 3.18$

There's always a solution in steel

I-167

Buckling of two story frame

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

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

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

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

Nomograph

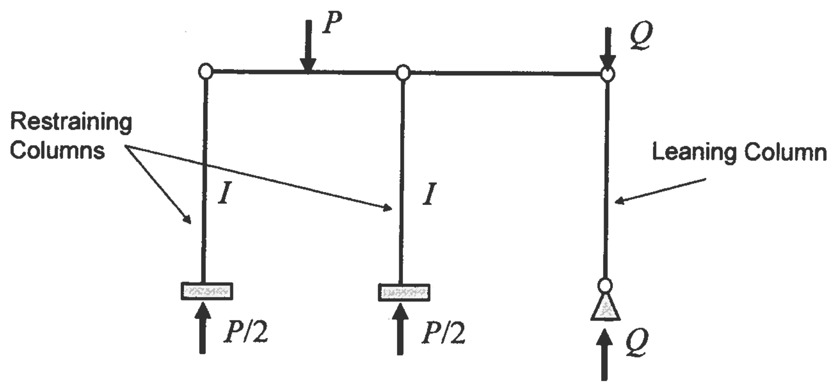
$K_{upper} = 1.79$

$K_{lower} = 3.18$

There's always a solution in steel

I-168

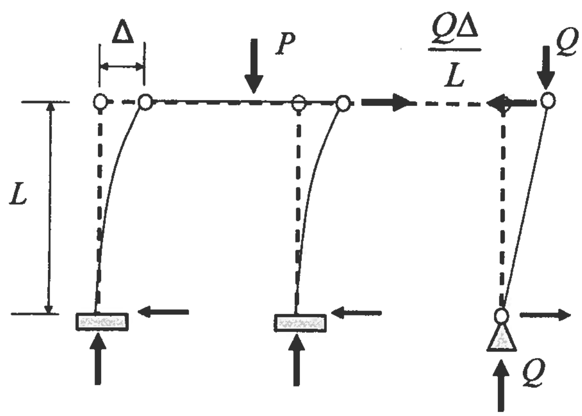
Buckling with Leaning Columns



There's always a solution in steel

I-169

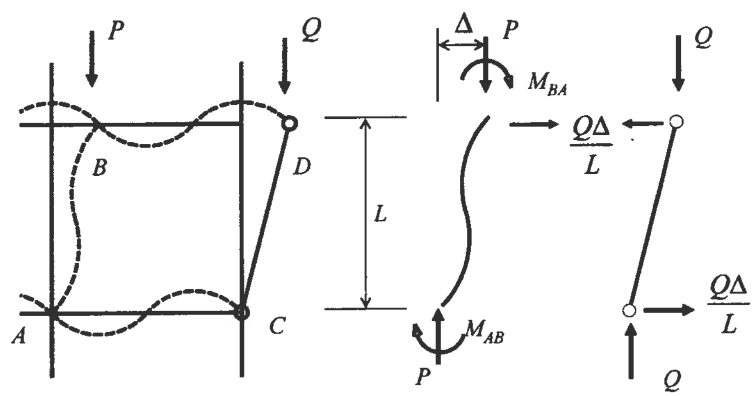
Buckling with Leaning Columns



There's always a solution in steel

I-170

Buckling with Leaning Columns



Using the same model as that used for developing the nomographs

There's always a solution in steel

I-171

Buckling with Leaning Columns

"Nomograph" type equation for sidesway uninhibited including leaning columns

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

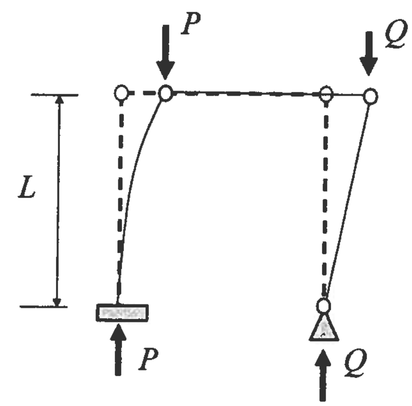
Q represents the sum of the load on all of the leaning columns attributed to the frame

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

There's always a solution in steel

I-172

Buckling with Leaning Columns



No load on leaning column
$Q = 0, K = 2.0$

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

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

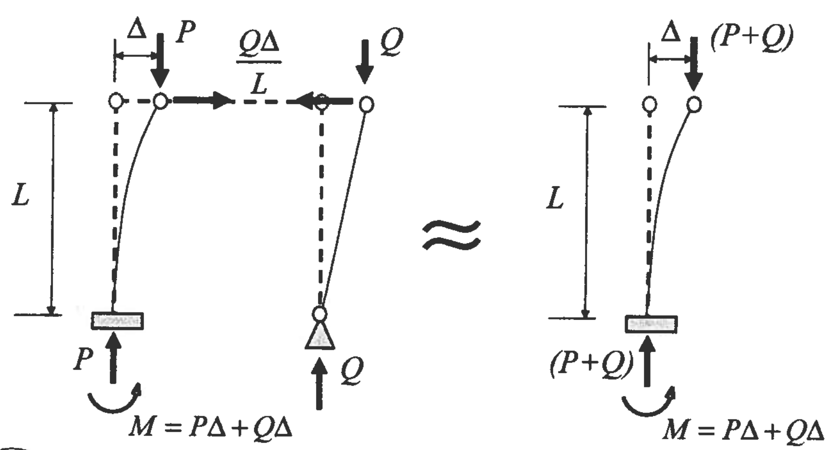
$Q/P = 10, K = 6.07$

What would you do if there were no load P?

There's always a solution in steel

I-173

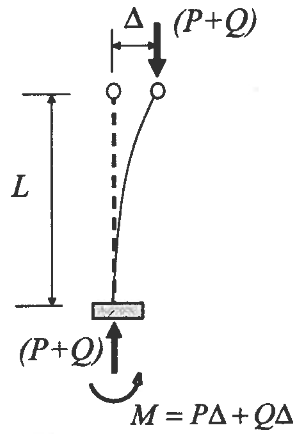
Buckling with Leaning Columns



There's always a solution in steel

I-174

Buckling with Leaning Columns



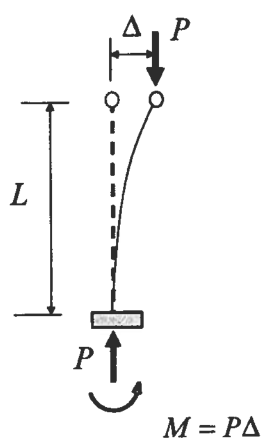
Design this column for the load $(P+Q)$ using the nomograph effective length factor, K_o.

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

There's always a solution in steel

I-175

Buckling with Leaning Columns



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

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

There's always a solution in steel

I-176

Buckling with Leaning 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

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

There's always a solution in steel

I-177

Effective Length Factor

- Problems with nomograph
  - Real structures rarely satisfy assumptions
    - Leaning 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

There's always a solution in steel

I-178

Stability

- What approach does your software use?

?

There's always a solution in steel

I-179

Stability

- What approach does your software use?
  - Calculates the elastic buckling load______
  - Calculates K based on the alignment chart equations_____
  - Requires K as an input variable for each column_____
  - Assumes a K based on a default_____
  - Assumes all $K = 1.0$______

There's always a solution in steel

I-180

AISC 360

C1.1 Stability Design Requirements

"Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of:

1. second-order effects,
2. flexural, shear and axial deformations,
3. geometric imperfections, and
4. member stiffness reduction due to residual stresses

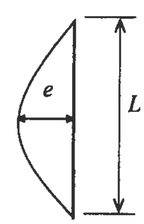
on the stability of the structure and its elements is permitted."

There's always a solution in steel

I-181

Geometric Imperfections

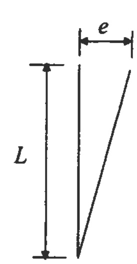
- Out-Of-Straightness



ASTM A6 Tolerance:

$e = L/1000$

- Out-Of-Plumbness



Code of Standard Practice Tolerance:

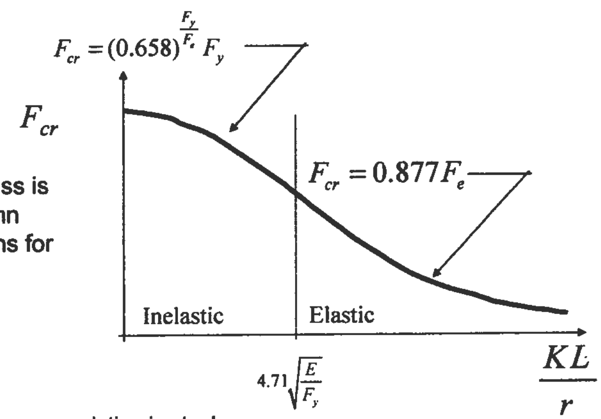
$e = L/500$

There's always a solution in steel

I-182

Geometric Imperfections

- Column Strength Equations



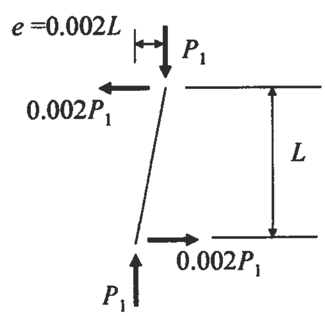
Out-of-straightness is included in column strength equations for both regions

There's always a solution in steel

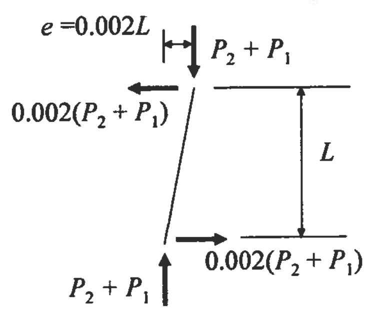
I-183

Geometric Imperfections

- Out-Of-Plumbness



Level 1, Top



Level 2, Next level down

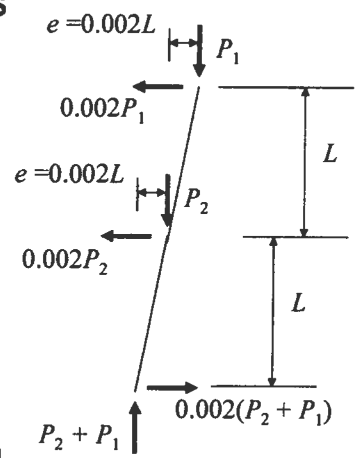
There's always a solution in steel

I-184

Geometric Imperfections

- Out-Of-Plumbness

Combine two levels



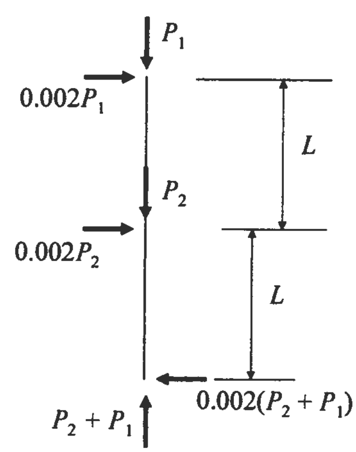
There's always a solution in steel

I-185

Geometric Imperfections

- Out-Of-Plumbness

Notional Loads



There's always a solution in steel

I-186

Geometric Imperfections

- Modeling Out-Of-Plumbness
  - The structure may be modeled in its out-of-plumb position directly.

or

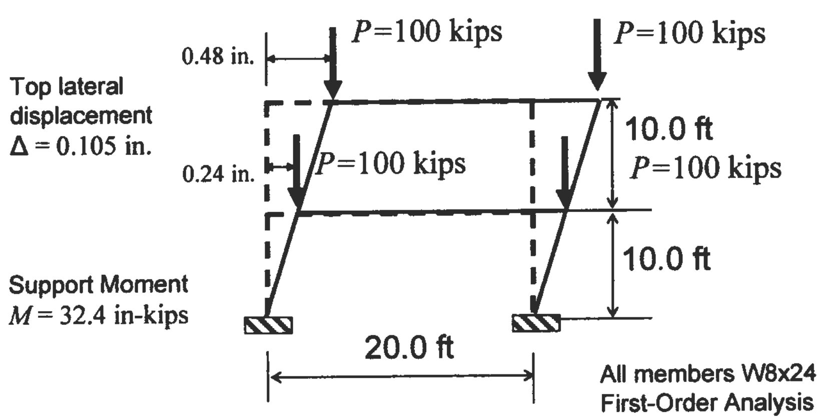
  - Notional loads may be used as a way to include the effect of the out of plumbness on a perfectly plumb structure.

There's always a solution in steel

I-187

Geometric Imperfections

- Out-Of-Plumbness

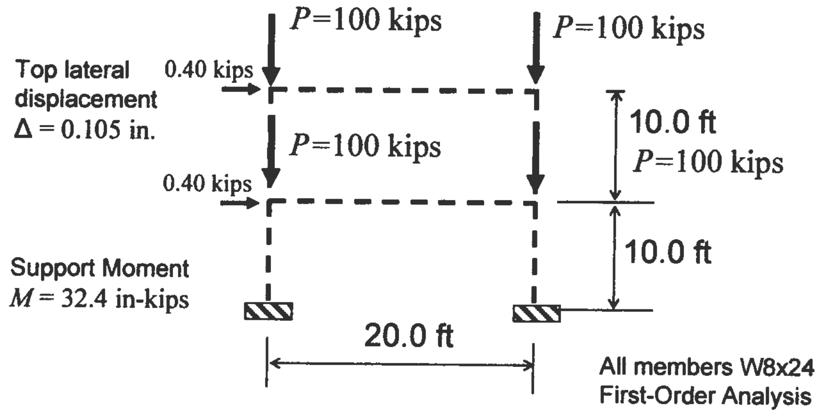


There's always a solution in steel

I-188

Geometric Imperfections

- Out-Of-Plumbness



There's always a solution in steel

I-189

Geometric Imperfections

- What approach does your software use?

?

There's always a solution in steel

I-190

Geometric Imperfections

- What approach does your software use?
  - Permits modeling of the out-of-plumb geometry_____
  - Automatically calculates and applies notional loads_____
  - Has no mechanism to include geometric imperfections_____

There's always a solution in steel

I-191

AISC 360

C1.1 Stability Design Requirements

"Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of:

1. second-order effects,
2. flexural, shear and axial deformations,
3. geometric imperfections, and
4. member stiffness reduction due to residual stresses

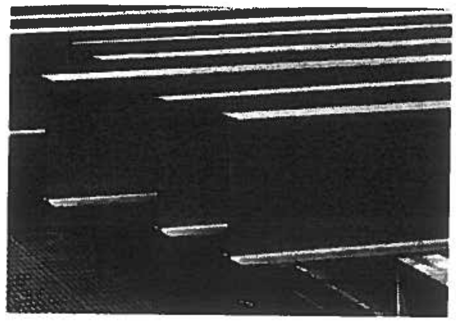
on the stability of the structure and its elements is permitted."

There's always a solution in steel

I-192

Residual Stress

- Source
  - Cooling during the production process



There's always a solution in steel

I-193

Residual Stress

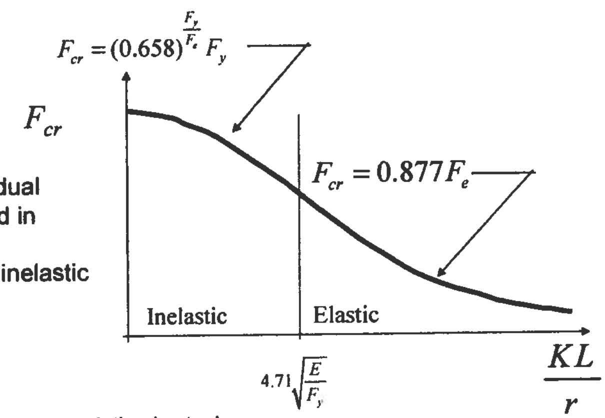
- Source
  - Cooling during the production process
- Influence
  - Member behaves inelastically at lower load than otherwise expected
    - Influences strength
  - Member deformation is greater than that predicted by elastic analysis
    - Influences second-order-effects

There's always a solution in steel

I-194

Residual Stress

- Column Strength Equations



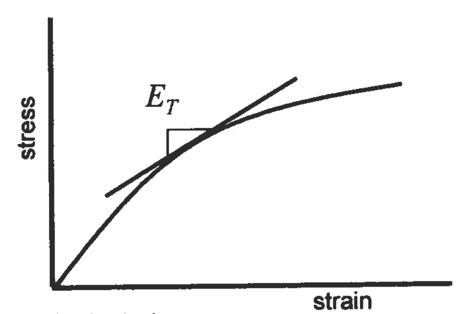
Influence of residual stresses included in column strength equations in the inelastic region only

There's always a solution in steel

I-195

Residual Stress

- Second-Order Effects
  - Stress-strain relationship no longer linear
  - Use the Tangent Modulus of Elasticity



There's always a solution in steel

I-196

Residual Stress

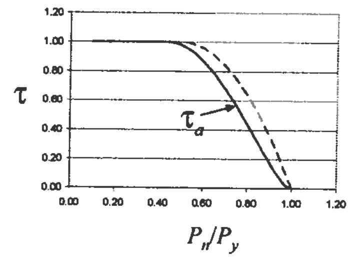
- Stiffness Reduction Factor (Commentary)

for effective length nomograph

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

$$\tau_a = 1.0 \quad \text{for } \frac{P_n}{P_y} \le 0.39$$

$$= -2.724\left(\frac{P_n}{P_y}\right)\ln\left(\frac{P_n}{P_y}\right) \quad \text{for } \frac{P_n}{P_y} > 0.39$$



There's always a solution in steel

I-197

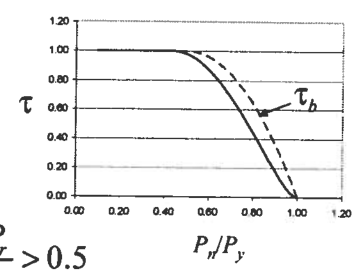
Residual Stress

- Stiffness Reduction Factor (Appendix 7)

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

$$\tau_b = 1.0 \quad \text{for } \frac{\alpha P_r}{P_y} \le 0.5$$

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



There's always a solution in steel

I-198

Residual Stress

- What approach does your software use?

?

There's always a solution in steel

I-199

Residual Stress

- What approach does your software use?
  - Automatically included in column strength equations _____
  - Impact on second-order effects included
    - Through use of R_m in calculation of B_2 _____
    - Through use of τ_b in a rigorous analysis_____
    - Through use of additional notional load _____
    - User defined values _____

There's always a solution in steel

I-200

AISC 360

C1.1 Stability Design Requirements

"Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of:

1. second-order effects,
2. flexural, shear and axial deformations,
3. geometric imperfections, and
4. member stiffness reduction due to residual stresses

on the stability of the structure and its elements is permitted."

There's always a solution in steel

I-201

AISC 360

C1.1 Stability Design Requirements

"The methods prescribed in this chapter and Appendix 7, Direct Analysis Method, satisfy these requirements."

There's always a solution in steel

I-202

AISC 360

C2.2a. Design by Second-Order Analysis
C2.2b. Design by First-Order Analysis
Appendix 7. Direct Analysis Method

or

Any method that gets the correct answer.

There's always a solution in steel

I-203

AISC 360

Summary

- What new considerations as a result of AISC 360-05;
  - Account for out-of-plumbness.
  - Account for impact of member second-order effects on sway second-order effects.
  - New opportunity to eliminate need to calculate the effective length factor.

There's always a solution in steel

I-204

Design for Combined Forces

Chapter H addresses members subject to axial force and flexure about one or both axes, with or without torsion, and to members subject to torsion only.

H1. Doubly- and Singly-Symmetric Members Subject to Flexure and Axial Force
H2. Unsymmetric and Other Members Subject to Flexure and Axial Force
H3. Members under Torsion and Combined Torsion, Flexure, Shear, and/or Axial Force

There's always a solution in steel

I-205

Design for Combined Forces

- 1989 ASD – 3 equations:

$$\frac{f_a}{F_a}+\frac{C_{mx}f_{bx}}{\left(1-\frac{f_a}{F'_{ex}}\right)F_{bx}}+\frac{C_{my}f_{by}}{\left(1-\frac{f_a}{F'_{ey}}\right)F_{by}}\leq 1.0$$

$$\frac{f_a}{0.6F_y}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}\leq 1.0$$

when $f_a/F_a \leq 0.15$

$$\frac{f_a}{F_a}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}\leq 1.0$$

There's always a solution in steel

I-206

Design for Combined Forces

- 1999 LRFD – 2 equations:

For $P_u/\phi P_n \geq 0.2$
$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

For $P_u/\phi P_n < 0.2$
$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

There's always a solution in steel

I-207

Design for Combined Forces

2005 - Doubly and Singly Symmetric Members – 2 equations:

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

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

There's always a solution in steel

I-208

Beam-Columns

- Definitions (ASD)

$P_r =$ required compressive strength (ASD)

$P_c = P_n / \Omega_c =$ allowable compressive strength

$M_r =$ required flexural strength (ASD)

$M_c = M_n / \Omega_b =$ allowable flexural strength

$\Omega_c = 1.67$

$\Omega_b = 1.67$

Determine required strengths by Chapter C

There's always a solution in steel

I-209

Beam-Columns

- Definitions (LRFD)

$P_r =$ required compressive strength (LRFD)

$P_c = \phi_c P_n =$ design compressive strength

$M_r =$ required flexural strength (LRFD)

$M_c = \phi_b M_n =$ design flexural strength

$\phi_c = 0.90$

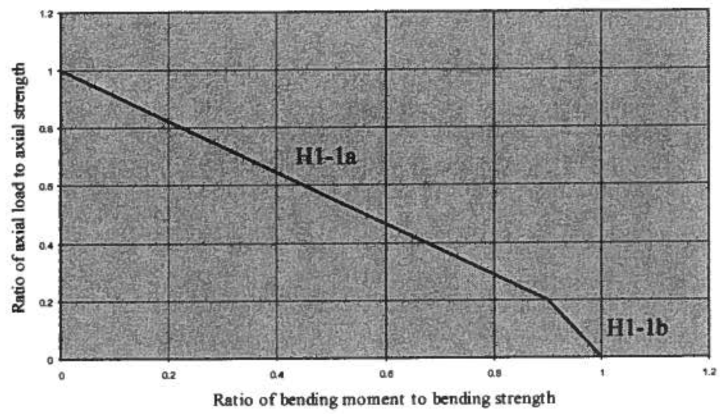
$\phi_b = 0.90$

Determine required strengths by Chapter C

There's always a solution in steel

I-210

Design for Combined Forces



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I-211

Design for Combined Forces

- In addition
  - H1.2 Double and Singly Symmetric Members in Flexure and Tension
  - H1.3 Doubly Symmetric Members in Single Axis Flexure and Compression
  - H2 Unsymmetric and Other Members Subject to Flexure and Axial Force

There's always a solution in steel

I-212

Design for Combined Forces

- Beam Column Design using Manual Tables
  - Part 6 of the Manual contains tables to assist in the design of members for combined forces
  - Available for W-shapes only
  - Entries included for all W-shapes
    - Could actually be used to design for pure bending, pure compression, and pure tension if desired.

There's always a solution in steel

I-213

Design for Combined Forces

Interaction Equations H1-1a and H1-1b

For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left[\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right] \leq 1.0$$

For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left[\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right] \leq 1.0$$

There's always a solution in steel

I-214

Design for Combined Forces

- These may be rewritten as

$$pP_r + b_x M_{rx} + b_y M_{ry} \leq 1.0 \qquad \text{(H1-1a)}$$

and

$$0.5pP_r + \frac{9}{8}\left(b_x M_{rx} + b_y M_{ry}\right) \leq 1.0 \qquad \text{(H1-1b)}$$

respectively

There's always a solution in steel

I-215

Design for Combined Forces

where

$$p = \frac{1}{P_c}$$

$$b_x = \frac{8}{9M_{cx}}$$

$$b_y = \frac{8}{9M_{cy}}$$

and units are 1/kips and 1/ft-kips

There's always a solution in steel

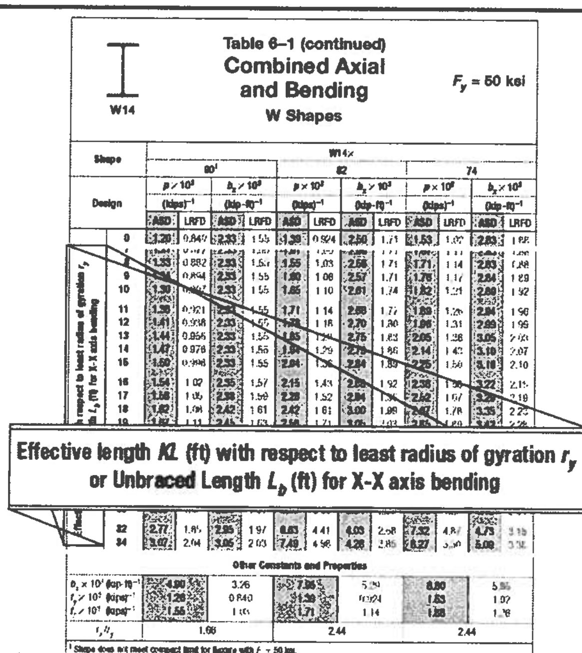
I-216

Design for Combined Forces

- For checking a specific section,
  - includes effective length for buckling
  - Includes unbraced length for bending
- Tables also include values for
  - tension yield and
  - tension rupture

There's always a solution in steel

I-217



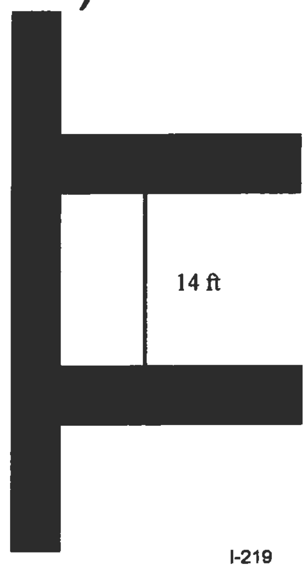
There's always a solution in steel

I-218

Example 2 (LRFD)

- Check the adequacy of an ASTM A992 W14x90 column subjected to an axial force of 500 kips, an *x*-axis bending moment of 253 ft-kips, and a y-axis bending moment of 30.0 ft-kips, from a second-order Direct Analysis (Appendix 7).

The column is 14 ft long, is bending about both axes, has a length of 14 ft about the *x*- and *y*-axis and an unbraced length of the compression flange of 14 ft.



There's always a solution in steel

I-219

Example 2 (LRFD)

Shape	W14×											
	90				82				74			
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
0	1.26	0.840	2.33	1.55	1.39	0.924	2.56	1.71	1.53	1.02	2.83	1.88
6	1.30	0.865	2.33	1.55	1.48	0.985	2.56	1.71	1.63	1.09	2.83	1.88
7	1.31	0.872	2.33	1.55	1.51	1.00	2.56	1.71	1.67	1.11	2.83	1.88
8	1.33	0.882	2.33	1.55	1.55	1.03	2.56	1.71	1.71	1.14	2.83	1.88
9	1.34	0.894	2.33	1.55	1.60	1.06	2.57	1.71	1.76	1.17	2.84	1.89
10	1.36	0.907	2.33	1.55	1.65	1.10	2.61	1.74	1.82	1.21	2.89	1.92
11	1.38	0.921	2.33	1.55	1.71	1.14	2.66	1.77	1.89	1.26	2.94	1.96
12	1.41	0.938	2.33	1.55	1.78	1.18	2.70	1.80	1.96	1.31	2.99	1.99
13	1.44	0.955	2.33	1.55	1.85	1.23	2.75	1.83	2.05	1.36	3.05	2.03
14	1.47	0.975	2.33	1.55	1.94	1.29	2.79	1.86	2.14	1.43	3.10	2.07
15	1.50	0.997	2.33	1.55	2.04	1.36	2.84	1.89	2.25	1.50	3.16	2.10
16	1.54	1.02	2.35	1.57	2.15	1.43	2.89	1.92	2.38	1.58	3.22	2.15
17	1.58	1.05	2.38	1.58	2.28	1.52	2.94	1.96	2.52	1.67	3.29	2.19
18	1.62	1.08	2.42	1.61	2.42	1.61	3.00	1.99	2.67	1.78	3.36	2.23
19	1.67	1.11	2.45	1.63	2.58	1.71	3.05	2.03	2.85	1.89	3.42	2.28
20	1.72	1.14	2.49	1.66	2.75	1.83	3.11	2.07	3.04	2.02	3.50	2.33
22	1.83	1.22	2.56	1.70	3.18	2.12	3.23	2.15	3.51	2.34	3.65	2.43
24	1.97	1.31	2.62	1.74	3.73	2.48	3.37	2.24	4.12	2.74	3.82	2.54
26	2.12	1.41	2.70	1.79	4.38	2.91	3.51	2.34	4.83	3.22	4.00	2.66
28	2.31	1.53	2.78	1.85	5.08	3.38	3.67	2.44	5.61	3.73	4.20	2.80
30	2.52	1.68	2.86	1.91	5.83	3.88	3.84	2.55	6.44	4.28	4.42	2.94
32	2.77	1.85	2.96	1.97	6.63	4.41	4.03	2.68	7.32	4.87	4.73	3.15
34	3.07	2.04	3.06	2.03	7.49	4.98	4.28	2.85	8.27	5.50	5.00	3.33
36	3.43	2.28	3.16	2.10	8.39	5.59	4.57	3.04	9.27	6.17	5.44	3.62
38	3.82	2.54	3.27	2.17	9.35	6.22	4.86	3.24	10.3	6.87	5.80	3.86
40	4.23	2.81	3.38	2.25	10.4	6.90	5.15	3.43	11.4	7.61	6.15	4.09

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

Other Constants and Properties

	90 ASD	90 LRFD	82 ASD	82 LRFD	74 ASD	74 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	4.80	3.19	7.95	5.29	8.80	5.85
$t_y \times 10^3$ (kips)$^{-1}$	1.26	0.840	1.39	0.924	1.53	1.02
$t_r \times 10^3$ (kips)$^{-1}$	1.56	1.04	1.71	1.14	1.88	1.25

There's always a solution in steel

I-220

Example 2(LRFD)

Determine which equation to use:

$$pP_u = 0.976 \times 10^{-3}(500) = 0.49 > 0.2$$

therefore use H1-1a

$$pP_u + b_x M_{ux} + b_y M_{uy} \leq 1.0$$

$$0.976 \times 10^{-3}(500) + 1.55 \times 10^{-3}(253) + 3.26 \times 10^{-3}(30.0) = 0.98 < 1.0$$

thus, the W14x90 is adequate

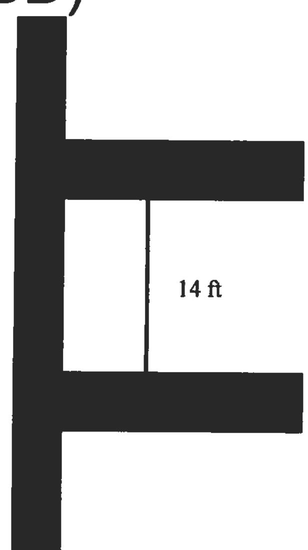
There's always a solution in steel

I-221

Example 2 (ASD)

- Check the adequacy of an ASTM A992 W14x90 column subjected to an axial force of 333 kips, an *x*-axis bending moment of 169 ft-kips, and a y-axis bending moment of 20 ft-kips, from a second-order Direct Analysis (Appendix 7).

The column is 14 ft long, is bending about both axes, has a length of 14 ft about the *x*- and *y*-axis and an unbraced length of the compression flange of 14 ft.



There's always a solution in steel

I-222

Example 2 (ASD)

Shape		W14×											
		90[f]				82				74			
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	1.26	0.840	2.33	1.55	1.39	0.924	2.56	1.71	1.53	1.02	2.83	1.88
	6	1.30	0.863	2.33	1.55	1.45	0.963	2.56	1.71	1.63	1.09	2.83	1.88
	7	1.31	0.872	2.33	1.55	1.51	1.00	2.56	1.71	1.67	1.11	2.83	1.88
	8	1.33	0.882	2.33	1.55	1.56	1.03	2.56	1.71	1.71	1.14	2.83	1.88
	9	1.34	0.894	2.33	1.55	1.60	1.06	2.57	1.71	1.76	1.17	2.84	1.89
	10	1.36	0.907	2.33	1.55	1.65	1.10	2.61	1.74	1.82	1.21	2.89	1.92
	11	1.38	0.921	2.33	1.55	1.71	1.14	2.66	1.77	1.89	1.26	2.94	1.96
	12	1.41	0.938	2.33	1.55	1.78	1.18	2.70	1.80	1.96	1.31	2.99	1.99
	13	1.44	0.958	2.33	1.55	1.85	1.23	2.75	1.83	2.05	1.36	3.05	2.03
	14	1.47	0.978	2.33	1.55	1.94	1.29	2.79	1.86	2.14	1.43	3.10	2.07
	15	1.50	[illegible]	2.33	1.55	2.04	1.36	2.84	1.89	2.25	1.50	3.16	2.10
	16	1.54	1.02	2.35	1.57	2.15	1.43	2.89	1.92	2.38	1.58	3.22	2.15
	17	1.58	1.05	2.38	1.59	2.28	1.52	2.94	1.96	2.52	1.67	3.29	2.19
	18	1.62	1.08	2.42	1.61	2.42	1.61	3.00	1.99	2.67	1.78	3.35	2.23
	19	1.67	1.11	2.45	1.63	2.58	1.71	3.05	2.03	2.85	1.89	3.42	2.28
	20	1.72	1.14	2.48	1.65	2.75	1.83	3.11	2.07	3.04	2.02	3.50	2.33
	22	1.83	1.22	2.55	1.70	3.18	2.12	3.23	2.15	3.51	2.34	3.65	2.43
	24	1.97	1.31	2.62	1.74	3.75	2.48	3.37	2.24	4.12	2.74	3.82	2.54
	26	2.12	1.41	2.70	1.79	4.38	2.91	3.51	2.34	4.83	3.22	4.00	2.66
	28	2.31	1.53	2.78	1.85	5.08	3.38	3.67	2.44	5.61	3.73	4.20	2.80
	30	2.52	1.68	2.88	1.91	5.83	3.88	3.84	2.55	6.44	4.28	4.42	2.94
	32	2.77	1.85	2.95	1.97	6.63	4.41	4.03	2.68	7.32	4.87	4.73	3.15
	34	3.07	2.04	3.05	2.03	7.49	4.98	4.28	2.85	8.27	5.50	5.08	3.38
	36	3.43	2.28	3.16	2.10	8.39	5.58	4.57	3.04	9.27	6.17	5.44	3.62
	38	3.82	2.54	3.27	2.17	9.35	6.22	4.86	3.24	10.3	6.87	5.80	3.86
	40	4.23	2.81	3.38	2.25	10.4	6.90	5.15	3.43	11.4	7.61	6.15	4.09
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		4.90		3.26		7.95		5.29		8.80		5.85	
$t_y \times 10^3$ (kips)$^{-1}$		1.26		0.840		1.39		0.924		1.53		1.02	
$t_r \times 10^3$ (kips)$^{-1}$		1.55		1.03		1.71		1.14		1.88		1.25	

There's always a solution in steel

I-223

Example 2 (ASD)

Determine which equation to use:

$$pP_a = 1.47\times10^{-3}(333) = 0.49 > 0.2$$

therefore use H1-1a

$$pP_a + b_xM_{ax} + b_yM_{ay} \le 1.0$$

$$1.47\times10^{-3}(333) + 2.33\times10^{-3}(169) + 4.90\times10^{-3}(20.0) = 0.98 < 1.0$$

thus, the W14x90 is adequate

There's always a solution in steel

I-224

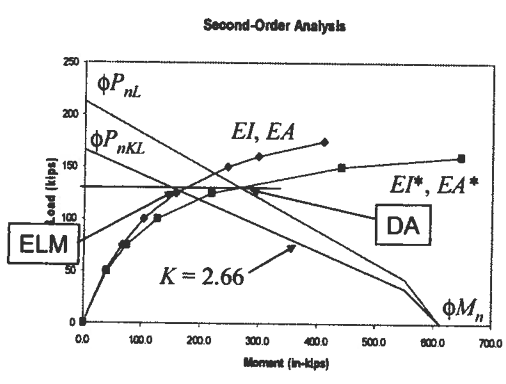
Effective Length vs. Actual Length

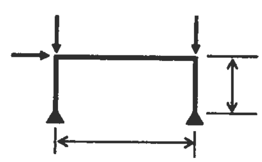
- 1960 AISC Specification introduced "effective length."
- 1963 AISC Specification introduced "effective length factor," K.
- 2005 AISC Specification introduced the concept of using $K = 1$ for moment frames.

There's always a solution in steel

I-225

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.

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I-226

Thank You

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Chicago, IL 60601

There's always a solution in steel

I-227

Chapter
IBC-17
CERTIFICATION

Part II – How you go about doing it!

- Direct Analysis Method 2nd order
- Effective Length Method K - calculation 2nd
- First-order Analysis Method
- Simplified Methods
- Building Examples
  - Warehouse
  - 4-story Commercial

There's always a solution in steel

II-1

IBC - Chap 17 IN
PROF ENGRS CODE

Stability Analysis and Design

- Determination of required strength
  - Three approaches are available in the specification
    - Appendix 7: Direct Analysis
      - This is the foundation for the other approaches presented
    - Section C2.2a: Second-order Analysis
      - This is called the Effective Length Method
    - Section C2.2b: First-order Analysis
      - This is the simplest approach if applicable

There's always a solution in steel

II-2

Direct Analysis

- Appendix 7
  - Applicable to all types of structures
  - Does not distinguish between systems
    - Braced frames
    - Moment frames
    - Shear wall systems
    - Any combination of systems
  - A powerful method with no K-factors: The future is here

There's always a solution in steel

II-3

Direct Analysis

- Second-order analysis
  - Use any second-order elastic analysis that considers P-Δ and P-δ effects
  - Options:
    - Any general second-order analysis method
    - Amplified first-order analysis (B_1–B_2)

Remember that the $B_1 - B_2$ method is a second-order analysis

There's always a solution in steel

II-4

Direct Analysis

- Use a reduced flexural and axial stiffness

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

to account for influence of inelasticity on second-order effects and to permit use of $K = 1.0$.

LOOK AT LIMIT States

There's always a solution in steel

II-5

Direct Analysis

- Inelastic response
  - Depends on the level of axial stress in the member

    when $\alpha P_r \leq 0.5P_y$; $\tau_b = 1.0$

    when $\alpha P_r > 0.5P_y$; $\tau_b = 4\left[\frac{\alpha P_r}{P_y}\left(1 - \frac{\alpha P_r}{P_y}\right)\right]$

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

There's always a solution in steel

II-6

Direct Analysis

- Apply notional loads, N_i, where

$$N_i = 0.002Y_i$$

Y_i = the total gravity load on that story

This accounts for an initial out-of-plumbness at the maximum of 1/500 as defined in the COSP. If a lesser out-of-plumbness is known, N_i can be reduced.

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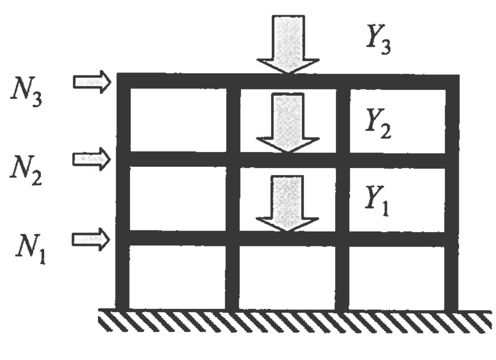
II-7

Direct Analysis



$$N_i = 0.002Y_i$$

There's always a solution in steel

II-8

Direct Analysis

- If the second-order effects are limited,

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

these notional loads can be treated as a minimum, otherwise they are added to all load cases.

There's always a solution in steel

II-9

Direct Analysis

- Design members using the provisions for individual members
  - Chapters E, F, G, H, I
- For compression members, Chapter E
  - use $K = 1.0$ for determining compressive strength

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II-10

Direct Analysis

- Analysis/Design process summary
  - Apply notional loads, $N_i = 0.002Y_i$
    - As a minimum lateral load if $B_2 \leq 1.5$
    - As an additional lateral load if $B_2 > 1.5$
  - Perform second-order analysis
    - Use nominal geometry
    - Use reduced stiffness, EI^* and EA^*
  - Design members using $K=1$ for compression

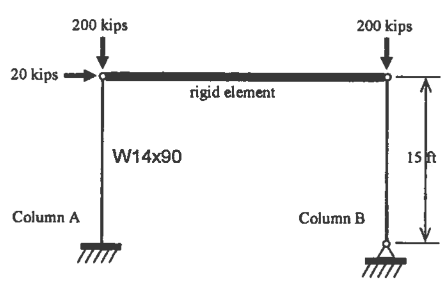
There's always a solution in steel

II-11

Example 3 (LRFD)

DA
DLW

- Design by Direct Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$$\boxed{1.2D + 0.5L + 1.6W}$$

There's always a solution in steel

II-12

Example 3 (LRFD)

DA
DLW

- Design by Direct Analysis
  - Notional load

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

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

  - Assume

$$B_2 \leq 1.5$$

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

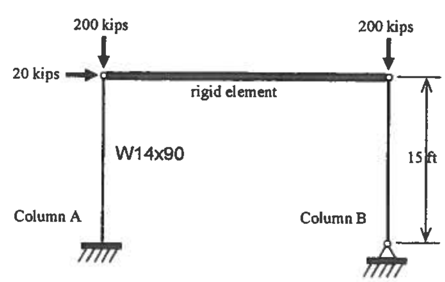
There's always a solution in steel

II-13

Example 3 (LRFD)

DA
DLW

- First-order elastic analysis with reduced stiffness



$$P_{nt} = 200 \text{ kips}$$
$$P_{lt} = 0 \text{ kips}$$
$$M_{ntx} = 0 \text{ ft-kips}$$
$$M_{ltx} = 300 \text{ ft-kips}$$
$$K_x = 1.0$$
$$K_y = 1.0$$
$$L_b = 15 \text{ ft}$$
$$C_b = 1.67$$

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

There's always a solution in steel

II-14

Example 3 (LRFD)

DA
DLW

- Stiffness reduction $\tau_b = 1.0$ for $\dfrac{\alpha P_r}{P_y} \le 0.5$

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

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

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

There's always a solution in steel

II-15

Example 3 (LRFD)

DA
DLW

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

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

$$C_m = 0.6 - 0.4(0/300) = 0.6$$

There's always a solution in steel

II-16

Example 3 (LRFD)

DA
DLW

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

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

$$P_{e1} = \frac{\pi^2 (0.8)(29{,}000)(999)}{\left(1.0(15.0)(12)\right)^2} = 7{,}060 \text{ kips}$$

There's always a solution in steel

II-17

Example 3 (LRFD)

DA
DLW

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

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

$$B_1 = \frac{0.6}{1 - \dfrac{(1.0)(200)}{7{,}060}} = 0.62 \ngtr 1.0 \therefore \; B_1 = 1.0$$

There's always a solution in steel

II-18

Example 3 (LRFD)

DA
DLW

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

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$$\Sigma P_{nt} = 400 \text{ kips}$$

Must be matched

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

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

There's always a solution in steel

II-19

Example 3 (LRFD)

DA
DLW

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

$$\Sigma P_{e2} = 0.85 \frac{\Sigma HL}{\Delta_H} = 0.85 \frac{(20)(15.0(12))}{1.68} = 1{,}830 \text{ kips}$$

There's always a solution in steel

II-20

Example 3 (LRFD)

DA
DLW

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

$$B_2 = \frac{1}{1 - \frac{(1.0)(400)}{1{,}830}} = 1.28$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads may be taken as 1.7 rather than 1.5.

There are no restrictions on this method

There's always a solution in steel

II-21

Example 3 (LRFD)

DA
DLW

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$M_u = 1.0(0.0) + 1.28(300) = 384 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

$$P_u = (200) + 1.28(0.0) = 200 \text{ kips}$$

There's always a solution in steel

II-22

Example 3 (LRFD)

DA
DLW

- Determine member strength

$K_x L = 15.0$ ft $\phi_c P_n = 1000$ kips

$K_y L = 15.0$ ft

$L_b = 15.0$ ft $\phi_b M_n = 573$ ft-kips

$\boxed{p = 0.998\times10^{-3} \quad b_x = 1.55\times10^{-3}}$

- Interaction Eq. H1-1a

$$\frac{200}{1,000}+\frac{8}{9}\left(\frac{384}{573}\right)=0.796<1.0 \;\therefore\; \text{ok}$$

$$\boxed{0.998\times10^{-3}(200)+1.55\times10^{-3}(384)=0.795<1.0 \;\therefore\; \text{ok}}$$

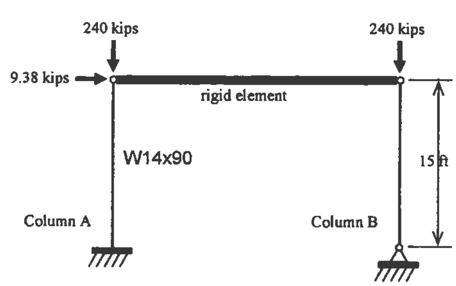
There's always a solution in steel

II-23

Example 3 (ASD)

DA
DLW

- Design by Direct Analysis



$P_D = 75.0$ kips

$P_L = 220$ kips

$W = 12.5$ kips

$$\boxed{D+0.75L+0.75W}$$

There's always a solution in steel

II-24

Example 3 (ASD)

DA
DLW

- Design by Direct Analysis
  - Notional load

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

$$N_i = 0.002(480) = 0.960 \text{ kips}$$

  - Assume

$$B_2 \leq 1.5$$

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

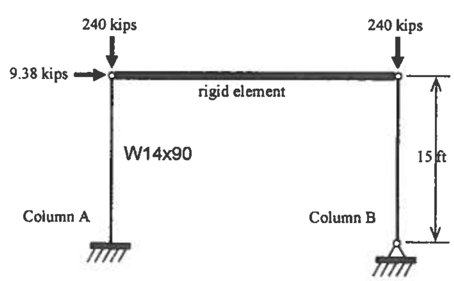
There's always a solution in steel

II-25

Example 3 (ASD)

DA
DLW

- First-order elastic analysis with reduced stiffness



$$P_{nt} = 240 \text{ kips}$$

$$P_{lt} = 0 \text{ kips}$$

$$M_{ntx} = 0 \text{ ft-kips}$$

$$M_{ltx} = 141 \text{ ft-kips}$$

$$K_x = 1.0$$

$$K_y = 1.0$$

$$L_b = 15 \text{ ft}$$

$$C_b = 1.67$$

$$\Delta_{1st} = \frac{Pl^3}{3EI^*} = \frac{9.38(15)^3(1{,}728)}{3(0.8)(29{,}000)(999)} = 0.786 \text{ in.}$$

There's always a solution in steel

II-26

Example 3 (ASD)

DA
DLW

- Stiffness reduction $\tau_b = 1.0$ for $\dfrac{\alpha P_r}{P_y} \le 0.5$

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

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

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

There's always a solution in steel

II-27

Example 3 (ASD)

DA
DLW

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

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

$$C_m = 0.6 - 0.4(0/141) = 0.6$$

There's always a solution in steel

II-28

Example 3 (ASD)

DA
DLW

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

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

$$P_{e1} = \frac{\pi^2 (0.8)(29{,}000)(999)}{\left(1.0(15.0)(12)\right)^2} = 7{,}060 \text{ kips}$$

There's always a solution in steel

II-29

Example 3 (ASD)

DA
DLW

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

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

$$B_1 = \frac{0.6}{1 - \frac{(1.6)(240)}{7{,}060}} = 0.63 \not\geq 1.0 \therefore\ B_1 = 1.0$$

There's always a solution in steel

II-30

Example 3 (ASD)

DA
DLW

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

$$B_2 = \frac{1}{1-\frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$\Sigma P_{nt} = 480$ kips

Must be matched

$\Sigma H = 9.38$ kips

$\Delta_H = 0.787$ in.

There's always a solution in steel

II-31

Example 3 (ASD)

DA
DLW

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

$$\Sigma P_{e2} = 0.85\frac{\Sigma HL}{\Delta_H} = 0.85\frac{(9.38)(15.0(12))}{0.786} = 1{,}830 \text{ kips}$$

There's always a solution in steel

II-32

Example 3 (ASD)

DA
DLW

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

$$B_2 = \frac{1}{1 - \frac{(1.6)(480)}{1,830}} = 1.72$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads may be taken as 1.7 rather than 1.5. This exceeds 1.7 so we must go back and include the notional load.

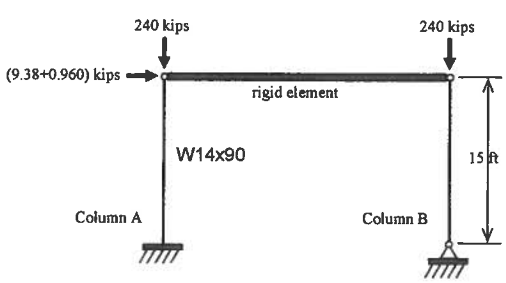
There's always a solution in steel

II-33

Example 3 (ASD)

DA
DLW

- First-order elastic analysis with reduced stiffness



$P_{nt} = 240$ kips

$P_{lt} = 0$ kips

$M_{ntx} = 0$ ft-kips

$M_{ltx} = 155$ ft-kips

$K_x = 1.0$

$K_y = 1.0$

$L_b = 15$ ft

$C_b = 1.67$

$$\Delta_{1st} = \frac{Pl^3}{3EI^*} = \frac{9.38(15)^3(1,728)}{3(0.8)(29,000)(999)} = 0.786 \text{ in.}$$

Could modify Δ but don't need to.

There's always a solution in steel

II-34

Example 3 (ASD)

DA
DLW

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$M_u = 1.0(0.0) + 1.72(155) = 267 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

$$P_u = (240) + 1.72(0.0) = 240 \text{ kips}$$

There's always a solution in steel

II-35

Example 3 (ASD)

DA
DLW

- Determine member strength

$K_x L = 15.0$ ft $\quad \dfrac{P_n}{\Omega_c} = 667$ kips

$K_y L = 15.0$ ft

$L_b = 15.0$ ft $\quad \dfrac{M_n}{\Omega_b} = 382$ ft-kips

$p = 1.50\times10^{-3}$

$b_x = 2.33\times10^{-3}$

- Interaction Eq. H1-1a

$$\frac{240}{667} + \frac{8}{9}\left(\frac{267}{382}\right) = 0.981 < 1.0 \;\therefore\; \text{ok}$$

$$\boxed{1.50\times10^{-3}(240) + 2.33\times10^{-3}(267) = 0.982 < 1.0 \;\therefore\; \text{ok}}$$

There's always a solution in steel

II-36

LRFD – ASD Comparison 3

DA
DLW

- Differences
  - Load Combinations

	LRFD	ASD
	$1.2D + 0.5L + 1.6W$	$D + 0.75L + 0.75W$
	$P_u = 200$ kips	$P_a = 240$ kips

  - Structure Amplification

	LRFD	ASD
	$\alpha = 1.0$	$\alpha = 1.6$
	$B_2 = 1.28$	$B_2 = 1.73$

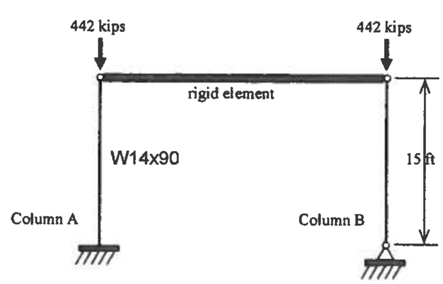
There's always a solution in steel

II-37

Example 4 (LRFD)

DA
DL

- Design by Direct Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$\boxed{1.2D + 1.6L}$

There's always a solution in steel

II-38

Example 4 (LRFD)

DA
DL

- Design by Direct Analysis
  - Notional load

$$Y_i = (442 + 442) = 884 \text{ kips}$$
$$N_i = 0.002(884) = 1.77 \text{ kips}$$

  - Since there is no lateral load, the notional load must be applied.

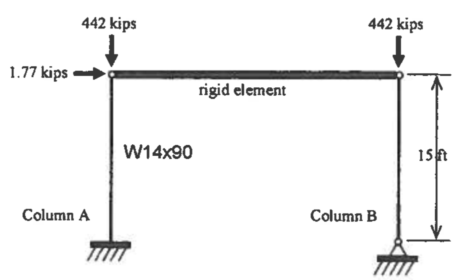
There's always a solution in steel

II-39

Example 4 (LRFD)

DA
DL

- 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
$C_b = 1.67$

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

There's always a solution in steel

II-40

Example 4 (LRFD)

DA
DL

- Stiffness reduction $\tau_b = 1.0$ for $\dfrac{\alpha P_r}{P_y} \le 0.5$

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

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

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

There's always a solution in steel

II-41

Example 4 (LRFD)

DA
DL

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

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

$$C_m = 0.6 - 0.4(0/26.6) = 0.6$$

There's always a solution in steel

II-42

Example 4 (LRFD)

DA
DL

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

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

$$P_{e1} = \frac{\pi^2 (0.8)(29,000)(999)}{(1.0(15.0)(12))^2} = 7,060 \text{ kips}$$

There's always a solution in steel

II-43

Example 4 (LRFD)

DA
DL

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

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

$$B_1 = \frac{0.6}{1 - \frac{(1.0)(442)}{7,060}} = 0.64 \not> 1.0 \therefore \; B_1 = 1.0$$

There's always a solution in steel

II-44

Example 4 (LRFD)

DA
DL

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

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$\Sigma P_{nt} = 884 \text{ kips}$

Must be matched

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

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

There's always a solution in steel

II-45

Example 4 (LRFD)

DA
DL

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

$$\Sigma P_{e2} = 0.85 \frac{\Sigma HL}{\Delta_H} = 0.85 \frac{(1.77)(15.0(12))}{0.148} = 1{,}830 \text{ kips}$$

No change from DLW load combination

There's always a solution in steel

II-46

Example 4 (LRFD)

DA
DL

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

$$B_2 = \frac{1}{1-\frac{(1.0)(884)}{1,830}} = 1.93$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads and the requirement to use the Direct Analysis Method is 1.7 rather than 1.5.

We have already included the notional load.

There's always a solution in steel

II-47

Example 4 (LRFD)

DA
DL

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$M_u = 1.0(0.0) + 1.93(26.6) = 51.3 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

$$P_u = (442) + 1.93(0.0) = 442 \text{ kips}$$

There's always a solution in steel

II-48

Example 4 (LRFD)

DA
DL

- Determine member strength

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

$K_y L = 15.0$ ft

$L_b = 15.0$ ft $\phi_b M_n = 573$ ft-kips

$$\boxed{p = 0.998\times10^{-3} \quad b_x = 1.55\times10^{-3}}$$

- Interaction Eq. H1-1a

$$\frac{442}{1{,}000}+\frac{8}{9}\left(\frac{51.3}{573}\right)=0.52<1.0 \;\therefore\; \text{ok}$$

$$\boxed{0.998\times10^{-3}(442)+1.55\times10^{-3}(51.3)=0.52<1.0 \;\therefore\; \text{ok}}$$

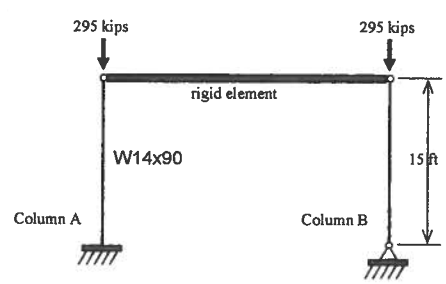
There's always a solution in steel

II-49

Example 4 (ASD)

DA
DL

- Design by Direct Analysis



$P_D = 75.0$ kips

$P_L = 220$ kips

$W = 12.5$ kips

$$\boxed{D+L}$$

There's always a solution in steel

II-50

Example 4 (ASD)

DA
DL

- Design by Direct Analysis
  - Notional load

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

  - Since there is no lateral load, the notional load must be applied.

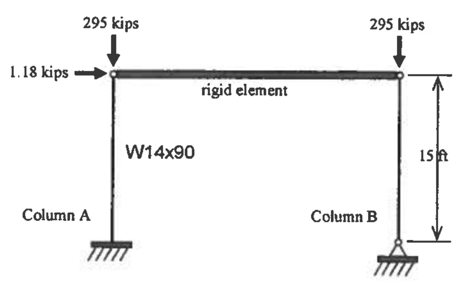
There's always a solution in steel

II-51

Example 4 (ASD)

DA
DL

- 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
$C_b = 1.67$

$$\Delta_{1st} = \frac{Pl^3}{3EI^*} = \frac{1.18(15)^3(1{,}728)}{3(0.8)(29{,}000)(999)} = 0.0989 \text{ in.}$$

There's always a solution in steel

II-52

Example 4 (ASD)

DA
DL

- Stiffness reduction $\tau_b = 1.0$ for $\dfrac{\alpha P_r}{P_y} \le 0.5$

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

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

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

There's always a solution in steel

II-53

Example 4 (ASD)

DA
DL

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

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

$$C_m = 0.6 - 0.4(0/17.7) = 0.6$$

There's always a solution in steel

II-54

Example 4 (ASD)

DA
DL

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

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

$$P_{e1} = \frac{\pi^2 (0.8)(29{,}000)(999)}{\left(1.0(15.0)(12)\right)^2} = 7{,}060 \text{ kips}$$

There's always a solution in steel

II-55

Example 4 (ASD)

DA
DL

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

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

$$B_1 = \frac{0.6}{1 - \dfrac{(1.6)(295)}{7{,}060}} = 0.64 \not> 1.0 \therefore\ B_1 = 1.0$$

There's always a solution in steel

II-56

Example 4 (ASD)

DA
DL

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

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$\Sigma P_{nt} = 590$ kips

Must be matched

$\Sigma H = 1.18$ kips

$\Delta_H = 0.0989$ in.

There's always a solution in steel

II-57

Example 4 (ASD)

DA
DL

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

$$\Sigma P_{e2} = 0.85 \frac{\Sigma HL}{\Delta_H} = 0.85 \frac{(1.18)(15.0(12))}{0.0989} = 1{,}830 \text{ kips}$$

No change from DLW
load combination

There's always a solution in steel

II-58

Example 4 (ASD)

DA
DL

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

$$B_2 = \frac{1}{1 - \frac{(1.6)(590)}{1,830}} = 2.07$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads and the requirement to use the Direct Analysis Method is 1.7 rather than 1.5.

We have already included the notional load.

There's always a solution in steel

II-59

Example 4 (ASD)

DA
DL

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$M_u = 1.0(0.0) + 2.07(17.7) = 36.6 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

$$P_u = (295) + 2.07(0.0) = 295 \text{ kips}$$

There's always a solution in steel

II-60

Example 4 (ASD)

DA
DL

- Determine member strength

$K_xL = 15.0$ ft

$K_yL = 15.0$ ft

$L_b = 15.0$ ft

$\dfrac{P_n}{\Omega_c} = 667$ kips

$\dfrac{M_n}{\Omega_b} = 382$ ft-kips

$p = 1.50\times10^{-3}$

$b_x = 2.33\times10^{-3}$

- Interaction Eq. H1-1a

$$\frac{295}{667}+\frac{8}{9}\left(\frac{36.6}{382}\right)=0.527<1.0 \ \therefore \text{ ok}$$

$$\boxed{1.50\times10^{-3}(295)+2.33\times10^{-3}(36.6)=0.528<1.0 \ \therefore \text{ ok}}$$

There's always a solution in steel

II-61

LRFD – ASD Comparison 4

DA
DL

- Differences
  - Load Combinations

LRFD	ASD
$1.2D+1.6L$	$D+L$
$P_u = 442$ kips	$P_a = 295$ kips

  - Structure Amplification

$\alpha = 1.0$	$\alpha = 1.6$
$B_2 = 1.93$	$B_2 = 2.07$

There's always a solution in steel

II-62

Example 5

- Consider a similar structure but one that has been designed so that the lateral drift under the service wind load,

  $W = 12.5$ kips, is limited to

$$\frac{L}{400} = \frac{15(12)}{400} = 0.45 \text{ in.}$$

There's always a solution in steel

II-63

Example 5

- Thus,

$$I = \frac{PL^3}{3E\Delta} = \frac{12.5(15)^3(1{,}728)}{3(29{,}000)(0.45)} = 1{,}860 \text{ in.}^4$$

- Select

$$\text{W}14\times159$$

$$I = 1{,}900 \text{ in.}^4 \quad A = 46.7 \text{ in.}^2$$

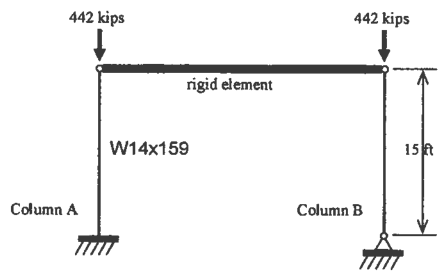
There's always a solution in steel

II-64

Example 5 (LRFD)

DA
DL
Stiff

- Design by Direct Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$\boxed{1.2D + 1.6L}$

There's always a solution in steel

II-65

Example 5 (LRFD)

DA
DL
Stiff

- Design by Direct Analysis
  - Notional load

$$Y_i = (442 + 442) = 884 \text{ kips}$$
$$N_i = 0.002(884) = 1.77 \text{ kips}$$

  - Since there is no lateral load, the notional load must be applied.

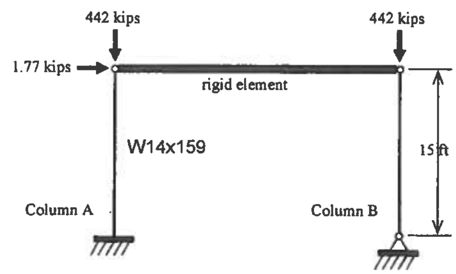
There's always a solution in steel

II-66

Example 5 (LRFD)

DA
DL
Stiff

- 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
$C_b = 1.67$

$$\Delta_{1st} = \frac{Pl^3}{3EI^*} = \frac{1.77(15)^3(1,728)}{3(0.8)(29,000)(1,900)} = 0.0781 \text{ in.}$$

Because of increased stiffness, deflection is reduced.

There's always a solution in steel

II-67

Example 5 (LRFD)

DA
DL
Stiff

- Stiffness reduction $\tau_b = 1.0$ for $\dfrac{\alpha P_r}{P_y} \le 0.5$

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

$$\frac{\alpha P_r}{P_y} = \frac{1.0(442)}{(50 \text{ ksi})(46.7 \text{ in.}^2)} = 0.19 < 0.5$$

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

There's always a solution in steel

II-68

Example 5 (LRFD)

DA
DL
Stiff

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

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

$$C_m = 0.6 - 0.4\left(0/26.6\right) = 0.6$$

There's always a solution in steel

II-69

Example 5 (LRFD)

DA
DL
Stiff

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

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

$$P_{e1} = \frac{\pi^2 \left(0.8\right)(29{,}000)(1{,}900)}{\left(1.0\left(15.0\right)(12)\right)^2} = 13{,}400 \text{ kips}$$

There's always a solution in steel

II-70

Example 5 (LRFD)

DA
DL
Stiff

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

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

$$B_1 = \frac{0.6}{1 - \frac{(1.0)(442)}{13,400}} = 0.62 \not> 1.0 \therefore\ B_1 = 1.0$$

There's always a solution in steel

II-71

Example 5 (LRFD)

DA
DL
Stiff

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

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$$\Sigma P_{nt} = 884 \text{ kips}$$

Must be matched

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

$$\Delta_H = 0.0781 \text{ in.}$$

There's always a solution in steel

II-72

Example 5 (LRFD)

DA
DL
Stiff

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

$$\Sigma P_{e2} = 0.85\frac{\Sigma HL}{\Delta_H} = 0.85\frac{(1.77)(15.0(12))}{0.0781} = 3,470 \text{ kips}$$

Note that the value has increased due to the increase in stiffness of the structure. Was 1,830 kips.

There's always a solution in steel

II-73

Example 5 (LRFD)

DA
DL
Stiff

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

$$B_2 = \frac{1}{1-\frac{(1.0)(884)}{3,470}} = 1.34$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads and the requirement to use the Direct Analysis Method is 1.7 rather than 1.5.

We have already included the notional load.

There's always a solution in steel

II-74

Example 5 (LRFD)

DA
DL
Stiff

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$M_u = 1.0(0.0) + 1.34(26.6) = 35.6 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

$$P_u = (442) + 1.34(0.0) = 442 \text{ kips}$$

There's always a solution in steel

II-75

Example 5 (LRFD)

DA
DL
Stiff

- Determine member strength

$K_x L = 15.0$ ft — $\phi_c P_n = 1{,}810$ kips — $p = 0.551\times10^{-3}$

$K_y L = 15.0$ ft

$L_b = 15.0$ ft — $\phi_b M_n = 1{,}080$ ft-kips — $b_x = 0.826\times10^{-3}$

- Interaction Eq. H1-1a

$$\frac{442}{1{,}810} + \frac{8}{9}\left(\frac{35.6}{1{,}080}\right) = 0.27 < 1.0 \;\therefore\; \text{ok}$$

$$\boxed{0.551\times10^{-3}(442) + 0.826\times10^{-3}(35.6) = 0.27 < 1.0 \;\therefore\; \text{ok}}$$

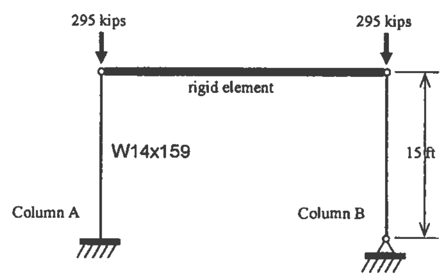
There's always a solution in steel

II-76

Example 5 (ASD)

DA
DL
Stiff

- Design by Direct Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$\boxed{D+L}$

There's always a solution in steel

II-77

Example 5 (ASD)

DA
DL
Stiff

- Design by Direct Analysis
  - Notional load

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

  - Since there is no lateral load, the notional load must be applied.

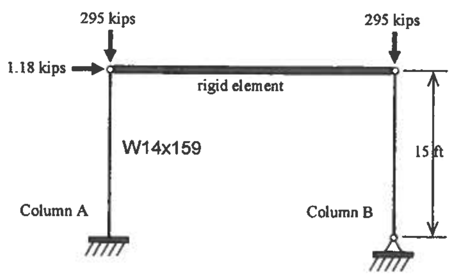
There's always a solution in steel

II-78

Example 5 (ASD)

DA
DL
Stiff

- 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

$C_b = 1.67$

$$\Delta_{1st} = \frac{Pl^3}{3EI^*} = \frac{1.18(15)^3(1,728)}{3(0.8)(29,000)(1,900)} = 0.0520 \text{ in.}$$

Because of increased stiffness, deflection is reduced.

There's always a solution in steel

II-79

Example 5 (ASD)

DA
DL
Stiff

- Stiffness reduction $\tau_b = 1.0$ for $\dfrac{\alpha P_r}{P_y} \le 0.5$

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

$$\frac{\alpha P_r}{P_y} = \frac{1.6(295)}{(50 \text{ ksi})(46.7 \text{ in.}^2)} = 0.20 < 0.5$$

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

There's always a solution in steel

II-80

Example 5 (ASD)

DA
DL
Stiff

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

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

$$C_m = 0.6 - 0.4\left(0/17.7\right) = 0.6$$

There's always a solution in steel

II-81

Example 5 (ASD)

DA
DL
Stiff

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

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

$$P_{e1} = \frac{\pi^2\left(0.8\right)(29{,}000)(1{,}900)}{\left(1.0\left(15.0\right)(12)\right)^2} = 13{,}400 \text{ kips}$$

There's always a solution in steel

II-82

Example 5 (ASD)

DA
DL
Stiff

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

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

$$B_1 = \frac{0.6}{1 - \frac{(1.6)(295)}{13,400}} = 0.62 \not> 1.0 \therefore \; B_1 = 1.0$$

There's always a solution in steel

II-83

Example 5 (ASD)

DA
DL
Stiff

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

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$\Sigma P_{nt} = 590$ kips

$\Sigma H = 1.18$ kips

$\Delta_H = 0.0520$ in.

Must be matched

There's always a solution in steel

II-84

Example 5 (ASD)

DA
DL
Stiff

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

$$\Sigma P_{e2} = 0.85\frac{\Sigma HL}{\Delta_H} = 0.85\frac{(1.18)(15.0(12))}{0.0520} = 3,470 \text{ kips}$$

Note that the value has increased due to the increase in stiffness of the structure. Was 1,830 kips.

There's always a solution in steel

II-85

Example 5 (ASD)

DA
DL
Stiff

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

$$B_2 = \frac{1}{1-\frac{(1.6)(590)}{3,470}} = 1.37$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads and the requirement to use the Direct Analysis Method is 1.7 rather than 1.5.

We have already included the notional load.

There's always a solution in steel

II-86

Example 5 (ASD)

DA
DL
Stiff

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$M_u = 1.0(0.0) + 1.37(17.7) = 24.2 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

$$P_u = (295) + 1.37(0.0) = 295 \text{ kips}$$

There's always a solution in steel

II-87

Example 5 (ASD)

DA
DL
Stiff

- Determine member strength

$K_x L = 15.0$ ft $\qquad \dfrac{P_n}{\Omega_c} = 1{,}210$ kips

$K_y L = 15.0$ ft

$L_b = 15.0$ ft $\qquad \dfrac{M_n}{\Omega_b} = 716$ ft-kips

$p = 0.829 \times 10^{-3}$

$b_x = 1.25 \times 10^{-3}$

- Interaction Eq. H1-1a

$$\frac{295}{1{,}210} + \frac{8}{9}\left(\frac{24.2}{716}\right) = 0.274 < 1.0 \ \therefore \text{ ok}$$

$$\boxed{0.829 \times 10^{-3}(295) + 1.25 \times 10^{-3}(24.2) = 0.275 < 1.0 \ \therefore \text{ ok}}$$

There's always a solution in steel

II-88

LRFD – ASD Comparison 5

DA
DL
Stiff

- Differences
  - Load Combinations

LRFD	ASD
$1.2D+1.6L$	$D+L$
$P_u = 442$ kips	$P_a = 295$ kips

  - Structure Amplification

LRFD	ASD
$\alpha = 1.0$	$\alpha = 1.6$
$B_2 = 1.32$	$B_2 = 1.37$

There's always a solution in steel

II-89

Direct Analysis

- This approach may be used in ALL cases

It must be used when $B_2 = \dfrac{\Delta_{2nd-order}}{\Delta_{1st-order}} > 1.5$

Other options when $B_2 = \dfrac{\Delta_{2nd-order}}{\Delta_{1st-order}} \leq 1.5$

C2.2a Design by Second-order analysis
C2.2b Design by First-order analysis

There's always a solution in steel

II-90

Direct Analysis

- This approach may be used in ALL cases

It must be used when $B_2 = \frac{\Delta_{2nd-order}}{\Delta_{1st-order}} > 1.7$

Other options when $B_2 = \frac{\Delta_{2nd-order}}{\Delta_{1st-order}} \leq 1.7$

If using EI^* and EA^*

C2.2a Design by Second-order analysis
C2.2b Design by First-order analysis

There's always a solution in steel

II-91

Applicable Approaches

- We can calculate B_2, even before the structure is designed, to determine which approaches may be used in the design.

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

There's always a solution in steel

II-92

Applicable Approaches

- We normally know the total gravity load for each load combination:

$$\alpha \Sigma P_{nt}$$

- We can determine a measure of lateral stiffness based on our design drift limit.

$$\Delta_H = \frac{L}{\text{drift index}}$$

There's always a solution in steel

II-93

Applicable Approaches

- Thus,

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H} = R_M \Sigma H (\text{drift index})$$

Remember that the load and the drift limit must be matched and your final design must confirm that you have at least met the drift limit. It is recommended that the wind drift limit be set for the 10-year wind.

There's always a solution in steel

II-94

Applicable Approaches

- The second-order amplification can then be determined for every gravity load combination. Or, the worst case amplification can be determined using the greatest total gravity load.

$$B_2 = \frac{1}{1 - \dfrac{\text{total gravity load}}{R_M \Sigma H (\text{drift index})}} \geq 1.0$$

There's always a solution in steel

II-95

Applicable Approaches

- For now we will assume that we are permitted to use the other approved methods.
  - Design by Second-order Analysis (Effective Length Method)
  - Design by First-order Analysis

There's always a solution in steel

II-96

Effective Length Method

- C2.2a Design by second-order elastic analysis
  - Apply notional loads, $N_i = 0.002Y_i$ in gravity only load cases
  - Perform a second-order elastic analysis
    - Use nominal geometry
    - Use nominal stiffness
  - Determine K from a sidesway buckling analysis

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

There's always a solution in steel

II-97

Effective Length Method

- C2.2a Design by second-order elastic analysis
  - How does this differ from Direct Analysis?
    - Do not use the reduced stiffness, $EI*$ and $EA*$.
    - Must determine K.
  - How does this differ from what you should have been doing all along?
    - Must consider notional loads.

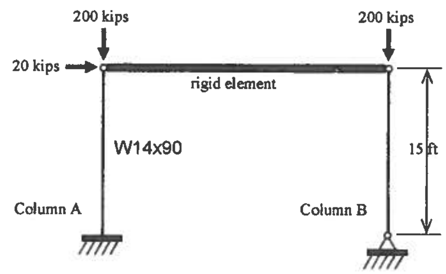
There's always a solution in steel

II-98

Example 6 (LRFD)

ELM
DLW

- Design by Second-Order Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$$\boxed{1.2D + 0.5L + 1.6W}$$

There's always a solution in steel

II-99

Example 6 (LRFD)

ELM
DLW

- Design by Second-Order Analysis
  - Notional load

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

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

  - Since there is a lateral load, there is no need to add a notional load

We are assuming that $B_2 < 1.5$.
If it was not, we could not use this method

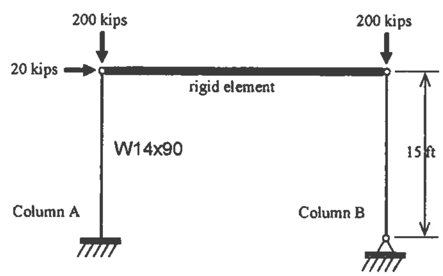
There's always a solution in steel

II-100

Example 6 (LRFD)

ELM
DLW

- First-order elastic analysis



$P_{nt} = 200$ kips

$P_{lt} = 0$ kips

$M_{ntx} = 0$ ft-kips

$M_{ltx} = 300$ ft-kips

$K_x = 2.0$

$K_y = 1.0$

$L_b = 15$ ft

$C_b = 1.67$

$$\Delta_{1st} = \frac{Pl^3}{3EI} = \frac{20(15)^3(1728)}{3(29000)(999)} = 1.34 \text{ in.}$$

There's always a solution in steel

II-101

Example 6 (LRFD)

ELM
DLW

- Design by Second-Order Analysis
  - Amplify first-order analysis, member effect

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

$$C_m = 0.6 - 0.4\left(0/300\right) = 0.6$$

There's always a solution in steel

II-102

Example 6 (LRFD)

ELM
DLW

- Design by Second-Order Analysis

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

$$P_{e1} = \frac{\pi^2 (29000)(999)}{\left(1.0(15.0)(12)\right)^2} = 8{,}830 \text{ kips}$$

There's always a solution in steel

II-103

Example 6 (LRFD)

ELM
DLW

- Design by Second-Order Analysis
  - Amplify first-order analysis; member effect

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

$$B_1 = \frac{0.6}{1 - \dfrac{(1.0)(200)}{8{,}830}} = 0.61 \ngtr 1.0 \therefore \; B_1 = 1.0$$

There's always a solution in steel

II-104

Example 6 (LRFD)

ELM
DLW

- Design by Second-Order Analysis
  - Amplify first-order analysis; structure effect

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$\Sigma P_{nt} = 400 \text{ kips}$

Must be matched

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

$\Delta_H = 1.34 \text{ in.}$

There's always a solution in steel

II-105

Example 6 (LRFD)

ELM
DLW

- Design by Second-Order Analysis
  - Amplify first-order analysis; structure effect

$$\Sigma P_{e2} = 0.85 \frac{\Sigma HL}{\Delta_H} = 0.85 \frac{(20)(15.0(12))}{1.34} = 2,280 \text{ kips}$$

There's always a solution in steel

II-106

Example 6 (LRFD)

ELM
DLW

$$B_2 = \frac{1}{1 - \frac{(1.0)(400)}{2,280}} = 1.21 = \frac{\Delta_{2nd}}{\Delta_{1st}} < 1.5$$

Thus, this method is applicable

There's always a solution in steel

II-107

Example 6 (LRFD)

ELM
DLW

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$M_u = 1.0(0.0) + 1.21(300) = 363 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

$$P_u = (200) + 1.21(0.0) = 200 \text{ kips}$$

There's always a solution in steel

II-108

Example 6 (LRFD)

ELM
DLW

- Determine the effective length, including the leaning column

$$K_x = 2.0 \text{ (for a cantilever column)}$$

- To include the leaning column

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

$$K_{nx} = 2.0\sqrt{1+\frac{200}{200}} = 2.83$$

See earlier development

There's always a solution in steel

II-109

Example 6 (LRFD)

ELM
DLW

- Determine member strength

$K_{nx}L = 2.83(15.0\text{ ft}) = 42.5\text{ ft}$ $\phi_c P_n = 721\text{ kips}$

$K_y L = 15.0\text{ ft}$

$L_b = 15.0\text{ ft}$ $\phi_b M_n = 573\text{ ft-kips}$

- Interaction Eq. H1-1a

$$\frac{200}{721}+\frac{8}{9}\left(\frac{363}{573}\right) = 0.840 < 1.0 \quad \therefore \text{ ok}$$

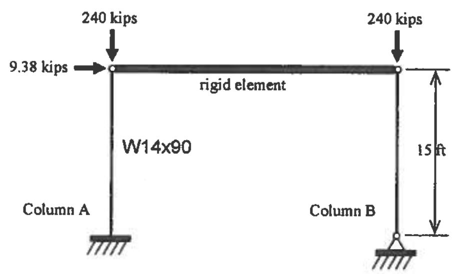
There's always a solution in steel

II-110

Example 6 (ASD)

ELM
DLW

- Design by Second-Order Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$$\boxed{D + 0.75L + 0.75W}$$

There's always a solution in steel

II-111

Example 6 (ASD)

ELM
DLW

- Design by Second-Order Analysis
  - Notional load

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

$$N_i = 0.002(480) = 0.960 \text{ kips}$$

  - Since there is a lateral load, there is no need to add a notional load

We are assuming that $B_2 < 1.5$.
If it was not, we could not use this method

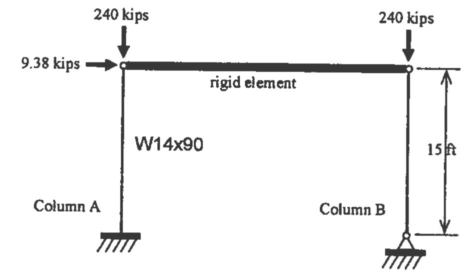
There's always a solution in steel

II-112

Example 6 (ASD)

ELM
DLW

- First-order elastic analysis



$P_{nt} = 240$ kips
$P_{lt} = 0$ kips
$M_{ntx} = 0$ ft-kips
$M_{ltx} = 141$ ft-kips
$K_x = 2.0$
$K_y = 1.0$
$L_b = 15$ ft
$C_b = 1.67$

$$\Delta_{1st} = \frac{Pl^3}{3EI} = \frac{9.38(15)^3(1728)}{3(29000)(999)} = 0.629 \text{ in.}$$

There's always a solution in steel

II-113

Example 6 (ASD)

ELM
DLW

- Design by Second-Order Analysis
  - Amplify first-order analysis, member effect

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

$$C_m = 0.6 - 0.4\left(0/141\right) = 0.6$$

There's always a solution in steel

II-114

Example 6 (ASD)

ELM
DLW

- Design by Second-Order Analysis

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

$$P_{e1} = \frac{\pi^2 (29000)(999)}{\left(1.0(15.0)(12)\right)^2} = 8,830 \text{ kips}$$

There's always a solution in steel

II-115

Example 6 (ASD)

ELM
DLW

- Design by Second-Order Analysis
  - Amplify first-order analysis; member effect

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

$$B_1 = \frac{0.6}{1 - \dfrac{(1.6)(240)}{8,830}} = 0.63 \not> 1.0 \therefore \; B_1 = 1.0$$

There's always a solution in steel

II-116

Example 6 (ASD)

ELM
DLW

- Design by Second-Order Analysis
  - Amplify first-order analysis; structure effect

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

$\Sigma P_{nt} = 480$ kips

Must be matched

$\sum H = 9.38$ kips

$\Delta_H = 0.629$ in.

There's always a solution in steel

II-117

Example 6 (ASD)

ELM
DLW

- Design by Second-Order Analysis
  - Amplify first-order analysis; structure effect

$$\Sigma P_{e2} = 0.85 \frac{\Sigma HL}{\Delta_H} = 0.85 \frac{(9.38)(15.0(12))}{0.629} = 2{,}280 \text{ kips}$$

There's always a solution in steel

II-118

Example 6 (ASD)

ELM
DLW

$$B_2 = \frac{1}{1 - \frac{(1.6)(480)}{2,280}} = 1.5 = \frac{\Delta_{2nd}}{\Delta_{1st}} < 1.5$$

Thus, this method is applicable

There's always a solution in steel

II-119

Example 6 (ASD)

ELM
DLW

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$M_a = 1.0(0.0) + 1.50(141) = 212 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

$$P_a = (240) + 1.50(0.0) = 240 \text{ kips}$$

There's always a solution in steel

II-120

Example 6 (ASD)

ELM
DLW

- Determine the effective length, including the leaning column

$$K_x = 2.0 \text{ (for a cantilever column)}$$

- To include the leaning column

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

$$K_{nx} = 2.0\sqrt{1+\frac{240}{240}} = 2.83$$

See earlier development

There's always a solution in steel

II-121

Example 6 (ASD)

ELM
DLW

- Determine member strength

$K_x^*L = 2.83(15.0\text{ ft}) = 42.5\text{ ft}$ $\qquad \frac{P_n}{\Omega_c} = 481$ kips

$K_yL = 15.0$ ft

$L_b = 15.0$ ft $\qquad \frac{M_n}{\Omega_b} = 382$ ft-kips

- Interaction Eq. H1-1a

$$\frac{240}{481}+\frac{8}{9}\left(\frac{212}{382}\right) = 0.992 < 1.0 \;\therefore\; \text{ok}$$

There's always a solution in steel

II-122

LRFD – ASD Comparison 6 ELM DLW

- Differences
  - Load Combinations

	LRFD	ASD
	$1.2D + 0.5L + 1.6W$	$D + 0.75L + 0.75W$
	$P_u = 200$ kips	$P_a = 240$ kips

  - Structure Amplification

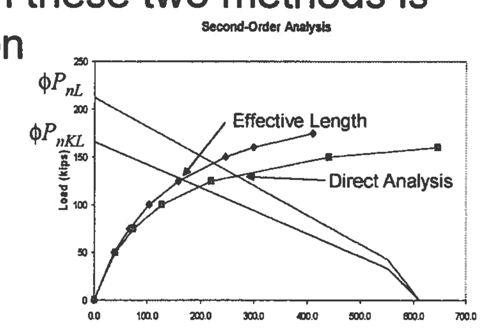
	$\alpha = 1.0$	$\alpha = 1.6$
	$B_2 = 1.21$	$B_2 = 1.5$

There's always a solution in steel

II-123

Issues with Effective Length Method

- Accuracy of our determination of effective length
- Difference between these two methods is really the calibration



There's always a solution in steel

II-124

First-Order Analysis Method

- C2.2b Design by First-order analysis
  - Limitations
    - Same as for Effective Length Method

$$B_2 \leq 1.5$$

    - In addition to Effective Length Method

$$\alpha P_r \leq 0.5 P_y$$

There's always a solution in steel

II-125

First-Order Analysis Method

- C2.2b Design by First-order analysis
  - Apply additional notional loads,

$$N_i = 2.1\left(\Delta_{1st-order}/L\right)Y_i \geq 0.0042Y_i$$

  - Perform a first-order analysis
    - Use nominal geometry
    - Use nominal stiffness
  - Apply B_1 multiplier to moment in beam-columns
  - Use K=1.0 in beam-column design

There's always a solution in steel

II-126

First-Order Analysis Method

- C2.2b Design by First-order analysis
- How does this differ from Direct Analysis?
  - Does not use the reduced stiffness, EI^* and EA^*.
  - Notional load always applied
  - Don't need to do a second-order analysis except for B_1.

There's always a solution in steel

II-127

First-Order Analysis Method

- C2.2b Design by First-order analysis
- How does this differ from Effective Length Method?
  - Notional load always applied
  - Don't need to do a second-order analysis except for B_1.

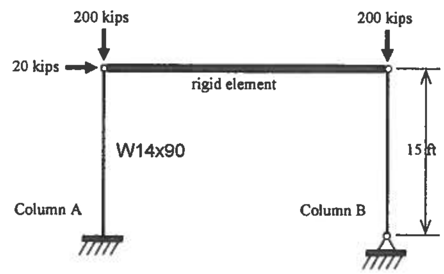
There's always a solution in steel

II-128

Example 7 (LRFD)

FOA
DLW

- Design by First-Order Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$$\boxed{1.2D + 0.5L + 1.6W}$$

There's always a solution in steel

II-129

Example 7 (LRFD)

FOA
DLW

- Design by First-Order Analysis
  - Limitations
    - From our earlier calculations $B_2 \le 1.5$
    - Load magnitude

$$\alpha P_r = 200 \text{ kips}$$

$$0.5P_y = 0.5(50.0)(26.5) = 663 \text{ kips}$$

thus,

$$\alpha P_r \le 0.5P_y$$

Design by First-Order Analysis may be used

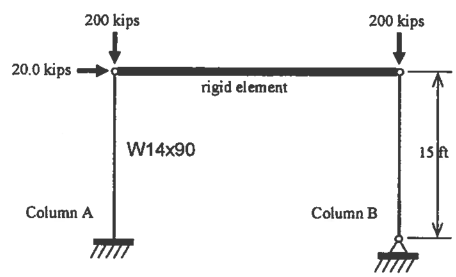
There's always a solution in steel

II-130

Example 7 (LRFD)

FOA
DLW

- First-order elastic analysis



$$P_u = 200 \text{ kips}$$
$$M_{ux} = 300 \text{ ft-kips}$$
$$K_x = 1.0$$
$$K_y = 1.0$$
$$L_b = 15 \text{ ft}$$
$$C_b = 1.67$$

$$\Delta_{1st} = \frac{Pl^3}{3EI} = \frac{20(15)^3(1728)}{3(29000)(999)} = 1.34 \text{ in.}$$

There's always a solution in steel

II-131

Example 7 (LRFD)

FOA
DLW

- Design by First-Order Analysis
  - Notional load

$$Y_i = (200 + 200) = 400 \text{ kips}$$
$$N_i = 2.1\left(\frac{\Delta}{L}\right)Y_i \geq 0.0042Y_i$$
$$= 2.1\left(\frac{1.34}{15(12)}\right)(400) \geq 0.0042(400)$$
$$= 6.25 \text{ kips} \geq 1.68 \text{ kips}$$
$$= 6.25 \text{ kips}$$

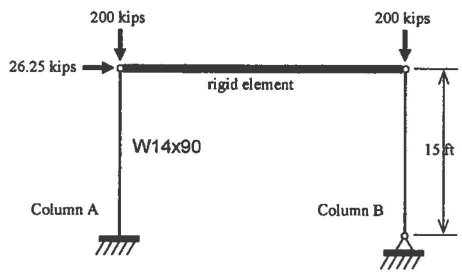
There's always a solution in steel

II-132

Example 7 (LRFD)

FOA
DLW

- First-order elastic analysis with notional load



$P_u = 200$ kips

$M_{ux} = 394$ ft-kips

Only change

$K_x = 1.0$

$K_y = 1.0$

$L_b = 15$ ft

$C_b = 1.67$

There's always a solution in steel

II-133

Example 7 (LRFD)

FOA
DLW

- Design by First-Order Analysis
  - Amplify first-order analysis, member effect

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

$$C_m = 0.6 - 0.4(0/394) = 0.6$$

There's always a solution in steel

II-134

Example 7 (LRFD)

FOA
DLW

- Design by First-Order Analysis

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

$$P_{e1} = \frac{\pi^2 (29000)(999)}{\left(1.0(15.0)(12)\right)^2} = 8,830 \text{ kips}$$

There's always a solution in steel

II-135

Example 7 (LRFD)

FOA
DLW

- Design by First-Order Analysis
  - Amplify first-order analysis; member effect

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

$$B_1 = \frac{0.6}{1 - \frac{(1.0)(200)}{8,830}} = 0.61 \not\geq 1.0 \therefore\ B_1 = 1.0$$

There's always a solution in steel

II-136

Example 7 (LRFD)

FOA
DLW

Design moment

$$M_r = B_1 M_u$$

$$M_r = 1.0(394) = 394 \text{ ft-kips}$$

Design force

$$P_r = P_u = 200 \text{ kips}$$

There's always a solution in steel

II-137

Example 7 (LRFD)

FOA
DLW

- Determine member strength

$K_x L = 15.0$ ft $\quad \phi_c P_n = 1000$ kips

$K_y L = 15.0$ ft

$L_b = 15.0$ ft $\quad \phi_b M_n = 573$ ft-kips

- Interaction Eq. H1-1a

$$\frac{200}{1000} + \frac{8}{9}\left(\frac{394}{573}\right) = 0.811 < 1.0 \;\therefore\; \text{ok}$$

There's always a solution in steel

II-138

First-Order Analysis Method

- This approach assumes that the drift amplification is at its maximum permitted value of 1.5.
- The amplification is accounted for through the use of the larger notional load.
- This has been calibrated to the Direct Analysis Method.
- When permitted it is clearly the simplest.

There's always a solution in steel

II-139

Stability Analysis and Design

Table 2–1
Summary Comparison of Methods for Stability Analysis and Design

	Direct Analysis Method	Effective Length Method	First-Order Analysis Method
Limitations on Use[1]	None	$\Delta_{2nd}/\Delta_{1st} \le 1.5$	$\Delta_{2nd}/\Delta_{1st} \le 1.5$ $\alpha P_r/P_y \le 0.5$
Analysis Type	Second-order elastic[2]		First-order elastic
Geometry of Structure	All three methods use the undeformed geometry in the analysis.		
Minimum or Additional Lateral Loads Required in the Analysis	Minimum[3]; 0.2% of the story gravity load	Minimum; 0.2% of the story gravity load	Additive; at least 0.42% of the story gravity load
Member Stiffnesses Used in the Analysis	Reduced EA and EI	Nominal EA and EI	
Design of Columns	$K = 1$ for all frames	$K = 1$ for braced frames. For moment frames, determine K from sidesway buckling analysis[4]	$K = 1$ for all frames[5]
Specification Reference for Method	Appendix 7	Section C2.2a	Section C2.2b

1 $\Delta_{2nd}/\Delta_{1st}$ is the ratio of second-order drift to first-order drift, which can be taken to be equal to B_2 calculated per Section C2.1b. $\Delta_{2nd}/\Delta_{1st}$ is determined using LRFD load combinations or a multiple of 1.6 times ASD load combinations.
2 Either a general second-order analysis method or second-order analysis by amplified first-order analysis (the "B_1-B_2 method" described in Section C2.1b) can be used.
3 This notional load is additive if $\Delta_{2nd}/\Delta_{1st} > 1.5$.
4 $K=1$ is permitted for moment frames when $\Delta_{2nd}/\Delta_{1st} \le 1.1$.
5 An additional amplification for member curvature effects is required for columns in moment frames.

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II-140

Basic Design Values

Analysis and Design

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Simplified Method (see Note 1)

Step 1. Perform first-order analysis. Use 0.2% of total story gravity load as minimum lateral load in all load combinations.
Step 2. Establish the design story drift limit and determine the lateral load required to produce it.
Step 3. Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For ASD, multiply by 1.6.
Step 4. Multiply first-order results by the tabular value. K=1, except for moment frames when the tabular value is greater than 1.1.

Design Story Drift Limit	Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)										
	0	5	10	20	30	40	50	60	80	100	120
H/100	1	1.1	1.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
H/200	1	[illegible]	1.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
H/300	[illegible]	1	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.5	[illegible]	[illegible]
H/400	1	1	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.3	1.4	[illegible]
H/500	1	1	[illegible]	[illegible]	[illegible]	1.1	1.1	[illegible]	1.2	1.3	1.4

Summary

Elastic Methods (for plastic design, see Appendix 1)	Effective Length	Forces and Moments	Limitations	Reference
First-order analysis method – second-order effects captured from effects of additional lateral load	K = 1 for all frames (see Note 2)	From analysis	$\Delta_{2nd}/\Delta_{1st} \leq 1.5$; Axial load limited	Section C2.2b
Effective length method – second-order analysis with 0.2% of total story gravity load as minimum lateral load in all load combinations (see Note 3)	K = 1, except for moment frames with $\Delta_{2nd}/\Delta_{1st} > 1.1$	From analysis (see Note 3)	$\Delta_{2nd}/\Delta_{1st} \leq 1.5$	Section C2.2a
Direct analysis method – second-order analysis with notional lateral load and reduced EI and AE (see Note 3)	K = 1 for all frames	From analysis (see Note 3)	None	Appendix 7

Notes:

1. Derived from the effective length method, using the B_1-B_2 approximation with B_1 taken equal to B_2.
2. An additional amplification for member curvature effects is required for columns in moment frames.
3. The B_1-B_2 approximation (Section C2.1b) can be used to accomplish a second-order analysis within the limitation that $B_2 \leq 1.5$. Also, B_1 and B_2 can be taken equal to the multiplier tabulated for the simplified method above.
4. $\Delta_{2nd}/\Delta_{1st}$ is the ratio of second-order drift to first-order drift, which is also represented by B_2.

$R_m = 0.85$

There's always a solution in steel

II-141

Stability Analysis and Design

	Direct Analysis	Effective Length	First-Order Analysis
Specification Reference	Appendix 7	Section C.2.2a	Section C.2.2b
Limits on Applicability?	No	Yes	Yes
Type of analysis	Second-Order	Second-Order	First-Order
Member stiffness	Reduced EI & EA	Nominal EI& EA	Nominal EI & EA
Notional lateral load	Yes, sometimes	Yes, sometimes	Yes, always
Column effective length	K=1	Calculate K	K=1

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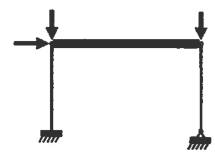
II-142

Comparison of Results

- Three approaches



Method	Example	Loading	LRFD			ASD		
			B_1	B_2	Int. Eq.	B_1	B_2	Int. Eq.
DA	3	D+L+W	1.00	1.280	0.796	1.00	1.730	0.983
DA	4	D+L	1.00	1.930	0.520	1.00	2.070	0.527
ELM	6	D+L+W	1.00	1.210	0.840	1.00	1.508	0.992
ELM		D+L		$B_2>1.5$			$B_2>1.5$	
FOA	7	D+L+W	1.00	1.280	0.811		$B_2>1.5$	
FOA		D+L		$B_2>1.5$			$B_2>1.5$	

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II-143

Simplified Approaches

- There are two simplified approaches that may be used, particularly for preliminary design. The first is based on "Design by Second-Order Analysis" and the second is based on "Design by First-Order Analysis."
- Look first at simplifying the calculation of the amplification factors.

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II-144

Second-Order Analysis

- For the amplified first-order analysis

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

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II-145

Second-Order Analysis

- Simplifying Assumption

$$M_r = B_1 M_{nt} + B_2 M_{lt} = B_2 \left(M_{nt} + M_{lt} \right) = B_2 M_u$$
$$P_r = P_{nt} + B_2 P_{lt} = B_2 \left(P_{nt} + P_{lt} \right) = B_2 P_u$$

This is applicable as long as $B_2 \geq B_1$

With this approach, there is no need to do separate "no translation" and "translation" analyses.

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II-146

Second-Order Analysis

- Simplifying Assumption
  - When should we be concerned that $B_2 < B_1$
    - Very stiff lateral system – produces low B_2
    - High axial loads – produces high B_1
    - Single curvature bending – produces high B_1

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II-147

Second-Order Analysis

- Now look closely at B_2

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1.0$$

- The measure of lateral stiffness is

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H}$$

There's always a solution in steel

II-148

Second-Order Analysis

- If we set our drift limit at the beginning of the design, and confirm that limit has not been exceeded when we are finished, then

$$\Delta_{\max} = \frac{L}{\text{drift index}}$$

and

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H} = R_M \Sigma H (\text{drift index})$$

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II-149

Second-Order Analysis

- Substituting back into B_2,

$$B_2 = \frac{1}{1 - \left(\frac{\alpha \Sigma P_{nt}}{\Sigma H}\right)\left(\frac{1}{R_M (\text{drift index})}\right)} \geq 1.0$$

Note that this is now a function of the ratio of gravity load to lateral load and the drift index.

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II-150

GRAVITY/LATERAL FOR

Basic Design Values

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Simplified Method (see Note 1)

Step 1. Perform first-order analysis. Use 0.2% of total story gravity load as minimum lateral load in all load combinations.
Step 2. Establish the design story drift limit and determine the lateral load required to produce it.
Step 3. Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For ASD, multiply by 1.6.
Step 4. Multiply first-order results by the tabular value. K=1, except for moment frames when the tabular value is greater than 1.1.

Design Story Drift Limit	Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)										
	0	5	10	20	30	40	50	60	80	100	120
H/100	1	1.1	1.1	1.3	1.5				When ratio exceeds 1.5, simplified method requires a stiffer structure		
H/200	1	1	1.1	1.1	1.2	1.3	1.4	1.5			
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	1.5		
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.5
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4

Elastic Methods (for plastic design, see Appendix 1)	Effective Length	Forces and Moments	Limitations	Reference
First-order analysis method – second-order effects captured from effects of additional lateral load	K = 1 for all frames (see Note 2)	From analysis	$\Delta_{2nd}/\Delta_{1st} \le 1.5$; Axial load limited	Section C2.2b
Effective length method – second-order analysis with 0.2% of total story gravity load as minimum lateral load in all load combinations (see Note 3)	K = 1, except for moment frames with $\Delta_{2nd}/\Delta_{1st} > 1.1$	From analysis (see Note 3)	$\Delta_{2nd}/\Delta_{1st} \le 1.5$	Section C2.2a
Direct analysis method – second-order analysis with notional lateral load and reduced EI and AE (see Note 3)	K = 1 for all frames	From analysis (see Note 3)	None	Appendix 7

Notes:
1. Derived from the effective length method, using the B_1-B_2 approximation with B_1 taken equal to B_2.
2. An additional amplification for member curvature effects is required for columns in moment frames.
3. The B_1-B_2 approximation (Section C2.1b) can be used to accomplish a second-order analysis within the limitation that $B_2 \le 1.5$. Also, B_1 and B_2 can be taken equal to the multiplier tabulated for the simplified method above.
4. $\Delta_{2nd}/\Delta_{1st}$ is the ratio of second-order drift to first-order drift, which is also represented by B_2.

$\boxed{R_m = 0.85}$

There's always a solution in steel

II-151

Basic Design Values

- Although this approach has its foundation in Design by Second-Order Analysis, that is only because of the K values shown in the table.
- The table could be used to determine the sway amplification for any set of drift limits and load ratio.

There's always a solution in steel

II-152

Simplified Method

- Step 1: Perform a first-order elastic analysis. Gravity load cases must include a minimum lateral load at each story equal to 0.002 times the story gravity load.
  - Reference: Specification C2.2a(3).
- Step 2: Establish the design story drift limit and determine the lateral load that produces that drift.
  - Reference: Specification C2.1b - Equation C2-6b

There's always a solution in steel

II-153

Simplified Method

- Step 3: Determine the ratio of the total story gravity load to the lateral determined in step 2. For ASD, α = 1.6.
  - Reference: Specification C2.2a(2)

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H} \qquad B_2 = \frac{1}{1 - \dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}} \geq 1.0$$

There's always a solution in steel

II-154

Simplified Method

- Step 4: Multiply all of the forces and moments from the first-order analysis by the value (B_2) obtained from the Table.
  - Reference: Specification C2.1b - user notes and Equation C2-3.

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II-155

Simplified Method

- Step 5: For all cases where the multiplier (B_2) is 1.1 or less, the effective length may be taken as the member length, K=1.0. Otherwise, determine K .
  - Reference: Specification C2.2a(4).
- Step 6: Ensure that the drift limit set in Step 2 is not exceeded and revise design as needed.

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II-156

Example 8 (LRFD)

S-ELM
DLW

Check the column of Example 6 using the approach from the Basic Design Value cards

1. Results of a first-order analysis

$$P_u = 200 \text{ kips}$$

$$M_u = 300 \text{ ft-kips}$$

2. Design story drift

$$W = 20 \text{ kips} \qquad \Delta_{max} = 1.34 \text{ in.} = \frac{H}{135}$$

There's always a solution in steel

II-157

Example 8 (LRFD)

S-ELM
DLW

3. Ratio of story gravity to lateral load from step 2

$$\frac{(200+200)}{20} = 20.0$$

4. Enter table with 20 and H/100

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II-158

Example 8 (LRFD)

S-ELM
DLW

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Simplified Method (see Note 1)

Step 1. Perform first-order analysis. Use 0.2% of total story gravity load as minimum lateral load in all load combinations.
Step 2. Establish the design story drift limit and determine the lateral load required to produce it.
Step 3. Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For ASD, multiply by 1.6.
Step 4. Multiply first-order results by the tabular value. K=1, except for moment frames when the tabular value is greater than 1.1.

Design Story Drift Limit	Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)										
	0	5	10	20	30	40	50	60	80	100	120
H/100	1	1.1	1.1	1.3	1.5	When ratio exceeds 1.5, simplified method requires a stiffer structure.					
H/200	1	1	1.1	1.1	1.2	1.3	1.4	1.5			
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	1.5		
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.5
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4

Elastic Methods (for plastic design, see Appendix 1)	Effective Length	Forces and Moments	Limitations	Reference
First-order analysis method – second-order effects captured from effects of additional lateral load	K = 1 for all frames (see Note 2)	From analysis	$\Delta_{2nd}/\Delta_{1st} \leq 1.5$; Axial load limited	Section C2.2b
Effective length method – second-order analysis with 0.2% of total story gravity load as minimum lateral load in all load combinations (see Note 3)	K = 1, except for moment frames with $\Delta_{2nd}/\Delta_{1st} > 1.1$	From analysis (see Note 3)	$\Delta_{2nd}/\Delta_{1st} \leq 1.5$	Section C2.2a
Direct analysis method – second-order analysis with notional lateral load and reduced EI and AE (see Note 3)	K = 1 for all frames	From analysis (see Note 3)	None	Appendix 7

Notes:

1. Derived from the effective length method, using the B_1-B_2 approximation with B_1 taken equal to B_2.
2. An additional amplification for member curvature effects is required for columns in moment frames.
3. The B_1-B_2 approximation (Section C2.1b) can be used to accomplish a second-order analysis within the limitation that $B_2 \leq 1.5$. Also, B_1 and B_2 can be taken equal to the multiplier tabulated for the simplified method above.
4. $\Delta_{2nd}/\Delta_{1st}$ is the ratio of second-order drift to first-order drift, which is also represented by B_2.

$R_m = 0.85$

There's always a solution in steel

II-159

Example 8 (LRFD)

S-ELM
DLW

- Amplified load and moment

$$P_u = 1.3(200) = 260 \text{ kips}$$

$$M_u = 1.3(300) = 390 \text{ ft-kips}$$

- Still need to determine K as before.

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II-160

Example 8 (ASD)

S-ELM
DLW

Check the column of Example 6 using the approach from the Basic Design Value cards

1. Results of a first-order analysis

$$P_a = 240 \text{ kips}$$
$$M_a = 141 \text{ ft-kips}$$

2. Design story drift

$$W = 9.38 \text{ kips} \qquad \Delta_{\max} = 0.629 \text{ in.} = \frac{H}{286}$$

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II-161

Example 8 (ASD)

S-ELM
DLW

3. Ratio of story gravity to lateral load from step 2

$$\frac{1.6(240+240)}{9.38} = 81.9$$

4. Enter table with 80.0 and H/200

There's always a solution in steel

II-162

Example 8 (ASD)

S-ELM
DLW

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Simplified Method (see Note 1)

Step 1. Perform first-order analysis. Use 0.2% of total story gravity load as minimum lateral load in all load combinations.
Step 2. Establish the design story drift limit and determine the lateral load required to produce it.
Step 3. Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For ASD, multiply by 1.6.
Step 4. Multiply first-order results by the tabular value. K=1, except for moment frames when the tabular value is greater than 1.1

Design Story Drift Limit	Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)										
	0	5	10	20	30	40	50	60	80	100	120
H/100	1	1.1	1.1	1.3	1.5				When ratio exceeds 1.5, simplified method requires a stiffer structure.		
H/200	1	1	1.1	1.1	1.2	1.3	1.4	1.5			
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	1.5		
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.5
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4

Elastic Methods (for plastic design, see Appendix 1)	Effective Length	Forces and Moments	Limitations	Reference
First-order analysis method – second-order effects captured from effects of additional lateral load	K = 1 for all frames (see Note 2)	From analysis	$\Delta_{2nd}/\Delta_{1st} \leq 1.5$; Axial load limited	Section C2.2b
Effective length method – second-order analysis with 0.2% of total story gravity load as minimum lateral load in all load combinations (see Note 3)	K = 1, except for moment frames with $\Delta_{2nd}/\Delta_{1st} > 1.1$	From analysis (see Note 3)	$\Delta_{2nd}/\Delta_{1st} \leq 1.5$	Section C2.2a
Direct analysis method – second-order analysis with notional lateral load and reduced EI and AE (see Note 3)	K = 1 for all frames	From analysis (see Note 3)	None	Appendix 7

Notes:

1. Derived from the effective length method, using the B_1-B_2 approximation with B_1 taken equal to B_2.
2. An additional amplification for member curvature effects is required for columns in moment frames.
3. The B_1-B_2 approximation (Section C2.1b) can be used to accomplish a second-order analysis within the limitation that $B_2 \leq 1.5$. Also, B_1 and B_2 can be taken equal to the multiplier tabulated for the simplified method above.
4. $\Delta_{2nd}/\Delta_{1st}$ is the ratio of second-order drift to first-order drift, which is also represented by B_2.

$\boxed{R_m = 0.85}$

There's always a solution in steel

II-163

Example 8 (ASD)

S-ELM
DLW

- Conclusion
  - This simplified approach may not be used for this ASD problem.
  - This is Example 6 (ASD) and the amplification in that example was right at the limit of 1.5.
  - The conservative nature of the simplification pushes us over the boundary.

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II-164

Basic Design Values

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Simplified Method (see Note 1)

Step 1. Perform first-order analysis. Use 0.2% of total story gravity load as minimum lateral load in all load combinations.
Step 2. Establish the design story drift limit and determine the lateral load required to produce it.
Step 3. Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For ASD, multiply by 1.6.
Step 4. Multiply first-order results by the tabular value. K=1, except for moment frames when the tabular value is greater than 1.1.

Design Story Drift Limit	Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)										
	0	5	10	20	30	40	50	60	80	100	120
H/100	1	1.1	1.1	1.3	1.5	When ratio exceeds 1.5, simplified method requires a stiffer structure.					
H/200	1	1	1.1	1.1	1.2	1.3	1.4	1.5			
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	1.5		
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.5
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4

Note that as the gravity load increases, for the same lateral stiffness, the second order amplification increases.

This is always the case, not just for the simplified approach presented here.

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II-165

Simplified Approaches

- The second simplified approach is based on Design by First-Order Analysis

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II-166

First-Order Analysis Method

- C2.2b Design by First-order analysis
  - Consider the notional load requirement

$$N_i = 2.1(\Delta_{1st-order}/L)Y_i \geq 0.0042Y_i$$

which came from the commentary equation

$$N_i = \left(\frac{B_2}{1-0.2B_2}\right)\frac{\Delta}{L}Y_i \geq \left(\frac{B_2}{1-0.2B_2}\right)(0.002Y_i) \quad \text{(C-A-7-3-1)}$$

with

$B_2 = 1.5$ and a drift ratio of 0.002

There's always a solution in steel

II-167

Modified First-Order Method

- B_2 as a function of drift index and gravity to lateral load ratio (similar to Basic Design Value Cards)

Amplified First-Order Analysis Multiplier, B2											
Drift Index	Gravity to Lateral Load Ratio										
	0	5	10	20	30	40	50	60	80	100	120
100	1.00	1.06	1.13	1.31	1.55	1.89	2.43	3.40	17.00		
150	1.00	1.04	1.09	1.19	1.31	1.46	1.65	1.89	2.66	4.64	17.00
200	1.00	1.03	1.06	1.13	1.21	1.31	1.42	1.55	1.89	2.43	3.40
250	1.00	1.02	1.05	1.10	1.16	1.23	1.31	1.39	1.60	1.89	2.30
300	1.00	1.02	1.04	1.09	1.13	1.19	1.24	1.31	1.46	1.65	1.89
350	1.00	1.02	1.03	1.07	1.11	1.16	1.20	1.25	1.37	1.51	1.68
400	1.00	1.01	1.03	1.06	1.10	1.13	1.17	1.21	1.31	1.42	1.55
450	1.00	1.01	1.03	1.06	1.09	1.12	1.15	1.19	1.26	1.35	1.46
500	1.00	1.01	1.02	1.05	1.08	1.10	1.13	1.16	1.23	1.31	1.39

$R_m = 0.85$

$$B_2 = \frac{1}{1-\left(\frac{\alpha\Sigma P_{nt}}{\Sigma H}\right)\left(\frac{1}{R_M\,(\text{drift index})}\right)} \geq 1.0$$

There's always a solution in steel

II-168

Modified First-Order Method

- Using the same format, the notional load multiplier, γ, becomes

Notional Load Multiplier											
Drift Index	Gravity to Lateral Load Ratio										
	0	5	10	20	30	40	50	60	80	100	120
100	0.0125	0.0135	0.0147	0.0177	0.0224	0.0304	0.0472	0.1063	-0.0708		
150	0.0083	0.0088	0.0092	0.0104	0.0118	0.0137	0.0163	0.0202	0.0386	0.4250	-0.0472
200	0.0063	0.0065	0.0067	0.0073	0.0080	0.0089	0.0099	0.0112	0.0152	0.0236	0.0631
250	0.0050	0.0052	0.0053	0.0057	0.0061	0.0065	0.0071	0.0077	0.0094	0.0121	0.0170
300	0.0042	0.0043	0.0044	0.0046	0.0049	0.0052	0.0055	0.0059	0.0069	0.0082	0.0101
350	0.0036	0.0036	0.0037	0.0039	0.0041	0.0043	0.0045	0.0048	0.0054	0.0062	0.0072
400	0.0031	0.0032	0.0032	0.0034	0.0035	0.0037	0.0038	0.0040	0.0044	0.0049	0.0056
450	0.0028	0.0028	0.0029	0.0030	0.0031	0.0032	0.0033	0.0035	0.0038	0.0041	0.0046
500	0.0025	0.0025	0.0026	0.0027	0.0027	0.0028	0.0029	0.0030	0.0033	0.0035	0.0039

There's always a solution in steel

$$N_i = \left(\frac{B_2}{1-0.2B_2} \right) \frac{\Delta}{L} Y_i = \gamma Y_i$$

II-169

Modified First-Order Method

- Application of the modified method
  1. Determine the drift index corresponding to the largest story drift that will be permitted for all load combinations.
  2. Determine the largest ratio of gravity load to the lateral load that determined the drift limit. For ASD multiply by 1.6
  3. Enter the table and select the notional load multiplier, γ.

There's always a solution in steel

II-170

Modified First-Order Method

- Application of the modified method
  4. Apply the notional load as an additional load in all load combinations.
  5. Carry out a First-Order Analysis
  6. For each beam-column amplify moment with B_1
  7. Design all beam-columns with $K = 1.0$.
  8. Design all components, including connections, for the determined moments and forces.

There's always a solution in steel

II-171

Modified First-Order Method

- Example of this approach
  - Consider the frame of Example 5, the stiffened column.
  - Consider D+L+W

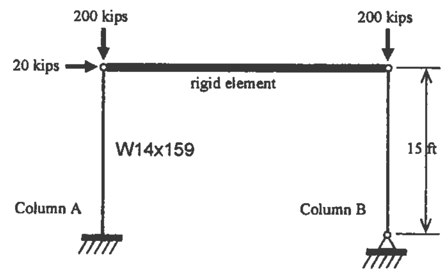
There's always a solution in steel

II-172

Example 9 (LRFD)

FOA
Chart
DLW

- Design by First-Order Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$$\boxed{1.2D + 0.5L + 1.6W}$$

There's always a solution in steel

II-173

Example 9 (LRFD)

FOA
Chart
DLW

- Design by First-Order Analysis
  - Limitations
    - From the chart we will show that $B_2 \leq 1.5$
    - Load magnitude

$\alpha P_r = 200$ kips

$0.5P_y = 0.5(50.0)(46.7) = 1170$ kips

thus,

$\alpha P_r = 200 \leq 0.5P_y = 1170$

Design by First-Order Analysis may be used

There's always a solution in steel

II-174

Example 9 (LRFD)

FOA
Chart
DLW

- Lateral displacement under service load

$$\Delta = \frac{12.5(15)^3(1,728)}{3(29,000)(1900)} = 0.441 \text{ in.}$$

- Drift ratio

$$\frac{\Delta}{L} = \frac{0.441}{15(12)} = 0.00245 \rightarrow \frac{H}{408}$$

There's always a solution in steel

II-175

Example 9 (LRFD)

FOA
Chart
DLW

- Determine the load ratio

$$\text{ratio} = \frac{Y_i}{\text{lateral}} = \frac{(200+200)}{12.5} = 32$$

- Enter table with drift index and load ratio

There's always a solution in steel

II-176

Example 9 (LRFD)

FOA
Chart
DLW

- The notional load multiplier, γ, becomes

Notional Load Multiplier											
Drift Index	Gravity to Lateral Load Ratio										
	0	5	10	20	30	40	50	60	80	100	120
100	0.0125	0.0135	0.0147	0.0177	0.0224	0.0304	0.0472	0.1083	-0.0708		
150	0.0083	0.0088	0.0092	0.0104	0.0118	0.0137	0.0163	0.0202	0.0386	0.4250	-0.0472
200	0.0063	0.0065	0.0067	0.0073	0.0080	0.0089	0.0099	0.0112	0.0152	0.0236	0.0531
250	0.0050	0.0052	0.0053	0.0057	0.0061	0.0065	0.0071	0.0077	0.0094	0.0121	0.0170
300	0.0042	0.0043	0.0044	0.0046	0.0049	0.0052	0.0055	0.0059	0.0069	0.0082	0.0101
350	0.0036	0.0036	0.0037	0.0039	0.0041	0.0043	0.0045	0.0048	0.0054	0.0062	0.0072
400	0.0031	0.0032	0.0032	0.0034	0.0035	0.0037	0.0038	0.0040	0.0044	0.0049	0.0056
450	0.0028	0.0028	0.0029	0.0030	0.0031	0.0032	0.0033	0.0035	0.0038	0.0041	0.0046
500	0.0025	0.0025	0.0026	0.0027	0.0027	0.0028	0.0029	0.0030	0.0033	0.0035	0.0039

$$N_i = \left(\frac{B_2}{1-0.2B_2}\right)\frac{\Delta}{L}Y_i = \gamma Y_i$$

Interpolate for

$$\gamma = 0.00354$$

There's always a solution in steel

II-177

Example 9 (LRFD)

FOA
Chart
DLW

- Design by First-Order Analysis
  - Notional load

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

$$N_i = 0.00354Y_i = 0.00354(400) = 1.42 \text{ kips}$$

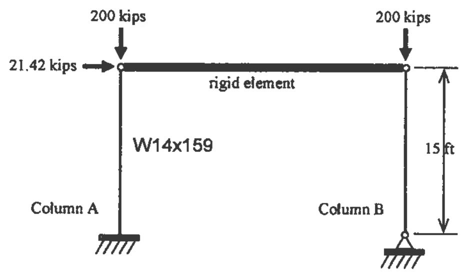
There's always a solution in steel

II-178

Example 9 (LRFD)

FOA
Chart
DLW

- First-order elastic analysis



$P_u = 200$ kips

$M_{ux} = 321$ ft-kips

$K_x = 1.0$

$K_y = 1.0$

$L_b = 15$ ft

$C_b = 1.67$

There's always a solution in steel

II-179

Example 9 (LRFD)

FOA
Chart
DLW

- Design by First-Order Analysis
  - Amplify first-order analysis, member effect

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

$$C_m = 0.6 - 0.4\left(0/321\right) = 0.6$$

There's always a solution in steel

II-180

Example 9 (LRFD)

FOA
Chart
DLW

- Design by First-Order Analysis

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

$$P_{e1} = \frac{\pi^2 (29000)(1900)}{(1.0(15.0)(12))^2} = 16{,}800 \text{ kips}$$

There's always a solution in steel

II-181

Example 9 (LRFD)

FOA
Chart
DLW

- Design by First-Order Analysis
  - Amplify first-order analysis; member effect

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

$$B_1 = \frac{0.6}{1 - \dfrac{(1.0)(200)}{16{,}800}} = 0.61 \ngeq 1.0 \therefore \; B_1 = 1.0$$

There's always a solution in steel

II-182

Example 9 (LRFD)

FOA
Chart
DLW

Design moment

$$M_r = B_1 M_u$$

$$M_r = 1.0(321) = 321 \text{ ft-kips}$$

Design force

$$P_r = P_u = 200 \text{ kips}$$

There's always a solution in steel

II-183

Example 9 (LRFD)

FOA
Chart
DLW

- Determine member strength

$K_x L = 15.0$ ft	$\phi_c P_n = 1{,}810$ kips	$p = 0.551\times10^{-3}$
$K_y L = 15.0$ ft		
$L_b = 15.0$ ft	$\phi_b M_n = 1{,}080$ ft-kips	$b_x = 0.826\times10^{-3}$

- Interaction Eq. H1-1b

$$\frac{200}{2(1{,}810)} + \left(\frac{321}{1{,}080}\right) = 0.352 < 1.0 \quad \therefore \text{ ok}$$

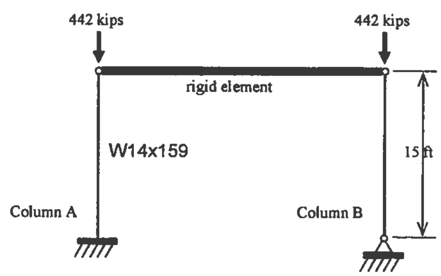
There's always a solution in steel

II-184

Example 10 (LRFD)

FOA Chart DL

- Consider the gravity only load case



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 12.5$ kips

$$\boxed{1.2D + 1.6L}$$

There's always a solution in steel

II-185

Example 10 (LRFD)

FOA Chart DL

- For this load case, the column load has increased. Thus,

$$\alpha P_r = 442 \text{ kips} < 0.5P_y = 1170 \text{ kips}$$

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

- The load ratio is then

$$\frac{884}{12.5} = 71$$

There's always a solution in steel

II-186

Example 10 (LRFD)

FOA
Chart
DL

- The notional load multiplier, γ, becomes

Notional Load Multiplier

Drift Index	Gravity to Lateral Load Ratio 0	5	10	20	30	40	50	60	80	100	120
100	0.0125	0.0135	0.0147	0.0177	0.0224	0.0304	0.0472	0.1063	-0.0708		
150	0.0083	0.0088	0.0092	0.0104	0.0118	0.0137	0.0163	0.0202	0.0388	0.4250	-0.0472
200	0.0063	0.0065	0.0067	0.0073	0.0080	0.0089	0.0099	0.0112	0.0152	0.0236	0.0531
250	0.0050	0.0052	0.0053	0.0057	0.0061	0.0065	0.0071	0.0077	0.0094	0.0121	0.0170
300	0.0042	0.0043	0.0044	0.0046	0.0049	0.0052	0.0055	0.0059	0.0069	0.0082	0.0101
350	0.0036	0.0036	0.0037	0.0039	0.0041	0.0043	0.0045	0.0048	0.0054	0.0062	0.0072
400	0.0031	0.0032	0.0032	0.0034	0.0035	0.0037	0.0038	0.0040	0.0044	0.0049	0.0058
450	0.0028	0.0028	0.0029	0.0030	0.0031	0.0032	0.0033	0.0035	0.0038	0.0041	0.0046
500	0.0025	0.0025	0.0026	0.0027	0.0027	0.0026	0.0029	0.0030	0.0033	0.0035	0.0039

$$N_i = \left(\frac{B_2}{1-0.2B_2}\right)\frac{\Delta}{L}Y_i = \gamma Y_i$$

Interpolate for

$$\gamma = 0.00422$$

There's always a solution in steel

II-187

Example 10 (LRFD)

FOA
Chart
DL

- Design by First-Order Analysis
  - Notional load

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

$$N_i = 0.00422Y_i = 0.00422(884) = 3.73 \text{ kips}$$

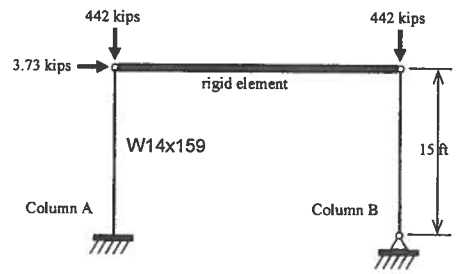
There's always a solution in steel

II-188

Example 10 (LRFD)

FOA
Chart
DL

- First-order elastic analysis



$P_u = 442$ kips

$M_{ux} = 56.0$ ft-kips

$K_x = 1.0$

$K_y = 1.0$

$L_b = 15$ ft

$C_b = 1.67$

There's always a solution in steel

II-189

Example 10 (LRFD)

FOA
Chart
DL

- Design by First-Order Analysis
  - Amplify first-order analysis, member effect

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

$$C_m = 0.6 - 0.4\left(0/56.0\right) = 0.6$$

There's always a solution in steel

II-190

Example 10 (LRFD)

FOA
Chart
DL

- Design by First-Order Analysis

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

$$P_{e1} = \frac{\pi^2 (29000)(1900)}{\left(1.0(15.0)(12)\right)^2} = 16{,}800 \text{ kips}$$

There's always a solution in steel

II-191

Example 10 (LRFD)

FOA
Chart
DL

- Design by First-Order Analysis
  - Amplify first-order analysis; member effect

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

$$B_1 = \frac{0.6}{1 - \frac{(1.0)(442)}{16{,}800}} = 0.62 \ngtr 1.0 \therefore\ B_1 = 1.0$$

There's always a solution in steel

II-192

Example 10 (LRFD)

FOA Chart DL

Design moment

$$M_r = B_1 M_u$$

$$M_r = 1.0(56.0) = 56.0 \text{ ft-kips}$$

Design force

$$P_r = P_u = 442 \text{ kips}$$

There's always a solution in steel

II-193

Example 10 (LRFD)

FOA Chart DL

- Determine member strength

$K_x L = 15.0$ ft

$K_y L = 15.0$ ft

$L_b = 15.0$ ft

$\phi_c P_n = 1{,}810$ kips

$\phi_b M_n = 1{,}080$ ft-kips

$\boxed{p = 0.551 \times 10^{-3}}$

$\boxed{b_x = 0.826 \times 10^{-3}}$

- Interaction Eq. H1-1a

$$\frac{442}{1{,}810} + \left(\frac{8}{9}\right)\left(\frac{56.0}{1{,}080}\right) = 0.290 < 1.0 \ \therefore \text{ ok}$$

There's always a solution in steel

II-194

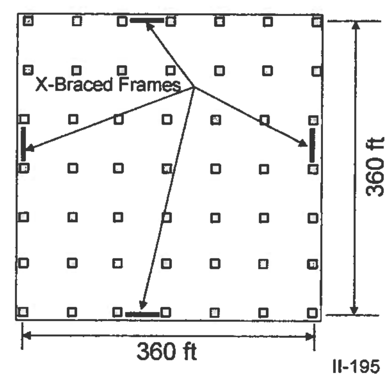
Building Example 1 Warehouse

ELM
ASD

Design the 360 ft by 360 ft warehouse structure shown. Design by Second-Order Analysis (Effective Length Method).

Eave height = 40 ft
Nominal Loads:
Dead load = 20 psf
- Roofing = 5 psf
- Deck = 2 psf
- Structure = 4 psf
- Mechanical = 5 psf
- Collateral = 4 psf

Roof snow load = 30 psf
Precast Walls = 100 psf
Wind pressure = 20 psf
Mean Wind Uplift = 10 psf



There's always a solution in steel

II-195

Building Example 1 Warehouse

ELM
ASD

- Design using ASD Load Combinations:
  - D + S
  - D + W
  - D + 0.75(S + W)
- Structural system
  - Flexible roof diaphragm
  - Open web steel joists and Joist Girders
  - One bay X-bracing in each exterior frame
- Walls
  - Tilt-up non-load bearing 8 in. precast concrete

There's always a solution in steel

II-196

Building Example 1 Warehouse

ELM
ASD

- Roof deck design:
  - Use Type 22 wide rib deck spanning 6 ft.
  - Minimum of 2 – side lap screw per 6 ft span with other fastening to be selected to satisfy diaphragm requirements
- Roof Structure:
  - Joists at 6 ft on center
  - Joist Girders at 60 ft

There's always a solution in steel

II-197

Building Example 1 Warehouse

ELM
ASD

- Gravity only Columns (40 ft long, pinned ends):
  - Tributary area to interior columns = $60 \text{x} 60 = 3600$ ft^2
  - Column load = $(20 + 30)(3600)/1000 = 180$ kips
  - Select an HSS section for economy
  - From the AISC Manual with $K = 1.0$ and $L = 40$ ft, select an HSS $12 \text{X} 12 \text{X} 3/8$ (Wt. = 58 plf), P_n/Ω_c = 220 kips

There's always a solution in steel

II-198

Building Example 1
Warehouse

ELM
ASD

- Serviceability criteria
  - For non-load bearing concrete walls, use a drift criteria of $H/100 = (40)(12)/100 = 4.8$ in. for a 10 year wind. This is approximately 6.8 in. for a 50 year wind based on ASCE 7.
  - To determine if serviceability criteria is satisfied, use a first-order analysis.

There's always a solution in steel

II-199

Building Example 1
Warehouse

ELM
ASD

- Serviceability criteria
  - Determine the in-plane deflection of the flexible diaphragm
    - Beam shear

$$V = w\left(\frac{H}{2}\right)\left(\frac{\text{Width}}{2}\right) = 20\left(\frac{40}{2}\right)\left(\frac{360}{2}\right) = 72{,}000 \text{ lbs.}$$

    - Shear per foot along edge of diaphragm at wall

$$v = \frac{V}{L} = \frac{72{,}000}{360} = 200 \text{ lbs/ft}$$

There's always a solution in steel

II-200

Building Example 1
Warehouse

ELM
ASD

- Serviceability criteria
  - Determine the in-plane deflection of the flexible diaphragm
  - From the 3rd Edition of the SDI "Diaphragm Design Manual":
    - The allowable shear for Type 22, wide rib deck, with a 36/4 weld pattern and 2 s.l.s. = 255 lbs./ft and G' = 13,500 lbs./in.
    - Deck deflection from wind (neglecting flexural deflection):

$$\Delta_{deck} = \frac{wL^2}{8DG'} = \frac{20(40/2)(360)^2}{8(360)(13,500)} = 1.33 \text{ in.}$$

There's always a solution in steel

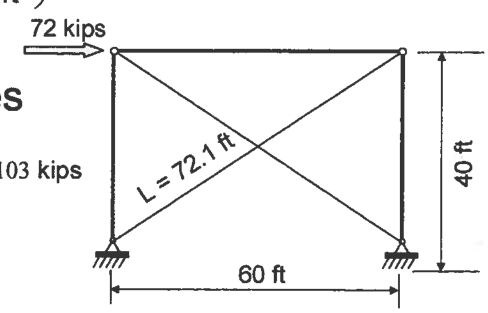
II-201

Building Example 1
Warehouse

ELM
ASD

- Determine tension only brace force

$$\text{Brace Force} = 72 \text{ kips}\left(\frac{72.1 \text{ ft}}{60 \text{ ft}}\right) = 86.5 \text{ kips}$$



- Select double angles

Use A36 steel
2L's 4x4x5/16, A= 4.8 in.^{2}, $\frac{P_n}{\Omega} = 103$ kips

Assume yielding controls

There's always a solution in steel

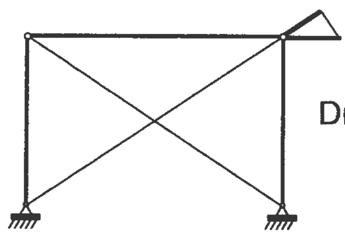
II-202

Building Example 1
Warehouse

ELM
ASD

- Determine braced frame drift
  - Elongation of brace

$$\Delta L = \frac{PL}{AE} = \frac{86.5\left(72.1(12)\right)}{4.8(29{,}000)} = 0.538 \text{ in.}$$



$$\text{Drift} = \frac{72.1 \text{ ft}}{60.0 \text{ ft}}(0.538 \text{ in.}) = 0.646 \text{ in.}$$

There's always a solution in steel

II-203

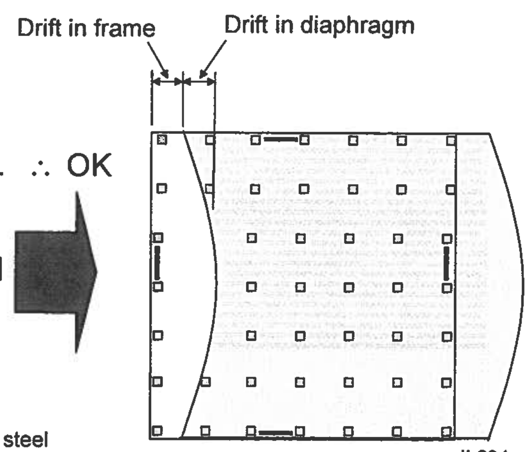
Building Example 1
Warehouse

ELM
ASD

- Check total drift

$$\text{Total drift} = 0.65 + 1.33$$
$$= 1.98 \text{ in.} < 6.8 \text{ in.} \quad \therefore \text{ OK}$$

Wind



There's always a solution in steel

II-204

Building Example 1
Warehouse

ELM
ASD

- Calculate Column loads in braced frame
  - Dead load
    $$P_a = (20)(60)(30)/1000 = 36 \text{ kips}$$
  - Snow Load
    $$P_a = (30)(60)(30)/1000 = 54 \text{ kips}$$
  - Wind uplift
    $$P_a = (-10)(60)(30)/1000 = -18 \text{ kips}$$

There's always a solution in steel

II-205

Building Example 1
Warehouse

ELM
ASD

- Design the columns at the braced bays.
  - For (D + S):
    $$P_a = 36 + 54 = 90 \text{ kips}$$
  - For (D + W):
    $$P_a = 36 - 18 + (72)(40/60) = 66.0 \text{ kips}$$
  - For (D +0.75(S + W)):
    $$P_a = 36 + 0.75(54 - 18) + 0.75(72)(40/60) = 99.0 \text{ kips}$$

There's always a solution in steel

II-206

Building Example 1
Warehouse

ELM
ASD

- Select column

$K = 1.0$

$H = 40$ ft

$P_a = 99$ kips

With the Effective Length Method, you need to determine K.
Since this is a braced frame, K=1.0

- Use

HSS $10\times10\times5/16$, $A = 11.1$ in.^{2}, $P_n/\Omega = 112$ kips

There's always a solution in steel

II-207

Building Example 1
Warehouse

ELM
ASD

- To this point, all calculations have been carried out with a first-order analysis
- Now consider the amplified first-order analysis results.
- Since there are no moments, there is no need to calculate B_1.

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

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

There's always a solution in steel

II-208

Building Example 1
Warehouse

ELM
ASD

- Sway amplification

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} \geq 1.0$$

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

$\Sigma P_{nt} =$ Total gravity load supported by story

$\Sigma P_{e2} = R_m \Sigma HL / \Delta_H$

$R_m = 1.0$ (Braced Frame)

$\Sigma H =$ lateral force causing Δ_H

There's always a solution in steel

II-209

Building Example 1
Warehouse

ELM
ASD

- Calculate the gravity load for each load case.

Load Case	ΣP_{nt}, kips
D	$(20)(360)(360)/1000 = 2590$ kips
Precast Walls	$(100)(40)(360)(4)/1000 = 5760$ kips
S	$(30)(360)(360)/1000 = 3890$ kips
W	$(-10)(360)(360)/1000 = -1300$ kips

There's always a solution in steel

II-210

Building Example 1
Warehouse

ELM
ASD

- Determine horizontal force

$$\Sigma H = 2(72) = 144 \text{ kips}$$

- Determine the drift

Drift = average diaphragm deflection + braced frame drift

$$\Delta_H = 0.67(1.33) + 0.650 = 1.54 \text{ in.}$$

Average for parabolic displacement of diaphragm

There's always a solution in steel

II-211

Building Example 1
Warehouse

ELM
ASD

- Determine frame stiffness

$$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta_H}$$

$$= 1.0 \frac{144(40)(12)}{1.54} = 44{,}900 \text{ kips}$$

There's always a solution in steel

II-212

Building Example 1 Warehouse

ELM
ASD

- Including the precast walls
  - Assume the walls parallel to the wind resist overturning directly by their own stiffness
  - Assume that the weight of the windward and leeward walls is applied at their mid-height
  - Walls move through only ½ of the drift
  - To account for these, use only ¼ of the wall weight.

There's always a solution in steel

II-213

Building Example 1 Warehouse

ELM
ASD

- Calculation of B_2

Load Combination	$\alpha\Sigma P_{nt} = 1.6\Sigma P_{nt}$	$B_2 = \frac{1}{1-\frac{\alpha\Sigma P_{nt}}{44,900}}$
D+Precast+S	$1.6(2590+1440+3890)=12,700$	1.39<1.5
D+Precast+W	$1.6(2590+1440+(-1300))=4,370$	1.11<1.5
D+Precast +0.75(S+W)	$1.6(2590+1440+0.75(3890-1300))=9,560$	1.27<1.5

There's always a solution in steel

II-214

Building Example 1 Warehouse

ELM
ASD

- Calculation of B_2
  - Since all B_2 values are less than 1.5, the effective length method may be used.

There's always a solution in steel

II-215

Building Example 1 Warehouse

ELM
ASD

- Determine the notional loads

Load Combination	$N_i = 0.002Y_i$	Wind load
D+Precast+S	$N = 0.002\left(\frac{12,700}{1.6}\right) = 15.9$ kips	>0 kips
D+Precast+W	$N = 0.002\left(\frac{4,370}{1.6}\right) = 5.46$ kips	<144 kips
D+Precast +0.75(S+W)	$N = 0.002\left(\frac{9,560}{1.6}\right) = 12.0$ kips	<0.75(144) =108 kips

There's always a solution in steel

II-216

Building Example 1
Warehouse

ELM
ASD

- Since the wind load is greater than the notional loads, there is no need to add notional loads to the combinations that include wind.
- The notional load must be added to the D+Precast+S combination.

There's always a solution in steel

II-217

Building Example 1
Warehouse

ELM
ASD

- Notional Loads
  - Force in Brace

$$\frac{15.9/2}{72.0}(86.5) = 9.55 \text{ kips}$$

  - Force in Column

$$15.9/2\left(\frac{40}{60}\right) = 5.30 \text{ kips}$$

There's always a solution in steel

II-218

Building Example 1
Warehouse

ELM
ASD

- Diaphragm

(D+Precast+W)

$$\text{Daiphragm Shear} = B_2(200)$$
$$= 1.11(200) = 222 \text{ plf } \leq 255 \text{ plf}$$

Thus, the diaphragm shear strength is adequate including second-order effects.

There's always a solution in steel

II-219

Building Example 1
Warehouse

ELM
ASD

- Brace

(D+Precast+S+N)

$$B_2(\text{Brace}) = 1.39(9.55) = 13.3 \text{ kips}$$

(D+Precast+W)

$$B_2(\text{Brace}) = 1.11(86.5) = 96.0 \text{ kips} ☆$$

(D+Precast+0.75(W+S))

$$B_2(\text{Brace}) = 1.27(64.9) = 82.4 \text{ kips}$$

There's always a solution in steel

II-220

Building Example 1
Warehouse

ELM
ASD

- Column

(D+Precast+S+N)

$$B_2(\text{Column}) = 90.0 + 1.39(5.30) = 97.4 \text{ kips}$$

(D+Precast+W)

$$B_2(\text{Column}) = 18.0 + 1.11(48.0) = 71.3 \text{ kips}$$

(D+Precast+0.75(W+S))

$$B_2(\text{Column}) = 63.0 + 1.27(36) = 109 \text{ kips} \text{ ☆}$$

There's always a solution in steel

II-221

Building Example 1
Warehouse

ELM
ASD

- Brace 2L's 4x4x5/16

$$\frac{P_n}{\Omega} = 103 \text{ kips} > 96.0 \text{ kips}$$

- Column HSS 10x10x5/16

$$\frac{P_n}{\Omega} = 112 \text{ kips} > 109 \text{ kips}$$

There's always a solution in steel

II-222

Building Example 1
Warehouse

ELM
ASD

- Conclusions
  - ELM is appropriate since no need to calculate K , by definition.
  - Different load combinations control design of different elements.
  - Each load combination has its own B_2.
  - Even though it is a braced frame, second-order amplification is significant.

There's always a solution in steel

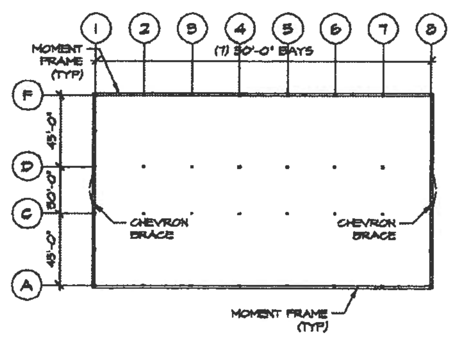
II-223

Building Example 2
4-Story Commercial

DA
ASD

- From the Companion CD to the 13th edition Manual

Carry out the analysis and design of the members of the moment frame and braced frame for the given 4-story commercial building using the Direct Analysis method.



There's always a solution in steel

II-224

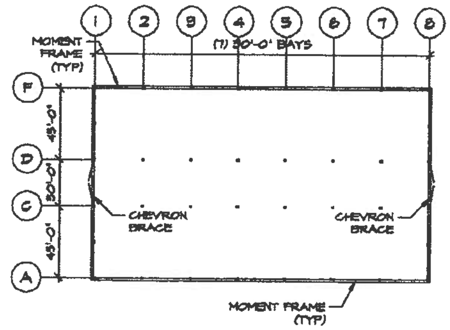
Building Example 2
4-Story Commercial

DA
ASD

- From the Companion CD to the 13th edition Manual

Roof Load
- Snow + Rain 25 psf
- Dead 20 psf

Floor
- Live 80 psf
- Dead 75 psf
- Cladding varies

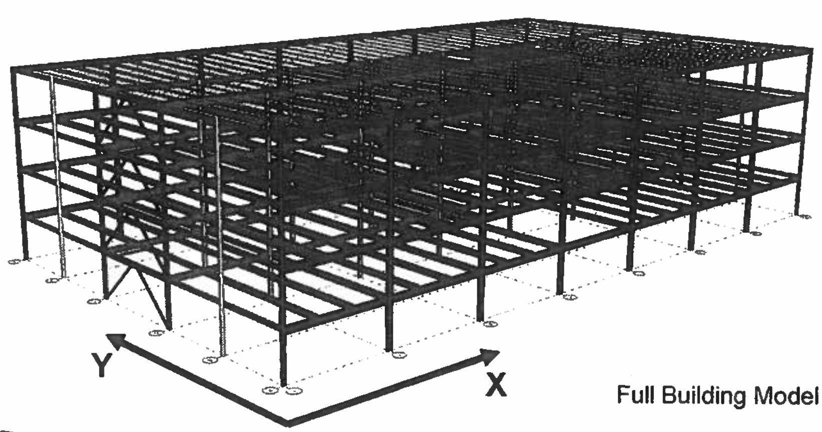


There's always a solution in steel

II-225

Building Example 2
4-Story Commercial

DA
ASD



Full Building Model

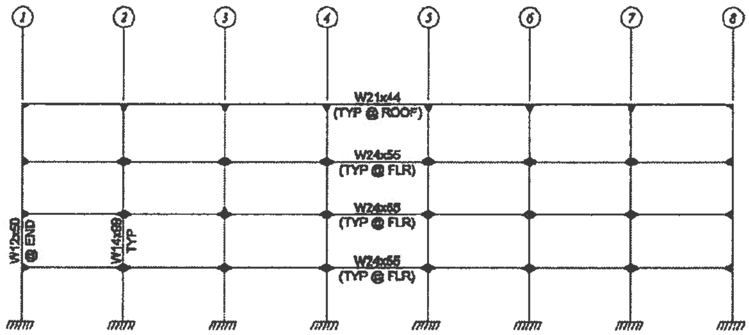
There's always a solution in steel

II-226

Building Example 2
4-Story Commercial

DA
ASD

- Moment Frame (resists forces in X direction)



FRAME LAYOUT (GRID A & F)

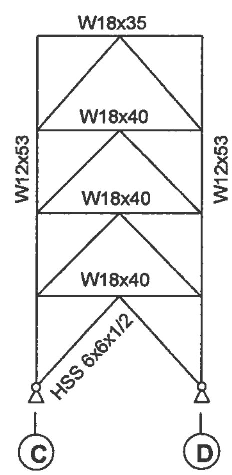
There's always a solution in steel

II-227

Building Example 2
4-Story Commercial

DA
ASD

- Braced Frame
  (resists forces in Y direction)

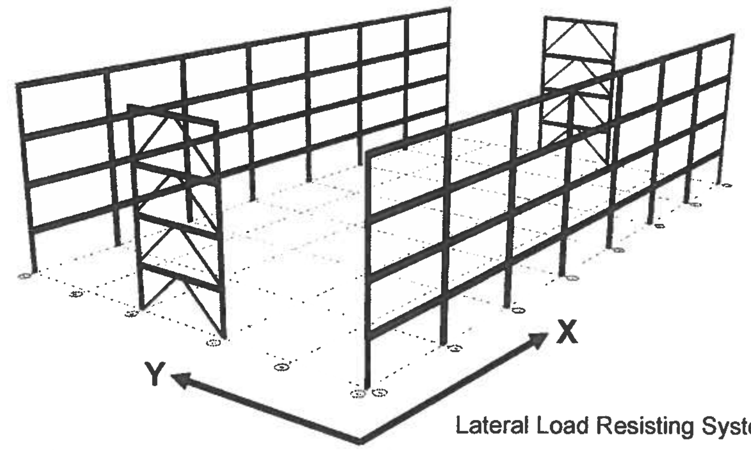


There's always a solution in steel

II-228

Building Example 2
4-Story Commercial

DA
ASD



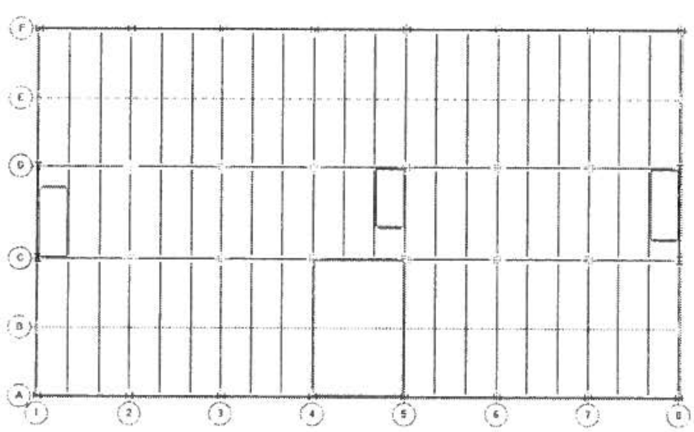
Lateral Load Resisting System

There's always a solution in steel

II-229

Building Example 2
4-Story Commercial

DA
ASD



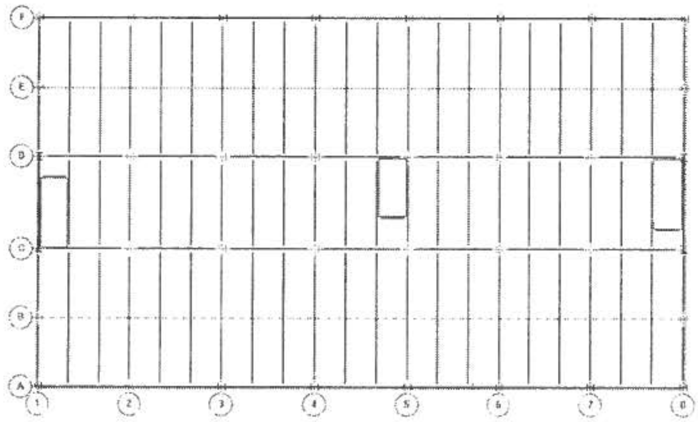
There's always a solution in steel

2nd Floor Framing

II-230

Building Example 2
4-Story Commercial

DA
ASD



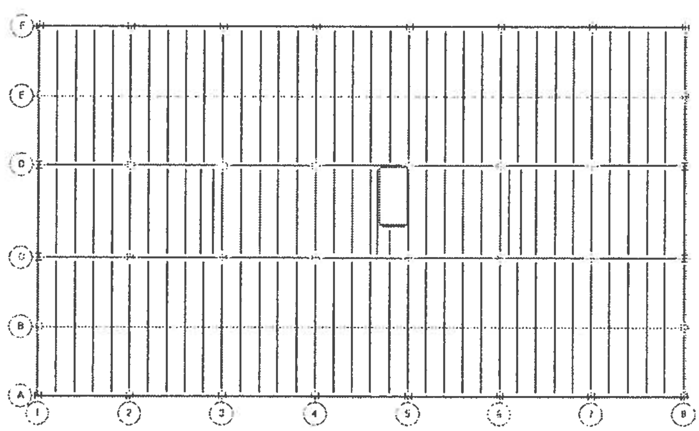
There's always a solution in steel

3rd and 4th Floor Framing

II-231

Building Example 2
4-Story Commercial

DA
ASD



There's always a solution in steel

Roof Framing

II-232

Building Example 2
4-Story Commercial

DA
ASD

- Load Cases
  - Dead (1)
  - Live (1)
  - Roof Live (Snow+Rain) (1)
  - Wind (12)
  - Seismic (4)

There's always a solution in steel

II-233

Building Example 2
4-Story Commercial

DA
ASD

- ASD Load Combinations

1. D
2. D+L
3. D+S
4. D+0.75(S+L)
5. D+W
6. D+0.7E
7. D+0.75W+0.75L+0.75S
8. D+0.525E+0.75L+0.75S
9. 0.6D+W
10. 0.6D+0.7E

There's always a solution in steel

II-234

Building Example 2
4-Story Commercial

DA
ASD

- Load Combinations for Analysis
  - 160 total load combinations
  - 12 of these as a result of the notional loads

D+N (4)	D+0.75(L+W) (24)
D+L+N (4)	D+0.75(S+W) (24)
D+S+N (4)	0.6D+W (24)
D+0.75(L+S)+N (4)	D+0.7E (8)
D+W (24)	D+0.75L+.525E (8)
D+0.75(L+S+W) (24)	0.6D+0.7E (8)

There's always a solution in steel

II-235

Building Example 2
4-Story Commercial

DA
ASD

- Could use a Planar Model (2D)
  - Requires some thought on modeling.
  - Can use forced symmetry.
  - May reduce number of load combinations.
  - Use of notional loads easily accommodated.
  - Be sure to account for "leaning columns."

There's always a solution in steel

II-236

Building Example 2
4-Story Commercial

DA
ASD

- Determine Notional Loads
  - Notional loads are determined for each gravity load case.
  - Notional loads are distributed according to the gravity load distribution.

$$N_i = 0.002Y_i$$

There's always a solution in steel

II-237

Building Example 2
4-Story Commercial

DA
ASD

Gravity load at each level, Y_i			
Level	**D (kips)**	**S (kips)**	**L (kips)**
Roof	785	638	
4	2244		817
3	2244		817
2	2127		766
Total	7400	638	2400

There's always a solution in steel

II-238

Building Example 2
4-Story Commercial

DA
ASD

Notional load at each level, N_i			
Level	**D (kips)**	**S (kips)**	**L (kips)**
Roof	1.57	1.28	
4	4.49		1.63
3	4.49		1.63
2	4.25		1.54
Total	14.8	1.28	4.80

$$N_i = 0.002Y_i$$

There's always a solution in steel

II-239

Building Example 2
4-Story Commercial

DA
ASD

- Determine amplification factors for amplified first-order analysis.
- Sway amplification is determined for each story, each direction, and each load combination.
- Member amplification will need to be determined for each member individually.

There's always a solution in steel

II-240

Building Example 2
4-Story Commercial

DA
ASD

Moment Frame　　　Braced Frame

Wind Load				
	H_X (kips)	ΣH_X	H_Y (kips)	ΣH_Y
Roof	19.68	19.68	33.94	33.94
4	36.84	56.52	63.77	97.71
3	33.48	**90.00**	58.38	156.09
2	31.43	121.43	55.24	211.33
Total	121.43		211.33	

Note the relative magnitude of these wind loads and the notional loads.

There's always a solution in steel

II-241

Building Example 2
4-Story Commercial

DA
ASD

Moment Frame　　　Braced Frame

Wind Drift				
	Total X (in.)	Δ_H X (in.)	Total Y (in.)	Δ_H Y (in.)
Roof	0.7325	0.0895	0.4846	0.0893
4	0.6430	0.1690	0.3953	0.1370
3	0.4740	**0.2516**	0.2583	0.1313
2	0.2224	0.2224	0.1270	0.1270

Drift based on reduced stiffness, $0.8\tau_b EI$ and $0.8EA$ and $\tau_b = 1.0$

There's always a solution in steel

II-242

Building Example 2
4-Story Commercial

DA
ASD

	Moment Frame	Braced Frame
	R_m =0.85	R_m=1.0
Roof	32,500	66,100
4	46,100	116,000
3	**49,300**	193,000
2	75,200	270,000

$$\Sigma P_{e2} = R_m \frac{\Sigma HL}{\Delta_H} = 0.85\left(\frac{90(13.5)(12)}{0.2516}\right) = 49,300$$

There's always a solution in steel

II-243

Building Example 2
4-Story Commercial

DA
ASD

Dead + Live

	(D+L)	1.6(D+L)	Moment Frame B_2 X	Braced Frame B_2 Y
Roof	785	1256	1.04	1.02
4	3846	6154	1.15	1.06
3	6907	11051	**1.29**	1.06
2	9800	15680	1.26	1.06

$$B_2 = \frac{1}{1-\frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} = \frac{1}{1-\frac{11,051}{49,300}} = 1.29$$

There's always a solution in steel

II-244

Building Example 2
4-Story Commercial

DA
ASD

Dead + 0.75(Live+Snow)

	D+0.75(L+S)	1.6(D+0.75(L+S))	B_2 X (Moment Frame)	B_2 Y (Braced Frame)
Roof	1264	2022	1.07	1.03
4	4120	6592	1.17	1.06
3	6977	11163	**1.29**	1.06
2	9679	15486	1.26	1.06

$$B_2 = \frac{1}{1-\frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} = \frac{1}{1-\frac{11,163}{49,300}} = 1.29$$

There's always a solution in steel

II-245

Building Example 2
4-Story Commercial

DA
ASD

- Check treatment of notional loads
- Amplification will be greatest for the largest gravity loads.
  - Since drift was based on reduced stiffness
  - In all cases

$$B_2 \leq 1.7$$

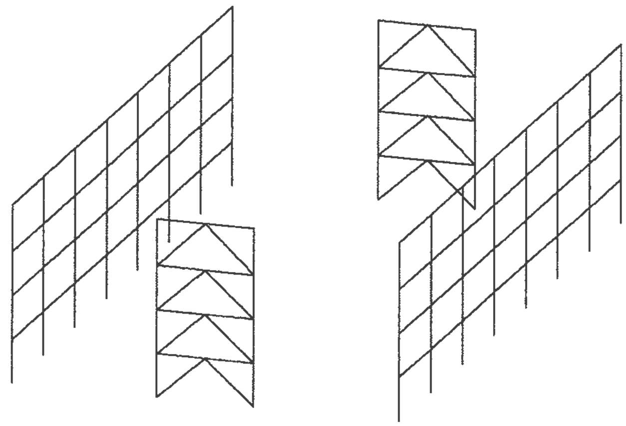
  - Thus, notional loads may be treated as minimum.

There's always a solution in steel

II-246

Building Example 2
4-Story Commercial

DA
ASD

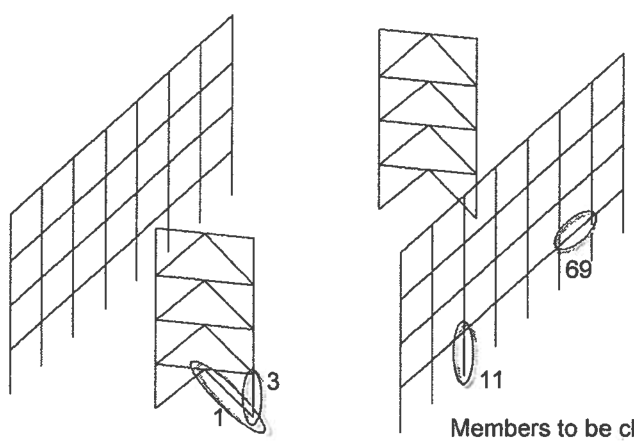


There's always a solution in steel

II-247

Building Example 2
4-Story Commercial

DA
ASD



Members to be checked

There's always a solution in steel

II-248

Building Example 2
4-Story Commercial

DA
ASD

- Member check

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

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

Assume

M_{nt}, P_{nt} come from gravity load analysis

M_{lt}, P_{lt} come from a lateral load analysis

There's always a solution in steel

II-249

Building Example 2
4-Story Commercial

DA
ASD

- Column in Braced Frame, #3, W12x53
  - Controlling load combination

$$D + 0.75L - 0.525E4$$

  - Analysis results by load case

$$D = 168 \text{ kips}$$

$$L = 52.7 \text{ kips}$$

$$E4 = -146 \text{ kips}$$

There's always a solution in steel

II-250

Building Example 2
4-Story Commercial

DA
ASD

- Column in Braced Frame, #3, W12x53
  - Sway amplification

$$B_2 = 1.07$$

  - Member amplification
    - No amplification since no moments
  - Member force

$$P_a = P_D + 0.75P_L - B_2(0.525P_{E4})$$

$$P_a = 168 + 0.75(52.7) - 1.07(0.525)(-146) = 290 \text{ kips}$$

There's always a solution in steel

II-251

Building Example 2
4-Story Commercial

DA
ASD

- Column in Braced Frame, #3, W12x53
  - Determine member strength

$$K = 1$$

$$L = 13.5 \text{ ft}$$

$$\frac{P_n}{\Omega} = 431 \text{ kips} > 290 \text{ kips}$$

  - W12x53 is adequate

There's always a solution in steel

II-252

Building Example 2
4-Story Commercial

DA
ASD

- Brace in Braced Frame, #1, HSS 6x6x1/2
  - Controlling load combination

$$D-0.7E4$$

  - Analysis results by load case

$$D=13.0 \text{ kips}$$

$$E4=-121 \text{ kips}$$

There's always a solution in steel

II-253

Building Example 2
4-Story Commercial

DA
ASD

- Brace in Braced Frame, #1, HSS 6x6x1/2
  - Sway amplification

$$B_2=1.05$$

  - Member amplification
    - No amplification since no moments
  - Member force

$$P_a=P_D-0.7P_{E4}$$

$$P_a=13.0-1.05(0.7)(-121)=102 \text{ kips}$$

There's always a solution in steel

II-254

Building Example 2
4-Story Commercial

DA
ASD

- Brace in Braced Frame, #1, HSS 6x6x1/2
  - Determine member strength

$$K = 1$$

$$L = 20.18 \text{ ft}$$

$$\frac{P_n}{\Omega} = 121 \text{ kips} > 102 \text{ kips}$$

  - HSS 6x6x1/2 is adequate

There's always a solution in steel

II-255

Building Example 2
4-Story Commercial

DA
ASD

- Column in Moment Frame, #11, W14x99
  - Controlling load combination

$$D - 0.7E2$$

  - Analysis results by load case

D	$P = 226$ kips	$M_T = 3.75$ ft-kips	$M_B = -1.86$ ft-kips
$E2$	$P = 0$ kips	$M_T = -105$ ft-kips	$M_B = 216$ ft-kips

There's always a solution in steel

II-256

Building Example 2
4-Story Commercial

DA
ASD

- Column in Moment Frame, #11, W14x99
  - Sway amplification

$$B_2 = 1.19$$

  - Member amplification

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

$$C_m = 0.6 - 0.4\left(\frac{1.86}{3.75}\right) = 0.40$$

$$\alpha P_r = 1.6(226 + 0) = 362 \text{ kips}$$

$$P_{e1} = \frac{\pi^2(29{,}000)(1{,}110)}{(13.5(12))^2} = 12{,}100 \text{ kips}$$

There's always a solution in steel

II-257

Building Example 2
4-Story Commercial

DA
ASD

- Column in Moment Frame, #11, W14x99
  - Member amplification

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} = \frac{0.4}{1 - \frac{362}{12{,}100}} = 1.03(0.4) = 0.41 < 1.0$$

  - Member force

$$P_a = P_D - B_2\left(0.7P_{E2}\right)$$

$$P_a = 226 - 1.19(0.7)(0) = 226 \text{ kips}$$

There's always a solution in steel

II-258

Building Example 2
4-Story Commercial

DA
ASD

- Column in Moment Frame, #11, W14x99
  - Member moment

$$M_a = B_1 M_D - B_2 \left(0.7 M_{E2}\right)$$

$$M_a = 1.0(-1.86) - 1.19(0.7)(216) = -182 \text{ ft-kips}$$

  - Determine member strength

$$K = 1.0,\ L = 13.5 \text{ ft},\ \frac{P_n}{\Omega} = 759 \text{ kips}$$

Building Example 2
4-Story Commercial

DA
ASD

- Column in Moment Frame, #11, W14x99
  - Determine member strength

$$L_p = 13.5 \text{ ft},\ \frac{M_n}{\Omega} = 430 \text{ ft-kips}$$

  - Interaction

$$\frac{226}{759} + \frac{8}{9}\left(\frac{182}{430}\right) = 0.30 + 0.38 = 0.68 < 1.0$$

  - W14x99 is adequate

Building Example 2
4-Story Commercial

DA
ASD

- Beam in Moment Frame, #69, W24x55
  - Controlling load combination

$$D + 0.75L + 0.525E2$$

  - Analysis results by load case

D	$M_T = -136$ ft-kips	$M_B = -139$ ft-kips
L	$M_T = -82.7$ ft-kips	$M_B = -83.8$ ft-kips
$E2$	$M_T = 133$ ft-kips	$M_B = -133$ ft-kips

There's always a solution in steel

II-261

Building Example 2
4-Story Commercial

DA
ASD

- Beam in Moment Frame, #69, W24x55
  - Sway amplification

$$B_2 = 1.26$$

  - Member amplification
    - No amplification since no axial load
  - Member moment

$$M_a = B_1\left(M_D + 0.75L\right) + +B_2\left(0.525M_{E2}\right)$$

$$M_a = 1.0\left(-139 + 0.75(-83.8)\right) + 1.26\left(0.525(-133)\right) = -290 \text{ ft-kips}$$

There's always a solution in steel

II-262

Building Example 2
4-Story Commercial

DA
ASD

- Beam in Moment Frame, #69, W24x55
  - Member strength

$$\frac{M_n}{\Omega} = 334 \text{ ft-kips} < 290 \text{ ft-kips}$$

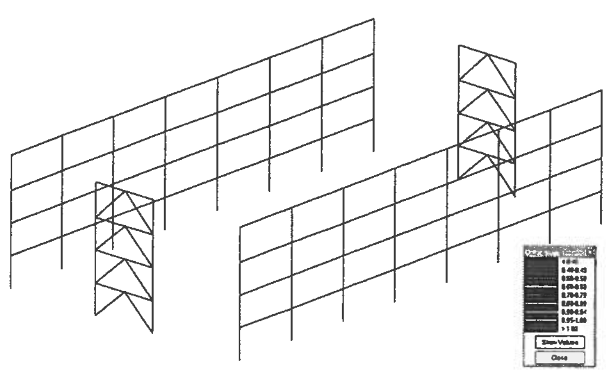
  - W24x55 is adequate

There's always a solution in steel

II-263

Building Example 2
4-Story Commercial

DA
ASD

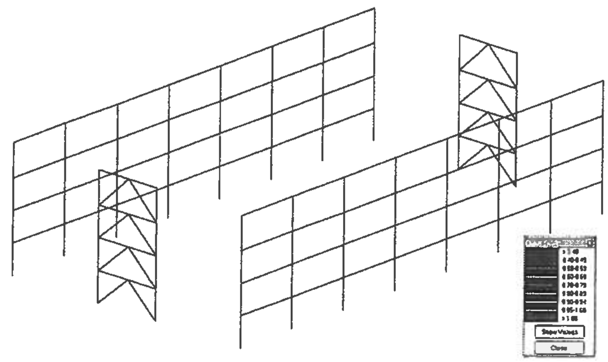


There's always a solution in steel

II-264

Building Example 2
4-Story Commercial

DA
ASD



There's always a solution in steel

II-265

Building Example 2
4-Story Commercial

DA
ASD

- Conclusions
  - Superposition is appropriate since amplified first-order analysis is used for 2nd order effects.
  - There is no need to analyze for 1.6 times the ASD load combinations since only a first-order analysis is carried out.
  - The sway amplification, B_2, may be different for each load combination, at each level, and in each direction.

There's always a solution in steel

II-266

Building Example 2
4-Story Commercial

DA
ASD

- Conclusions
  - Using Direct Analysis means that $K = 1.0$ in the moment frame (X).
  - Direct Analysis provides no advantage in the braced frame direction (Y) since $K = 1.0$ is already acceptable.
  - Y direction amplification is a bit higher than if EI and EA had been used. (from 1.05 to 1.06)

There's always a solution in steel

II-267

The Future is Now

- The 2005 AISC Specification (ANSI/AISC 360-05) is adopted by IBC 2006.
- ANSI/AISC 360-05 provides 3 approaches for determining the required strength of members.
- The Direct Analysis method provides an approach for eliminating the need to determine K.

There's always a solution in steel

II-268

The Future is Now

- The Effective Length method provides an approach quite close to the approach generally used in today's practice.
- The Design by First-Order Analysis method gives an opportunity to account for second-order effects through notional loads.

There's always a solution in steel

II-269

The Future is Now

- Conclusion
  - Each method can be used for an efficient analysis and can lead to an economical design. You decide which best fits your needs.

Design Steel Your Way!

There's always a solution in steel

II-270

Thank You

American Institute of Steel Construction
One East Wacker Drive
Chicago, IL 60601

There's always a solution in steel

II-271

Please give us your feedback!

www.aisc.org/cesurvey

American Institute of Steel Construction
One East Wacker Drive
Chicago, IL 60601

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II-272

Building Example 2

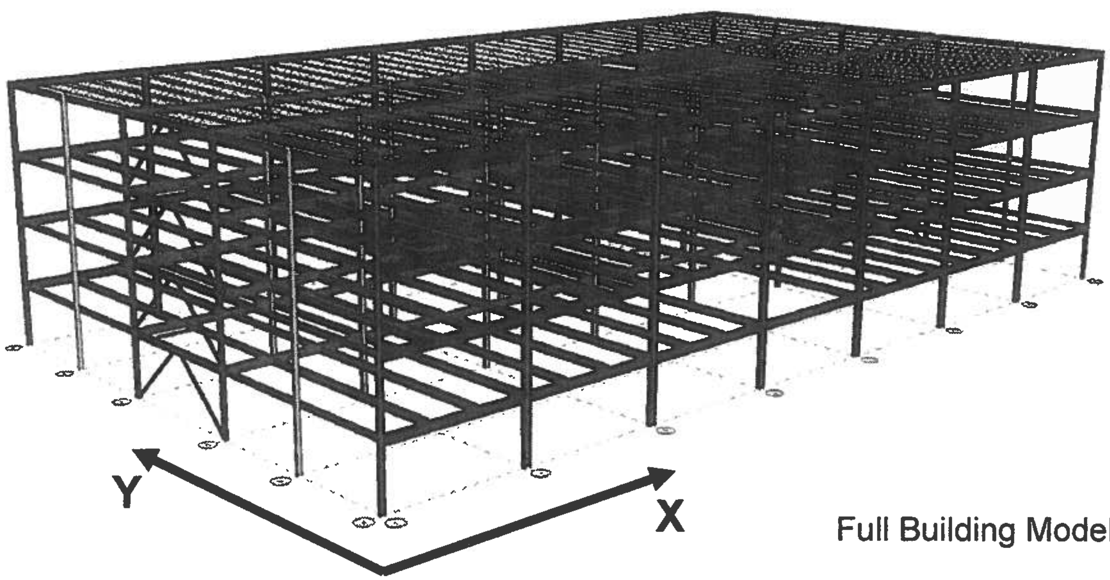
4-story Commercial

Selected slides and computer output

The RAM Structural System website is:
http://www.bentley.com/en-US/Products/RAM+Structural+System/

For questions on RAM contact:
Allen Adams, S.E.
Chief Structural Engineer
RAM International / Bentley
Allen.Adams@bentley.com

Building Example 2
4-Story Commercial



Full Building Model

There's always a solution in steel

AISC 360 Direct Analysis Validation Report

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC 02/11/09 10:09:39

DESIGN CODE
AISC 360-05 ASD

SECOND-ORDER ANALYSIS
P-Delta analysis was not performed

B1 Factors:
B1 factors were calculated and applied to gravity load case moments.

B2 Factors:
B2 factors were applied to lateral load case moments and axial loads.
RMX = 0.850
RMY = 1.000

Maximum B2 = 1.303 on Level 3rd at an angle of 0.00 degrees.
Load Combination : 1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W10.

NOTIONAL LOADS
Fraction of gravity loads used for Notional Loads:
Global X-axis : 0.0020
Global Y-axis : 0.0020

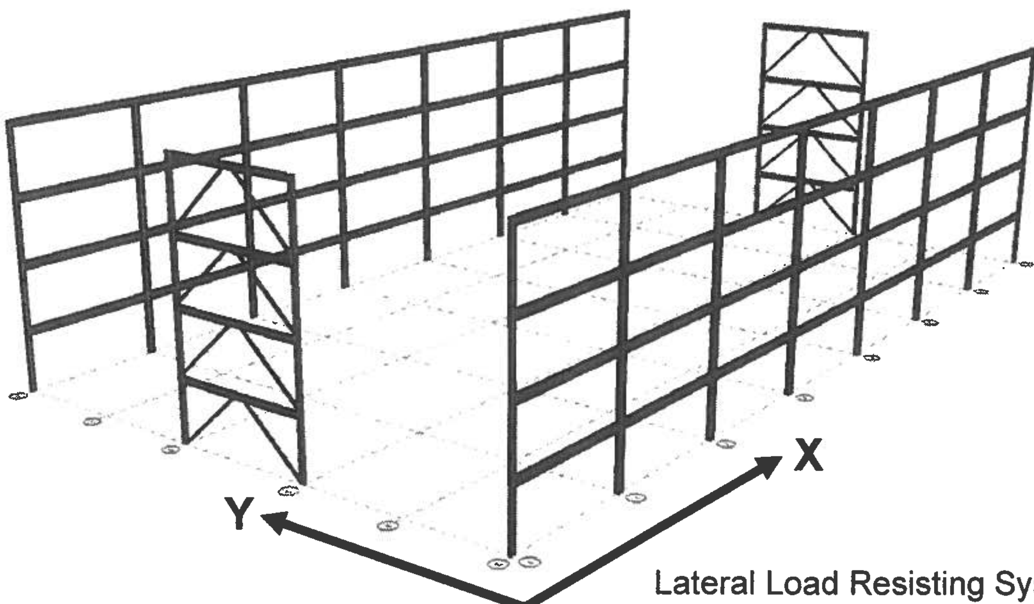
Generated Load Combinations:
Number of Selected Load Combinations = 160
Notional Loads were included with gravity combinations only.

REDUCED STIFFNESS
Flexural Stiffness:
The flexural stiffnesses were reduced.

Number of members with required $\tau b < 1.0 = 4$
Smallest required $\tau b = 0.972$
Column #: 4 on Level: 2nd
Load Combination: 1.000 D + 0.750 Lp + 0.525 E4
τb used in Analysis : 1.000
Analysis Invalid. Either specify larger notional loads (0.003Yi) or specify a smaller τb value.

Axial Stiffness:
The axial stiffnesses were reduced.

Building Example 2
4-Story Commercial

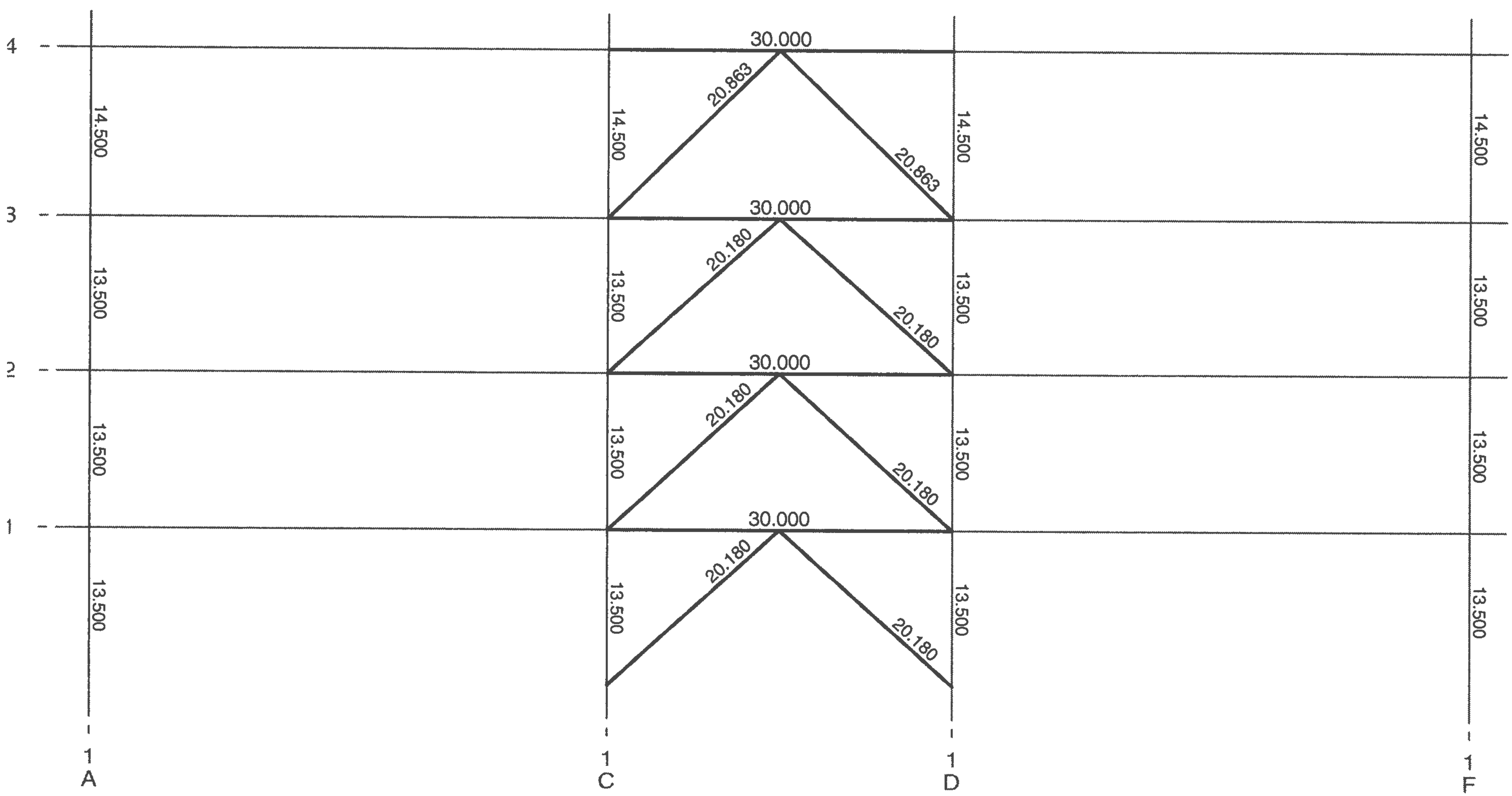


Lateral Load Resisting System

There's always a solution in steel

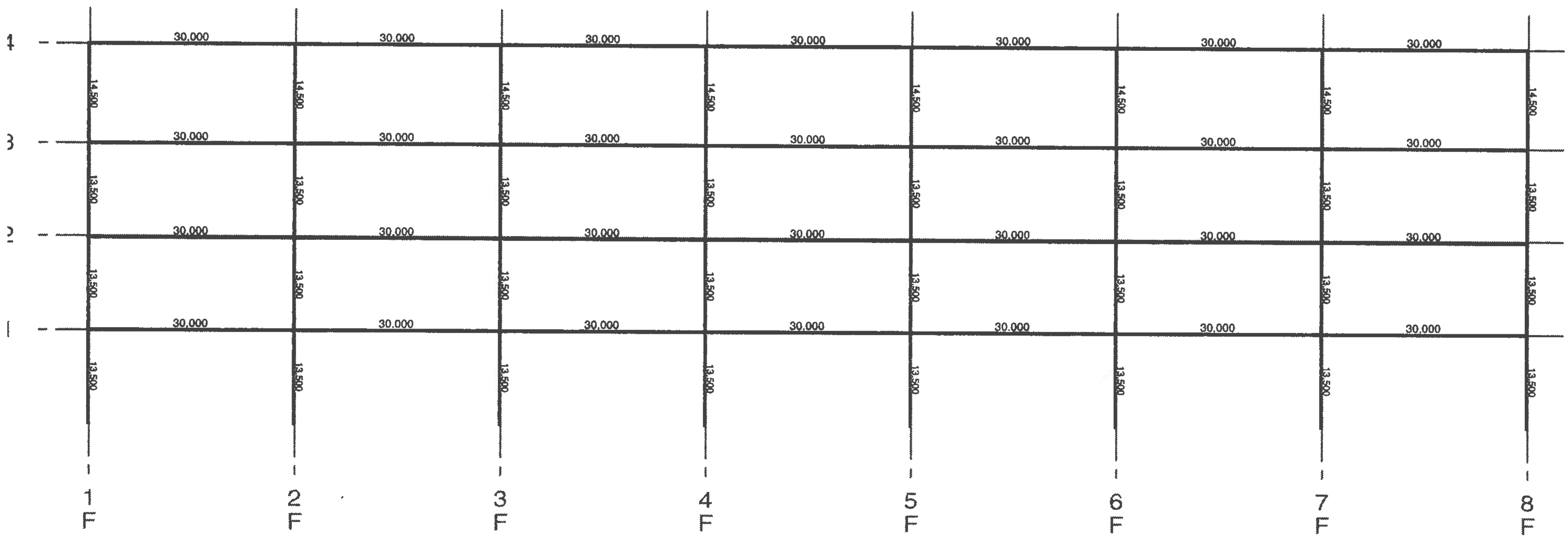
M Modeler V13.0 - Elevation

caBase: 8810 AISC 02/19/09 09:27:05



M Modeler V13.0 - Elevation

aBase: 8810 AISC 02/19/09 09:27:05



Frame Model Data

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC

02/19/09 09:27:05

STORY DATA:

Level	Story Label	Layout Type	Height ft
4	Roof	Roof	14.50
3	4th	3rd/4th	13.50
2	3rd	3rd/4th	13.50
1	2nd	2nd	13.50

FRAME MEMBERS
Frame #0:

Level: Roof

Steel Brace:

#	Level	X ft	Y ft	Fix xyt	Fy ksi	Section	BRB	T-O
1	Roof	210.000	60.000	PPP	46	HSS6X6X1/4	N	N
	4th	210.000	45.000	PPP				
2	Roof	210.000	60.000	PPP	46	HSS6X6X1/4	N	N
	4th	210.000	75.000	PPP				
3	Roof	0.000	60.000	PPP	46	HSS6X6X1/4	N	N
	4th	0.000	45.000	PPP				
4	Roof	0.000	60.000	PPP	46	HSS6X6X1/4	N	N
	4th	0.000	75.000	PPP				

Level: 4th
Frame #1:

Level: Roof

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
6	0.000	120.000	0.000	10.40	0.00	FFF	50	W12X58
				11.80	0.00	FFF		
10	30.000	120.000	0.000	10.40	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
14	60.000	120.000	0.000	10.35	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
18	90.000	120.000	0.000	10.35	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
22	120.000	120.000	0.000	10.35	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
26	150.000	120.000	0.000	10.35	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
30	180.000	120.000	0.000	10.40	0.00	FFF	50	W14X99
				11.80	0.00	FFF		

#	X	Y	Z Offset	RigMaj	RigMin	Fixity	Fy	Section
36	210.000	120.000	0.000	10.40	0.00	FFF	50	W12X58
				11.80	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
9	0.000	120.000	0.000	6.10	FFF	50	W21X50	---
	30.000	120.000	0.000	7.10	FFF			
28	30.000	120.000	0.000	7.10	FFF	50	W21X44	---
	60.000	120.000	0.000	7.10	FFF			
48	60.000	120.000	0.000	7.10	FFF	50	W21X44	---
	90.000	120.000	0.000	7.10	FFF			
67	90.000	120.000	0.000	7.10	FFF	50	W21X44	---
	120.000	120.000	0.000	7.10	FFF			
88	120.000	120.000	0.000	7.10	FFF	50	W21X44	---
	150.000	120.000	0.000	7.10	FFF			
107	150.000	120.000	0.000	7.10	FFF	50	W21X44	---
	180.000	120.000	0.000	7.10	FFF			
127	180.000	120.000	0.000	7.10	FFF	50	W21X50	---
	210.000	120.000	0.000	6.10	FFF			

Level: 4th

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
6	0.000	120.000	0.000	11.80	0.00	FFF	50	W12X58
				11.80	0.00	FFF		
10	30.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
14	60.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
18	90.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
22	120.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
26	150.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
30	180.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
36	210.000	120.000	0.000	11.80	0.00	FFF	50	W12X58
				11.80	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
10	0.000	120.000	0.000	6.10	FFF	50	W24X55	---
	30.000	120.000	0.000	7.10	FFF			
23	30.000	120.000	0.000	7.10	FFF	50	W24X55	---
	60.000	120.000	0.000	7.10	FFF			
36	60.000	120.000	0.000	7.10	FFF	50	W24X55	---
	90.000	120.000	0.000	7.10	FFF			
49	90.000	120.000	0.000	7.10	FFF	50	W24X55	---
	120.000	120.000	0.000	7.10	FFF			
63	120.000	120.000	0.000	7.10	FFF	50	W24X55	---
	150.000	120.000	0.000	7.10	FFF			
76	150.000	120.000	0.000	7.10	FFF	50	W24X55	---
	180.000	120.000	0.000	7.10	FFF			
89	180.000	120.000	0.000	7.10	FFF	50	W24X55	---
	210.000	120.000	0.000	6.10	FFF			

Level: 3rd

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
6	0.000	120.000	0.000	11.80	0.00	FFF	50	W12X58
				11.80	0.00	FFF		
10	30.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
14	60.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
18	90.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
22	120.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
26	150.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
30	180.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
36	210.000	120.000	0.000	11.80	0.00	FFF	50	W12X58
				11.80	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
10	0.000	120.000	0.000	6.10	FFF	50	W24X55	---
	30.000	120.000	0.000	7.10	FFF			
23	30.000	120.000	0.000	7.10	FFF	50	W24X55	---
	60.000	120.000	0.000	7.10	FFF			
36	60.000	120.000	0.000	7.10	FFF	50	W24X55	---

Frame Model Data

#	X	Y	Z Offset	RigEnd	Fixity	Fy	Section	T-O
	90.000	120.000	0.000	7.10	FFF			
49	90.000	120.000	0.000	7.10	FFF	50	W24X55	---
	120.000	120.000	0.000	7.10	FFF			
63	120.000	120.000	0.000	7.10	FFF	50	W24X55	---
	150.000	120.000	0.000	7.10	FFF			
76	150.000	120.000	0.000	7.10	FFF	50	W24X55	---
	180.000	120.000	0.000	7.10	FFF			
89	180.000	120.000	0.000	7.10	FFF	50	W24X55	---
	210.000	120.000	0.000	6.10	FFF			

Level: 2nd

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
6	0.000	120.000	0.000	11.80	0.00	FFF	50	W12X58
				0.00	0.00	FFF		
10	30.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
14	60.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
18	90.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
22	120.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
26	150.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
30	180.000	120.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
36	210.000	120.000	0.000	11.80	0.00	FFF	50	W12X58
				0.00	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
10	0.000	120.000	0.000	6.10	FFF	50	W24X55	---
	30.000	120.000	0.000	7.10	FFF			
23	30.000	120.000	0.000	7.10	FFF	50	W24X55	---
	60.000	120.000	0.000	7.10	FFF			
36	60.000	120.000	0.000	7.10	FFF	50	W24X55	---
	90.000	120.000	0.000	7.10	FFF			
49	90.000	120.000	0.000	7.10	FFF	50	W24X55	---
	120.000	120.000	0.000	7.10	FFF			
61	120.000	120.000	0.000	7.10	FFF	50	W24X55	---
	150.000	120.000	0.000	7.10	FFF			
74	150.000	120.000	0.000	7.10	FFF	50	W24X55	---

#	X	Y	Z Offset	RigEnd	Fixity	Fy	Section	T-O
	180.000	120.000	0.000	7.10	FFF			
87	180.000	120.000	0.000	7.10	FFF	50	W24X55	---
	210.000	120.000	0.000	6.10	FFF			

Frame #2:

Level: Roof

Steel Column:

#	X	Y	Z Offset	RigMaj	RigMin	Fixity	Fy	Section
	ft	ft	ft	in	in	xyt	ksi	
1	0.000	0.000	0.000	10.40	0.00	FFF	50	W12X58
				11.80	0.00	FFF		
7	30.000	0.000	0.000	10.40	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
11	60.000	0.000	0.000	10.35	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
15	90.000	0.000	0.000	10.35	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
19	120.000	0.000	0.000	10.35	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
23	150.000	0.000	0.000	10.35	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
27	180.000	0.000	0.000	10.40	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
31	210.000	0.000	0.000	10.40	0.00	FFF	50	W12X58
				11.80	0.00	FFF		

Steel Beam / Horiz Brace:

#	X	Y	Z Offset	RigEnd	Fixity	Fy	Section	T-O
	ft	ft	ft	in	xyt	ksi		
2	0.000	0.000	0.000	6.10	FFF	50	W21X50	---
	30.000	0.000	0.000	7.10	FFF			
23	30.000	0.000	0.000	7.10	FFF	50	W21X44	---
	60.000	0.000	0.000	7.10	FFF			
43	60.000	0.000	0.000	7.10	FFF	50	W21X44	---
	90.000	0.000	0.000	7.10	FFF			
62	90.000	0.000	0.000	7.10	FFF	50	W21X44	---
	120.000	0.000	0.000	7.10	FFF			
83	120.000	0.000	0.000	7.10	FFF	50	W21X44	---
	150.000	0.000	0.000	7.10	FFF			
102	150.000	0.000	0.000	7.10	FFF	50	W21X44	---
	180.000	0.000	0.000	7.10	FFF			
122	180.000	0.000	0.000	7.10	FFF	50	W21X50	---
	210.000	0.000	0.000	6.10	FFF			

Frame Model Data

Level: 4th

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
1	0.000	0.000	0.000	11.80	0.00	FFF	50	W12X58
				11.80	0.00	FFF		
7	30.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
11	60.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
15	90.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
19	120.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
23	150.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
27	180.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
31	210.000	0.000	0.000	11.80	0.00	FFF	50	W12X58
				11.80	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
2	0.000	0.000	0.000	6.10	FFF	50	W24X55	---
	30.000	0.000	0.000	7.10	FFF			
18	30.000	0.000	0.000	7.10	FFF	50	W24X55	---
	60.000	0.000	0.000	7.10	FFF			
31	60.000	0.000	0.000	7.10	FFF	50	W24X55	---
	90.000	0.000	0.000	7.10	FFF			
44	90.000	0.000	0.000	7.10	FFF	50	W24X55	---
	120.000	0.000	0.000	7.10	FFF			
58	120.000	0.000	0.000	7.10	FFF	50	W24X55	---
	150.000	0.000	0.000	7.10	FFF			
71	150.000	0.000	0.000	7.10	FFF	50	W24X55	---
	180.000	0.000	0.000	7.10	FFF			
84	180.000	0.000	0.000	7.10	FFF	50	W24X55	---
	210.000	0.000	0.000	6.10	FFF			

Level: 3rd

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
1	0.000	0.000	0.000	11.80	0.00	FFF	50	W12X58

#	X	Y	Z Offset	RigMaj	RigMin	Fixity	Fy	Section
				11.80	0.00	FFF		
7	30.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
11	60.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
15	90.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
19	120.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
23	150.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
27	180.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				11.80	0.00	FFF		
31	210.000	0.000	0.000	11.80	0.00	FFF	50	W12X58
				11.80	0.00	FFF		

Steel Beam / Horiz Brace:

#	X	Y	Z Offset	RigEnd	Fixity	Fy	Section	T-O
	ft	ft	ft	in	xyt	ksi		
2	0.000	0.000	0.000	6.10	FFF	50	W24X55	---
	30.000	0.000	0.000	7.10	FFF			
18	30.000	0.000	0.000	7.10	FFF	50	W24X55	---
	60.000	0.000	0.000	7.10	FFF			
31	60.000	0.000	0.000	7.10	FFF	50	W24X55	---
	90.000	0.000	0.000	7.10	FFF			
44	90.000	0.000	0.000	7.10	FFF	50	W24X55	---
	120.000	0.000	0.000	7.10	FFF			
58	120.000	0.000	0.000	7.10	FFF	50	W24X55	---
	150.000	0.000	0.000	7.10	FFF			
71	150.000	0.000	0.000	7.10	FFF	50	W24X55	---
	180.000	0.000	0.000	7.10	FFF			
84	180.000	0.000	0.000	7.10	FFF	50	W24X55	---
	210.000	0.000	0.000	6.10	FFF			

Level: 2nd

Steel Column:

#	X	Y	Z Offset	RigMaj	RigMin	Fixity	Fy	Section
	ft	ft	ft	in	in	xyt	ksi	
1	0.000	0.000	0.000	11.80	0.00	FFF	50	W12X58
				0.00	0.00	FFF		
7	30.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
11	60.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
15	90.000	0.000	0.000	11.80	0.00	FFF	50	W14X99

#	X	Y	Z Offset	RigMaj	RigMin	Fixity	Fy	Section
				0.00	0.00	FFF		
19	120.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
23	150.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
27	180.000	0.000	0.000	11.80	0.00	FFF	50	W14X99
				0.00	0.00	FFF		
31	210.000	0.000	0.000	11.80	0.00	FFF	50	W12X58
				0.00	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
2	0.000	0.000	0.000	6.10	FFF	50	W24X55	---
	30.000	0.000	0.000	7.10	FFF			
18	30.000	0.000	0.000	7.10	FFF	50	W24X55	---
	60.000	0.000	0.000	7.10	FFF			
31	60.000	0.000	0.000	7.10	FFF	50	W24X55	---
	90.000	0.000	0.000	7.10	FFF			
44	90.000	0.000	0.000	7.10	FFF	50	W24X55	---
	120.000	0.000	0.000	7.10	FFF			
56	120.000	0.000	0.000	7.10	FFF	50	W24X55	---
	150.000	0.000	0.000	7.10	FFF			
69	150.000	0.000	0.000	7.10	FFF	50	W24X55	---
	180.000	0.000	0.000	7.10	FFF			
82	180.000	0.000	0.000	7.10	FFF	50	W24X55	---
	210.000	0.000	0.000	6.10	FFF			

Frame #3:

Level: Roof

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
3	0.000	45.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		
4	0.000	75.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
4	0.000	45.000	0.000	0.00	PPF	50	W18X35	---
	0.000	75.000	0.000	0.00	PPF			

Frame Model Data

Level: 4th

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
3	0.000	45.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		
4	0.000	75.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
4	0.000	45.000	0.000	0.00	PPF	50	W18X40	---
	0.000	75.000	0.000	0.00	PPF			

Steel Brace:

#	Level	X ft	Y ft	Fix xyt	Fy ksi	Section	BRB	T-O
1	4th	0.000	60.000	PPP	46	HSS6X6X1/4	N	N
	3rd	0.000	45.000	PPP				
2	4th	0.000	60.000	PPP	46	HSS6X6X1/4	N	N
	3rd	0.000	75.000	PPP				

Level: 3rd

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
3	0.000	45.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		
4	0.000	75.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
4	0.000	45.000	0.000	0.00	PPF	50	W18X40	---
	0.000	75.000	0.000	0.00	PPF			

Steel Brace:

#	Level	X ft	Y ft	Fix xyt	Fy ksi	Section	BRB	T-O
1	3rd	0.000	60.000	PPP	46	HSS6X6X1/2	N	N
	2nd	0.000	45.000	PPP				
2	3rd	0.000	60.000	PPP	46	HSS6X6X1/2	N	N

#	Level	X	Y	Fix	Fy	Section	BRB	T-O
	2nd	0.000	75.000	PPP				

Level: 2nd

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
3	0.000	45.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	PPF		
4	0.000	75.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	PPF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
4	0.000	45.000	0.000	0.00	PPF	50	W18X40	---
	0.000	75.000	0.000	0.00	PPF			

Steel Brace:

#	Level	X ft	Y ft	Fix xyt	Fy ksi	Section	BRB	T-O
1	2nd	0.000	60.000	PPP	46	HSS6X6X1/2	N	N
	Base	0.000	45.000	PPP				
2	2nd	0.000	60.000	PPP	46	HSS6X6X1/2	N	N
	Base	0.000	75.000	PPP				

Frame #4:

Level: Roof

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
33	210.000	45.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		
34	210.000	75.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
142	210.000	45.000	0.000	0.00	PPF	50	W18X35	---
	210.000	75.000	0.000	0.00	PPF			

Level: 4th

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
33	210.000	45.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		
34	210.000	75.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
99	210.000	45.000	0.000	0.00	PPF	50	W18X40	---
	210.000	75.000	0.000	0.00	PPF			

Steel Brace:

#	Level	X ft	Y ft	Fix xyt	Fy ksi	Section	BRB	T-O
3	4th	210.000	60.000	PPP	46	HSS6X6X1/4	N	N
	3rd	210.000	45.000	PPP				
4	4th	210.000	60.000	PPP	46	HSS6X6X1/4	N	N
	3rd	210.000	75.000	PPP				

Level: 3rd

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
33	210.000	45.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		
34	210.000	75.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	FFF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
99	210.000	45.000	0.000	0.00	PPF	50	W18X40	---
	210.000	75.000	0.000	0.00	PPF			

Steel Brace:

#	Level	X ft	Y ft	Fix xyt	Fy ksi	Section	BRB	T-O
3	3rd	210.000	60.000	PPP	46	HSS6X6X1/2	N	N
	2nd	210.000	45.000	PPP				
4	3rd	210.000	60.000	PPP	46	HSS6X6X1/2	N	N
	2nd	210.000	75.000	PPP				

Level: 2nd

Steel Column:

#	X ft	Y ft	Z Offset ft	RigMaj in	RigMin in	Fixity xyt	Fy ksi	Section
33	210.000	45.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	PPF		
34	210.000	75.000	0.000	0.00	0.00	FFF	50	W12X53
				0.00	0.00	PPF		

Steel Beam / Horiz Brace:

#	X ft	Y ft	Z Offset ft	RigEnd in	Fixity xyt	Fy ksi	Section	T-O
97	210.000	45.000	0.000	0.00	PPF	50	W18X40	---
	210.000	75.000	0.000	0.00	PPF			

Steel Brace:

#	Level	X ft	Y ft	Fix xyt	Fy ksi	Section	BRB	T-O
3	2nd	210.000	60.000	PPP	46	HSS6X6X1/2	N	N
	Base	210.000	45.000	PPP				
4	2nd	210.000	60.000	PPP	46	HSS6X6X1/2	N	N
	Base	210.000	75.000	PPP				

NODES:

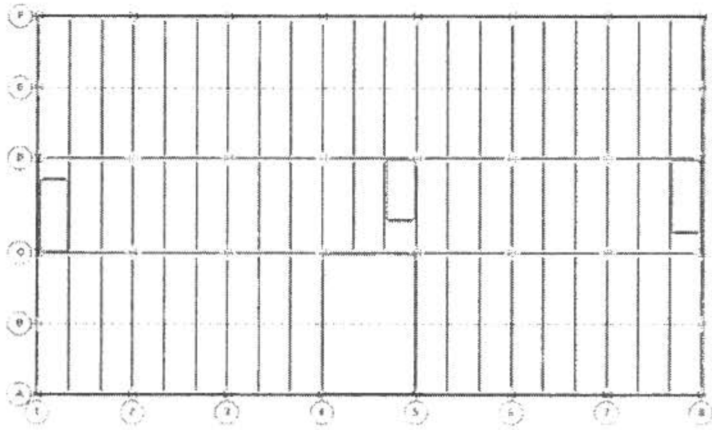
#	X ft	Y ft	Z ft	Fdtn	Diaphr (Diaph.# - Story Name)
1	0.000	0.000	55.000	N	1 - Roof
2	0.000	45.000	55.000	N	1 - Roof
3	0.000	60.000	55.000	N	None
4	0.000	75.000	55.000	N	1 - Roof
5	0.000	120.000	55.000	N	1 - Roof
6	30.000	0.000	55.000	N	1 - Roof
7	30.000	120.000	55.000	N	1 - Roof
8	60.000	0.000	55.000	N	1 - Roof
9	60.000	120.000	55.000	N	1 - Roof
10	90.000	0.000	55.000	N	1 - Roof
11	90.000	120.000	55.000	N	1 - Roof
12	120.000	0.000	55.000	N	1 - Roof
13	120.000	120.000	55.000	N	1 - Roof
14	150.000	0.000	55.000	N	1 - Roof
15	150.000	120.000	55.000	N	1 - Roof
16	180.000	0.000	55.000	N	1 - Roof
17	180.000	120.000	55.000	N	1 - Roof
18	210.000	0.000	55.000	N	1 - Roof
19	210.000	45.000	55.000	N	1 - Roof
20	210.000	60.000	55.000	N	None
21	210.000	75.000	55.000	N	1 - Roof
22	210.000	120.000	55.000	N	1 - Roof

#	X	Y	Z	Fdtn	Diaphr
23	0.000	0.000	40.500	N	1 - 4th
24	0.000	45.000	40.500	N	1 - 4th
25	0.000	60.000	40.500	N	None
26	0.000	75.000	40.500	N	1 - 4th
27	0.000	120.000	40.500	N	1 - 4th
28	30.000	0.000	40.500	N	1 - 4th
29	30.000	120.000	40.500	N	1 - 4th
30	60.000	0.000	40.500	N	1 - 4th
31	60.000	120.000	40.500	N	1 - 4th
32	90.000	0.000	40.500	N	1 - 4th
33	90.000	120.000	40.500	N	1 - 4th
34	120.000	0.000	40.500	N	1 - 4th
35	120.000	120.000	40.500	N	1 - 4th
36	150.000	0.000	40.500	N	1 - 4th
37	150.000	120.000	40.500	N	1 - 4th
38	180.000	0.000	40.500	N	1 - 4th
39	180.000	120.000	40.500	N	1 - 4th
40	210.000	0.000	40.500	N	1 - 4th
41	210.000	45.000	40.500	N	1 - 4th
42	210.000	60.000	40.500	N	None
43	210.000	75.000	40.500	N	1 - 4th
44	210.000	120.000	40.500	N	1 - 4th
45	0.000	0.000	27.000	N	1 - 3rd
46	0.000	45.000	27.000	N	1 - 3rd
47	0.000	60.000	27.000	N	None
48	0.000	75.000	27.000	N	1 - 3rd
49	0.000	120.000	27.000	N	1 - 3rd
50	30.000	0.000	27.000	N	1 - 3rd
51	30.000	120.000	27.000	N	1 - 3rd
52	60.000	0.000	27.000	N	1 - 3rd
53	60.000	120.000	27.000	N	1 - 3rd
54	90.000	0.000	27.000	N	1 - 3rd
55	90.000	120.000	27.000	N	1 - 3rd
56	120.000	0.000	27.000	N	1 - 3rd
57	120.000	120.000	27.000	N	1 - 3rd
58	150.000	0.000	27.000	N	1 - 3rd
59	150.000	120.000	27.000	N	1 - 3rd
60	180.000	0.000	27.000	N	1 - 3rd
61	180.000	120.000	27.000	N	1 - 3rd
62	210.000	0.000	27.000	N	1 - 3rd
63	210.000	45.000	27.000	N	1 - 3rd
64	210.000	60.000	27.000	N	None
65	210.000	75.000	27.000	N	1 - 3rd
66	210.000	120.000	27.000	N	1 - 3rd
67	0.000	0.000	13.500	N	1 - 2nd

Frame Model Data

#	X	Y	Z	Fdtn	Diaphr
68	0.000	45.000	13.500	N	1 - 2nd
69	0.000	60.000	13.500	N	None
70	0.000	75.000	13.500	N	1 - 2nd
71	0.000	120.000	13.500	N	1 - 2nd
72	30.000	0.000	13.500	N	1 - 2nd
73	30.000	120.000	13.500	N	1 - 2nd
74	60.000	0.000	13.500	N	1 - 2nd
75	60.000	120.000	13.500	N	1 - 2nd
76	90.000	0.000	13.500	N	1 - 2nd
77	90.000	120.000	13.500	N	1 - 2nd
78	120.000	0.000	13.500	N	1 - 2nd
79	120.000	120.000	13.500	N	1 - 2nd
80	150.000	0.000	13.500	N	1 - 2nd
81	150.000	120.000	13.500	N	1 - 2nd
82	180.000	0.000	13.500	N	1 - 2nd
83	180.000	120.000	13.500	N	1 - 2nd
84	210.000	0.000	13.500	N	1 - 2nd
85	210.000	45.000	13.500	N	1 - 2nd
86	210.000	60.000	13.500	N	None
87	210.000	75.000	13.500	N	1 - 2nd
88	210.000	120.000	13.500	N	1 - 2nd
89	0.000	0.000	0.000	Y	--
90	0.000	45.000	0.000	Y	--
91	0.000	75.000	0.000	Y	--
92	0.000	120.000	0.000	Y	--
93	30.000	0.000	0.000	Y	--
94	30.000	120.000	0.000	Y	--
95	60.000	0.000	0.000	Y	--
96	60.000	120.000	0.000	Y	--
97	90.000	0.000	0.000	Y	--
98	90.000	120.000	0.000	Y	--
99	120.000	0.000	0.000	Y	--
100	120.000	120.000	0.000	Y	--
101	150.000	0.000	0.000	Y	--
102	150.000	120.000	0.000	Y	--
103	180.000	0.000	0.000	Y	--
104	180.000	120.000	0.000	Y	--
105	210.000	0.000	0.000	Y	--
106	210.000	45.000	0.000	Y	--
107	210.000	75.000	0.000	Y	--
108	210.000	120.000	0.000	Y	--

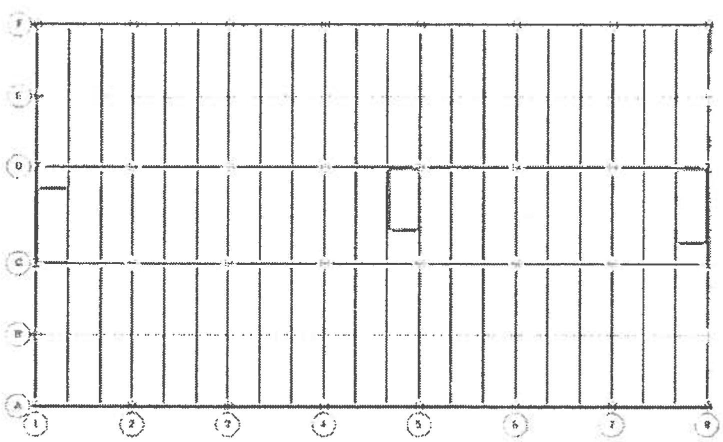
Building Example 2
4-Story Commercial



There's always a solution in steel

2nd Floor Framing

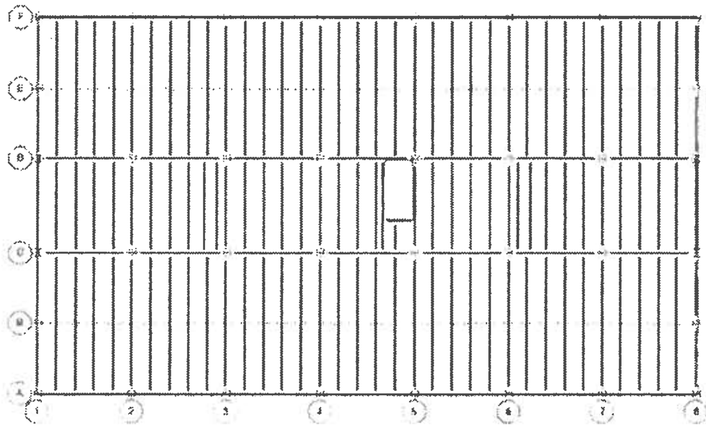
Building Example 2
4-Story Commercial



There's always a solution in steel

3rd and 4th Floor Framing

Building Example 2
4-Story Commercial



There's always a solution in steel

Roof Framing

Loads and Applied Forces

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC

02/19/09 08:23:44

LOAD CASE: Wind

Wind ASCE 7-05/IBC2006
Exposure: C
Basic Wind Speed (mph): 90.0 Importance Factor: 1.000
Apply Directionality Factor, Kd = 0.85
Use Topography Factor, Kzt: 1.00
Use Calculated Frequency for X-Dir.
Use Calculated Frequency for Y-Dir.
Gust Factor for Flexible Structures, G: Use Calculated G for X-Dir.
Gust Factor for Flexible Structures, G: Use Calculated G for Y-Dir.
Damping Ratio for Flexible Structures= 0.01
Mean Roof Height (ft): Top Story Height + Parapet = 55.00
Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 0.528 Structure is Flexible
Y-Direction: Natural Frequency = 0.890 Structure is Flexible
CpWindward = 0.80 qLeeward (qh) = 19.67 psf
GCpn (Parapet): Windward = 1.50 Leeward = -1.00

Height	Kz	Kzt	qz	Gust Factor G		CpLeeward		Pressure (psf)	
ft			psf	X	Y	X	Y	X	Y
55.00	1.116	1.000	19.669	1.005	0.878	-0.351	-0.500	22.761	22.472
40.50	1.046	1.000	18.442	1.005	0.878	-0.351	-0.500	21.774	21.609
27.00	0.961	1.000	16.933	1.005	0.878	-0.351	-0.500	20.561	20.548
13.50	0.849	1.000	14.962	1.005	0.878	-0.351	-0.500	18.976	19.162
0.00	0.849	1.000	14.962	1.005	0.878	-0.351	-0.500	18.976	19.162

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_1_X

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	19.68	0.00	105.00	60.00
4th	1	40.50	36.84	0.00	105.00	60.00
3rd	1	27.00	33.48	0.00	105.00	60.00
2nd	1	13.50	31.43	0.00	105.00	60.00

APPLIED STORY FORCES

Type: Wind_IBC06_1_X

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	19.68	0.00
4th	40.50	36.84	0.00
3rd	27.00	33.48	0.00
2nd	13.50	31.43	0.00
		121.43	0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	0.00	33.94	105.00	60.00
4th	1	40.50	0.00	63.77	105.00	60.00
3rd	1	27.00	0.00	58.38	105.00	60.00
2nd	1	13.50	0.00	55.24	105.00	60.00

APPLIED STORY FORCES

Type: Wind_IBC06_1_Y

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	0.00	33.94
4th	40.50	0.00	63.77
3rd	27.00	0.00	58.38
2nd	13.50	0.00	55.24
		0.00	211.32

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_2_X+E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	14.76	0.00	105.00	75.52
4th	1	40.50	27.63	0.00	105.00	75.52
3rd	1	27.00	25.11	0.00	105.00	75.52
2nd	1	13.50	23.57	0.00	105.00	75.55

APPLIED STORY FORCES

Type: Wind_IBC06_2_X+E

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	14.76	0.00
4th	40.50	27.63	0.00
3rd	27.00	25.11	0.00
2nd	13.50	23.57	0.00
		91.08	0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_2_X-E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	14.76	0.00	105.00	44.48
4th	1	40.50	27.63	0.00	105.00	44.48

3rd	1	27.00	25.11	0.00	105.00	44.48
2nd	1	13.50	23.57	0.00	105.00	44.45

APPLIED STORY FORCES

Type: Wind_IBC06_2_X-E

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	14.76	0.00
4th	40.50	27.63	0.00
3rd	27.00	25.11	0.00
2nd	13.50	23.57	0.00
		91.08	0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_2_Y+E

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	0.00	25.45	135.42	60.00
4th	1	40.50	0.00	47.83	135.42	60.00
3rd	1	27.00	0.00	43.78	135.42	60.00
2nd	1	13.50	0.00	41.43	135.42	60.00

APPLIED STORY FORCES

Type: Wind_IBC06_2_Y+E

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	0.00	25.45
4th	40.50	0.00	47.83
3rd	27.00	0.00	43.78
2nd	13.50	0.00	41.43
		0.00	158.49

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_2_Y-E

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	0.00	25.45	74.58	60.00
4th	1	40.50	0.00	47.83	74.58	60.00
3rd	1	27.00	0.00	43.78	74.58	60.00
2nd	1	13.50	0.00	41.43	74.58	60.00

APPLIED STORY FORCES

Type: Wind_IBC06_2_Y-E

Level	Ht	Fx	Fy
	ft	kips	kips

Roof	55.00	0.00	25.45
4th	40.50	0.00	47.83
3rd	27.00	0.00	43.78
2nd	13.50	0.00	41.43
		0.00	158.49

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_3_X+Y

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	14.76	25.45	105.00	60.00
4th	1	40.50	27.63	47.83	105.00	60.00
3rd	1	27.00	25.11	43.78	105.00	60.00
2nd	1	13.50	23.57	41.43	105.00	60.00

APPLIED STORY FORCES

Type: Wind_IBC06_3_X+Y

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	14.76	25.45
4th	40.50	27.63	47.83
3rd	27.00	25.11	43.78
2nd	13.50	23.57	41.43
		91.08	158.49

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_3_X-Y

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	14.76	-25.45	105.00	60.00
4th	1	40.50	27.63	-47.83	105.00	60.00
3rd	1	27.00	25.11	-43.78	105.00	60.00
2nd	1	13.50	23.57	-41.43	105.00	60.00

APPLIED STORY FORCES

Type: Wind_IBC06_3_X-Y

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	14.76	-25.45
4th	40.50	27.63	-47.83
3rd	27.00	25.11	-43.78
2nd	13.50	23.57	-41.43
		91.08	-158.49

Loads and Applied Forces

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_4_X+Y_CW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	11.07	19.09	74.58	75.52
4th	1	40.50	20.72	35.87	74.58	75.52
3rd	1	27.00	18.84	32.84	74.58	75.52
2nd	1	13.50	17.68	31.07	74.58	75.55

APPLIED STORY FORCES

Type: Wind_IBC06_4_X+Y_CW

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	11.07	19.09
4th	40.50	20.72	35.87
3rd	27.00	18.84	32.84
2nd	13.50	17.68	31.07
		68.31	118.87

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_4_X+Y_CCW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	11.07	19.09	135.42	44.48
4th	1	40.50	20.72	35.87	135.42	44.48
3rd	1	27.00	18.84	32.84	135.42	44.48
2nd	1	13.50	17.68	31.07	135.42	44.45

APPLIED STORY FORCES

Type: Wind_IBC06_4_X+Y_CCW

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	11.07	19.09
4th	40.50	20.72	35.87
3rd	27.00	18.84	32.84
2nd	13.50	17.68	31.07
		68.31	118.87

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_4_X-Y_CW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	11.07	-19.09	135.42	75.52
4th	1	40.50	20.72	-35.87	135.42	75.52

3rd	1	27.00	18.84	-32.84	135.42	75.52
2nd	1	13.50	17.68	-31.07	135.42	75.55

APPLIED STORY FORCES

Type: Wind_IBC06_4_X-Y_CW

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	11.07	-19.09
4th	40.50	20.72	-35.87
3rd	27.00	18.84	-32.84
2nd	13.50	17.68	-31.07
		68.31	-118.87

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC06_4_X-Y_CCW

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	11.07	-19.09	74.58	44.48
4th	1	40.50	20.72	-35.87	74.58	44.48
3rd	1	27.00	18.84	-32.84	74.58	44.48
2nd	1	13.50	17.68	-31.07	74.58	44.45

APPLIED STORY FORCES

Type: Wind_IBC06_4_X-Y_CCW

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	11.07	-19.09
4th	40.50	20.72	-35.87
3rd	27.00	18.84	-32.84
2nd	13.50	17.68	-31.07
		68.31	-118.87

Loads and Applied Forces

LOAD CASE: E

Seismic ASCE 7-05 / IBC 2006 Equivalent Lateral Force
Site Class: D Importance Factor: 1.00 Ss: 0.121 g S1: 0.060 g TL: 6.00 s
Fa: 1.600 Fv: 2.400 SDs: 0.129 g SD1: 0.096 g
Occupancy Category: I Seismic Design Category: B
Provisions for: Force
Ground Level: Base

Dir	Eccent	R	Ta Equation	Building Period-T
X	+ And -	3.0	Std,Ct=0.020,x=0.75	Calculated
Y	+ And -	3.0	Std,Ct=0.020,x=0.75	Calculated

Dir	Ta	Cu	T	T-used	Eq12.8-2	Eq12.8-3	Eq12.8-5	k
X	0.404	1.700	1.896	0.687	0.043	0.047	0.0100	1.093
Y	0.404	1.700	1.124	0.687	0.043	0.047	0.0100	1.093

Total Building Weight (kips) = 8152.95

APPLIED DIAPHRAGM FORCES

Type: EQ_IBC06_X_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	64.91	0.00	105.00	66.05
4th	1	40.50	148.39	0.00	105.00	66.05
3rd	1	27.00	95.25	0.00	105.00	66.05
2nd	1	13.50	42.21	0.00	105.00	67.77

APPLIED STORY FORCES

Type: EQ_IBC06_X_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	64.91	0.00
4th	40.50	148.39	0.00
3rd	27.00	95.25	0.00
2nd	13.50	42.21	0.00
		350.76	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_IBC06_X_-E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	64.91	0.00	105.00	53.95
4th	1	40.50	148.39	0.00	105.00	53.95
3rd	1	27.00	95.25	0.00	105.00	53.95
2nd	1	13.50	42.21	0.00	105.00	55.67

3rd	27.00	0.00	95.25
2nd	13.50	0.00	42.21
		0.00	350.76

LOAD CASE: N

Notional AISC 360-05
Fraction of Gravity Load : 0.0020
Ground Level: Base
Total Dead Load (kips) = 7403.09
Total Live Load (kips) = 2399.55
Total Snow Load (kips) = 638.28

APPLIED DIAPHRAGM FORCES

Type: NL_AISC360_DL_X

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	1.57	0.00	105.00	60.00
4th	1	40.50	4.49	0.00	105.00	60.00
3rd	1	27.00	4.49	0.00	105.00	60.00
2nd	1	13.50	4.25	0.00	105.00	61.69

APPLIED STORY FORCES

Type: NL_AISC360_DL_X

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	1.57	0.00
4th	40.50	4.49	0.00
3rd	27.00	4.49	0.00
2nd	13.50	4.25	0.00
		14.81	0.00

APPLIED DIAPHRAGM FORCES

Type: NL_AISC360_DL_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	55.00	0.00	1.57	105.00	60.00
4th	1	40.50	0.00	4.49	105.00	60.00
3rd	1	27.00	0.00	4.49	105.00	60.00
2nd	1	13.50	0.00	4.25	105.00	61.69

APPLIED STORY FORCES

Type: NL_AISC360_DL_Y

Level	Ht ft	Fx kips	Fy kips
Roof	55.00	0.00	1.57
4th	40.50	0.00	4.49
3rd	27.00	0.00	4.49
2nd	13.50	0.00	4.25
		0.00	14.81

APPLIED DIAPHRAGM FORCES

Type: NL_AISC360_LL_X

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	0.00	0.00	0.00	0.00
4th	1	40.50	1.63	0.00	105.00	60.00
3rd	1	27.00	1.63	0.00	105.00	60.00
2nd	1	13.50	1.53	0.00	105.00	62.00

APPLIED STORY FORCES

Type: NL_AISC360_LL_X

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	0.00	0.00
4th	40.50	1.63	0.00
3rd	27.00	1.63	0.00
2nd	13.50	1.53	0.00
		4.80	0.00

APPLIED DIAPHRAGM FORCES

Type: NL_AISC360_LL_Y

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	0.00	0.00	0.00	0.00
4th	1	40.50	0.00	1.63	105.00	60.00
3rd	1	27.00	0.00	1.63	105.00	60.00
2nd	1	13.50	0.00	1.53	105.00	62.00

APPLIED STORY FORCES

Type: NL_AISC360_LL_Y

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	0.00	0.00
4th	40.50	0.00	1.63
3rd	27.00	0.00	1.63
2nd	13.50	0.00	1.53
		0.00	4.80

APPLIED DIAPHRAGM FORCES

Type: NL_AISC360_Rf_X

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	1.28	0.00	105.00	60.00
4th	1	40.50	0.00	0.00	0.00	0.00

3rd	1	27.00	0.00	0.00	0.00	0.00
2nd	1	13.50	0.00	0.00	0.00	0.00

APPLIED STORY FORCES

Type: NL_AISC360_Rf_X

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	1.28	0.00
4th	40.50	0.00	0.00
3rd	27.00	0.00	0.00
2nd	13.50	0.00	0.00
		1.28	0.00

APPLIED DIAPHRAGM FORCES

Type: NL_AISC360_Rf_Y

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	55.00	0.00	1.28	105.00	60.00
4th	1	40.50	0.00	0.00	0.00	0.00
3rd	1	27.00	0.00	0.00	0.00	0.00
2nd	1	13.50	0.00	0.00	0.00	0.00

APPLIED STORY FORCES

Type: NL_AISC360_Rf_Y

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	55.00	0.00	1.28
4th	40.50	0.00	0.00
3rd	27.00	0.00	0.00
2nd	13.50	0.00	0.00
		0.00	1.28

Load Combinations

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC 02/19/09 08:23:44

LOAD COMBINATION CRITERIA:

Roof Live Load:	Snow
Snow Factor	Do Not Include Snow in Combinations with Seismic
Notional Loads	Consider with Combinations containing only gravity loads
Sds (for Ev)	0.000
RhoX	1.000
RhoY	1.000

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW
E1	E	EQ_IBC06_X_+E_F
E2	E	EQ_IBC06_X_-E_F
E3	E	EQ_IBC06_Y_+E_F
E4	E	EQ_IBC06_Y_-E_F
ND1	N	NL_AISC360_DL_X
ND2	N	NL_AISC360_DL_Y
NL1	N	NL_AISC360_LL_X
NL2	N	NL_AISC360_LL_Y
NR1	N	NL_AISC360_Rf_X
NR2	N	NL_AISC360_Rf_Y

LOAD COMBINATIONS: IBC06/ASCE7-05 ASD

1	*	1.000 D + 1.000 ND1
2	*	1.000 D + 1.000 ND2
3	*	1.000 D - 1.000 ND1
4	*	1.000 D - 1.000 ND2
5	*	1.000 D + 1.000 ND1 + 1.000 Lp + 1.000 NL1
6	*	1.000 D + 1.000 ND2 + 1.000 Lp + 1.000 NL2
7	*	1.000 D - 1.000 ND1 + 1.000 Lp - 1.000 NL1
8	*	1.000 D - 1.000 ND2 + 1.000 Lp - 1.000 NL2
9	*	1.000 D + 1.000 ND1 + 1.000 Sp + 1.000 NR1
10	*	1.000 D + 1.000 ND2 + 1.000 Sp + 1.000 NR2

11	*	1.000 D - 1.000 ND1 + 1.000 Sp - 1.000 NR1
12	*	1.000 D - 1.000 ND2 + 1.000 Sp - 1.000 NR2
13	*	1.000 D + 1.000 ND1 + 0.750 Lp + 1.000 NL1 + 0.750 Sp + 0.750 NR1
14	*	1.000 D + 1.000 ND2 + 0.750 Lp + 1.000 NL2 + 0.750 Sp + 0.750 NR2
15	*	1.000 D - 1.000 ND1 + 0.750 Lp - 1.000 NL1 + 0.750 Sp - 0.750 NR1
16	*	1.000 D - 1.000 ND2 + 0.750 Lp - 1.000 NL2 + 0.750 Sp - 0.750 NR2
17	*	1.000 D + 1.000 W1
18	*	1.000 D + 1.000 W2
19	*	1.000 D + 1.000 W3
20	*	1.000 D + 1.000 W4
21	*	1.000 D + 1.000 W5
22	*	1.000 D + 1.000 W6
23	*	1.000 D + 1.000 W7
24	*	1.000 D + 1.000 W8
25	*	1.000 D + 1.000 W9
26	*	1.000 D + 1.000 W10
27	*	1.000 D + 1.000 W11
28	*	1.000 D + 1.000 W12
29	*	1.000 D - 1.000 W1
30	*	1.000 D - 1.000 W2
31	*	1.000 D - 1.000 W3
32	*	1.000 D - 1.000 W4
33	*	1.000 D - 1.000 W5
34	*	1.000 D - 1.000 W6
35	*	1.000 D - 1.000 W7
36	*	1.000 D - 1.000 W8
37	*	1.000 D - 1.000 W9
38	*	1.000 D - 1.000 W10
39	*	1.000 D - 1.000 W11
40	*	1.000 D - 1.000 W12
41	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W1
42	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W2
43	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W3
44	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W4
45	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W5
46	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W6
47	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W7
48	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W8
49	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W9
50	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W10
51	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W11
52	*	1.000 D + 0.750 Lp + 0.750 Sp + 0.750 W12
53	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W1
54	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W2
55	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W3
56	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W4

57	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W5
58	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W6
59	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W7
60	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W8
61	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W9
62	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W10
63	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W11
64	*	1.000 D + 0.750 Lp + 0.750 Sp - 0.750 W12
65	*	1.000 D + 0.750 Lp + 0.750 W1
66	*	1.000 D + 0.750 Lp + 0.750 W2
67	*	1.000 D + 0.750 Lp + 0.750 W3
68	*	1.000 D + 0.750 Lp + 0.750 W4
69	*	1.000 D + 0.750 Lp + 0.750 W5
70	*	1.000 D + 0.750 Lp + 0.750 W6
71	*	1.000 D + 0.750 Lp + 0.750 W7
72	*	1.000 D + 0.750 Lp + 0.750 W8
73	*	1.000 D + 0.750 Lp + 0.750 W9
74	*	1.000 D + 0.750 Lp + 0.750 W10
75	*	1.000 D + 0.750 Lp + 0.750 W11
76	*	1.000 D + 0.750 Lp + 0.750 W12
77	*	1.000 D + 0.750 Lp - 0.750 W1
78	*	1.000 D + 0.750 Lp - 0.750 W2
79	*	1.000 D + 0.750 Lp - 0.750 W3
80	*	1.000 D + 0.750 Lp - 0.750 W4
81	*	1.000 D + 0.750 Lp - 0.750 W5
82	*	1.000 D + 0.750 Lp - 0.750 W6
83	*	1.000 D + 0.750 Lp - 0.750 W7
84	*	1.000 D + 0.750 Lp - 0.750 W8
85	*	1.000 D + 0.750 Lp - 0.750 W9
86	*	1.000 D + 0.750 Lp - 0.750 W10
87	*	1.000 D + 0.750 Lp - 0.750 W11
88	*	1.000 D + 0.750 Lp - 0.750 W12
89	*	1.000 D + 0.750 Sp + 0.750 W1
90	*	1.000 D + 0.750 Sp + 0.750 W2
91	*	1.000 D + 0.750 Sp + 0.750 W3
92	*	1.000 D + 0.750 Sp + 0.750 W4
93	*	1.000 D + 0.750 Sp + 0.750 W5
94	*	1.000 D + 0.750 Sp + 0.750 W6
95	*	1.000 D + 0.750 Sp + 0.750 W7
96	*	1.000 D + 0.750 Sp + 0.750 W8
97	*	1.000 D + 0.750 Sp + 0.750 W9
98	*	1.000 D + 0.750 Sp + 0.750 W10
99	*	1.000 D + 0.750 Sp + 0.750 W11
100	*	1.000 D + 0.750 Sp + 0.750 W12
101	*	1.000 D + 0.750 Sp - 0.750 W1
102	*	1.000 D + 0.750 Sp - 0.750 W2

103	*	1.000 D + 0.750 Sp - 0.750 W3
104	*	1.000 D + 0.750 Sp - 0.750 W4
105	*	1.000 D + 0.750 Sp - 0.750 W5
106	*	1.000 D + 0.750 Sp - 0.750 W6
107	*	1.000 D + 0.750 Sp - 0.750 W7
108	*	1.000 D + 0.750 Sp - 0.750 W8
109	*	1.000 D + 0.750 Sp - 0.750 W9
110	*	1.000 D + 0.750 Sp - 0.750 W10
111	*	1.000 D + 0.750 Sp - 0.750 W11
112	*	1.000 D + 0.750 Sp - 0.750 W12
113	*	0.600 D + 1.000 W1
114	*	0.600 D + 1.000 W2
115	*	0.600 D + 1.000 W3
116	*	0.600 D + 1.000 W4
117	*	0.600 D + 1.000 W5
118	*	0.600 D + 1.000 W6
119	*	0.600 D + 1.000 W7
120	*	0.600 D + 1.000 W8
121	*	0.600 D + 1.000 W9
122	*	0.600 D + 1.000 W10
123	*	0.600 D + 1.000 W11
124	*	0.600 D + 1.000 W12
125	*	0.600 D - 1.000 W1
126	*	0.600 D - 1.000 W2
127	*	0.600 D - 1.000 W3
128	*	0.600 D - 1.000 W4
129	*	0.600 D - 1.000 W5
130	*	0.600 D - 1.000 W6
131	*	0.600 D - 1.000 W7
132	*	0.600 D - 1.000 W8
133	*	0.600 D - 1.000 W9
134	*	0.600 D - 1.000 W10
135	*	0.600 D - 1.000 W11
136	*	0.600 D - 1.000 W12
137	*	1.000 D + 0.700 E1
138	*	1.000 D + 0.700 E2
139	*	1.000 D + 0.700 E3
140	*	1.000 D + 0.700 E4
141	*	1.000 D - 0.700 E1
142	*	1.000 D - 0.700 E2
143	*	1.000 D - 0.700 E3
144	*	1.000 D - 0.700 E4
145	*	1.000 D + 0.750 Lp + 0.525 E1
146	*	1.000 D + 0.750 Lp + 0.525 E2
147	*	1.000 D + 0.750 Lp + 0.525 E3
148	*	1.000 D + 0.750 Lp + 0.525 E4

Load Combinations

149	*	1.000 D + 0.750 Lp - 0.525 E1
150	*	1.000 D + 0.750 Lp - 0.525 E2
151	*	1.000 D + 0.750 Lp - 0.525 E3
152	*	1.000 D + 0.750 Lp - 0.525 E4
153	*	0.600 D + 0.700 E1
154	*	0.600 D + 0.700 E2
155	*	0.600 D + 0.700 E3
156	*	0.600 D + 0.700 E4
157	*	0.600 D - 0.700 E1
158	*	0.600 D - 0.700 E2
159	*	0.600 D - 0.700 E3
160	*	0.600 D - 0.700 E4

* = Load combination currently selected to use

Drift

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

02/19/09 08:20:50
Steel Code: IBC

CRITERIA:

Rigid End Zones:	Ignore Effects
Member Force Output:	At Face of Joint
P-Delta:	No
Diaphragm:	Rigid
Ground Level:	Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW
E1	E	EQ_IBC06_X_+E_F
E2	E	EQ_IBC06_X_-E_F
E3	E	EQ_IBC06_Y_+E_F
E4	E	EQ_IBC06_Y_-E_F
ND1	N	NL_AISC360_DL_X
ND2	N	NL_AISC360_DL_Y
NL1	N	NL_AISC360_LL_X
NL2	N	NL_AISC360_LL_Y
NR1	N	NL_AISC360_Rf_X
NR2	N	NL_AISC360_Rf_Y

RESULTS:

Location (ft): (105.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Lp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Sp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W1	0.7325	-0.0000	0.0895	-0.0000	0.0005	0.0000
	W2	0.0000	0.4846	0.0000	0.0893	0.0000	0.0005
	W3	0.5336	-0.0000	0.0644	-0.0000	0.0004	0.0000
	W4	0.5652	0.0000	0.0698	0.0000	0.0004	0.0000
	W5	0.0536	0.3634	0.0092	0.0669	0.0001	0.0004
	W6	-0.0536	0.3634	-0.0092	0.0669	0.0001	0.0004
	W7	0.5494	0.3634	0.0671	0.0669	0.0004	0.0004
	W8	0.5494	-0.3634	0.0671	-0.0669	0.0004	0.0004
	W9	0.3600	0.2726	0.0414	0.0502	0.0002	0.0003
	W10	0.4641	0.2726	0.0593	0.0502	0.0003	0.0003
	W11	0.3600	-0.2726	0.0414	-0.0502	0.0002	0.0003
	W12	0.4641	-0.2726	0.0593	-0.0502	0.0003	0.0003
	E1	2.4046	-0.0000	0.3051	-0.0000	0.0018	0.0000
	E2	2.4597	0.0000	0.3148	0.0000	0.0018	0.0000
	E3	0.0481	0.9433	0.0085	0.1783	0.0000	0.0010
	E4	-0.0481	0.9433	-0.0085	0.1783	0.0000	0.0010
	ND1	0.0829	-0.0000	0.0082	-0.0000	0.0000	0.0000
	ND2	0.0000	0.0310	0.0000	0.0049	0.0000	0.0000
	NL1	0.0238	-0.0000	0.0011	-0.0000	0.0000	0.0000
	NL2	0.0000	0.0085	0.0000	0.0007	0.0000	0.0000
	NR1	0.0142	-0.0000	0.0042	-0.0000	0.0000	0.0000
	NR2	0.0000	0.0062	0.0000	0.0023	0.0000	0.0000
4th	D	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Lp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Sp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	W1	0.6430	-0.0000	0.1690	-0.0000	0.0010	0.0000
	W2	0.0000	0.3953	0.0000	0.1370	0.0000	0.0008
	W3	0.4692	-0.0000	0.1224	-0.0000	0.0008	0.0000
	W4	0.4953	0.0000	0.1312	0.0000	0.0008	0.0000
	W5	0.0444	0.2965	0.0148	0.1028	0.0001	0.0006
	W6	-0.0444	0.2965	-0.0148	0.1028	0.0001	0.0006
	W7	0.4823	0.2965	0.1268	0.1028	0.0008	0.0006
	W8	0.4823	-0.2965	0.1268	-0.1028	0.0008	0.0006
	W9	0.3186	0.2224	0.0807	0.0771	0.0005	0.0005
	W10	0.4048	0.2224	0.1095	0.0771	0.0007	0.0005
	W11	0.3186	-0.2224	0.0807	-0.0771	0.0005	0.0005
	W12	0.4048	-0.2224	0.1095	-0.0771	0.0007	0.0005
	E1	2.0995	-0.0000	0.6046	-0.0000	0.0037	0.0000
	E2	2.1450	0.0000	0.6215	0.0000	0.0038	0.0000
	E3	0.0396	0.7650	0.0147	0.2930	0.0001	0.0018
	E4	-0.0396	0.7650	-0.0147	0.2930	0.0001	0.0018
	ND1	0.0746	-0.0000	0.0183	0.0000	0.0001	0.0000
	ND2	0.0000	0.0261	0.0000	0.0085	0.0000	0.0001
	NL1	0.0226	-0.0000	0.0050	0.0000	0.0000	0.0000

Story	LdC	Displacement		Story Drift		Drift Ratio	
	NL2	0.0000	0.0077	0.0000	0.0022	0.0000	0.0000
	NR1	0.0100	-0.0000	0.0038	-0.0000	0.0000	0.0000
	NR2	0.0000	0.0039	0.0000	0.0019	0.0000	0.0000
3rd	D	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Lp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Sp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	W1	0.4740	-0.0000	0.2516	-0.0000	0.0016	0.0000
	W2	0.0000	0.2583	0.0000	0.1314	0.0000	0.0008
	W3	0.3468	-0.0000	0.1843	-0.0000	0.0011	0.0000
	W4	0.3642	0.0000	0.1930	0.0000	0.0012	0.0000
	W5	0.0296	0.1937	0.0147	0.0985	0.0001	0.0006
	W6	-0.0296	0.1937	-0.0147	0.0985	0.0001	0.0006
	W7	0.3555	0.1937	0.1887	0.0985	0.0012	0.0006
	W8	0.3555	-0.1937	0.1887	-0.0985	0.0012	0.0006
	W9	0.2379	0.1453	0.1272	0.0739	0.0008	0.0005
	W10	0.2953	0.1453	0.1558	0.0739	0.0010	0.0005
	W11	0.2379	-0.1453	0.1272	-0.0739	0.0008	0.0005
	W12	0.2953	-0.1453	0.1558	-0.0739	0.0010	0.0005
	E1	1.4949	-0.0000	0.8369	-0.0000	0.0052	0.0000
	E2	1.5235	0.0000	0.8523	0.0000	0.0053	0.0000
	E3	0.0249	0.4720	0.0134	0.2595	0.0001	0.0016
	E4	-0.0249	0.4720	-0.0134	0.2595	0.0001	0.0016
	ND1	0.0563	-0.0000	0.0294	-0.0000	0.0002	0.0000
	ND2	0.0000	0.0176	0.0000	0.0088	0.0000	0.0001
	NL1	0.0177	-0.0000	0.0091	-0.0000	0.0001	0.0000
	NL2	0.0000	0.0055	0.0000	0.0026	0.0000	0.0000
	NR1	0.0062	-0.0000	0.0037	-0.0000	0.0000	0.0000
	NR2	0.0000	0.0020	0.0000	0.0012	0.0000	0.0000
2nd	D	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Lp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Sp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W1	0.2224	-0.0000	0.2224	-0.0000	0.0014	0.0000
	W2	0.0000	0.1270	0.0000	0.1270	0.0000	0.0008
	W3	0.1625	-0.0000	0.1625	-0.0000	0.0010	0.0000
	W4	0.1712	0.0000	0.1712	0.0000	0.0011	0.0000
	W5	0.0149	0.0952	0.0149	0.0952	0.0001	0.0006
	W6	-0.0149	0.0952	-0.0149	0.0952	0.0001	0.0006
	W7	0.1668	0.0952	0.1668	0.0952	0.0010	0.0006
	W8	0.1668	-0.0952	0.1668	-0.0952	0.0010	0.0006
	W9	0.1107	0.0714	0.1107	0.0714	0.0007	0.0004
	W10	0.1395	0.0714	0.1395	0.0714	0.0009	0.0004
	W11	0.1107	-0.0714	0.1107	-0.0714	0.0007	0.0004
	W12	0.1395	-0.0714	0.1395	-0.0714	0.0009	0.0004

Story	LdC	Displacement		Story Drift		Drift Ratio	
	E1	0.6580	-0.0000	0.6580	-0.0000	0.0041	0.0000
	E2	0.6712	0.0000	0.6712	0.0000	0.0041	0.0000
	E3	0.0115	0.2125	0.0115	0.2125	0.0001	0.0013
	E4	-0.0115	0.2125	-0.0115	0.2125	0.0001	0.0013
	ND1	0.0269	-0.0000	0.0269	-0.0000	0.0002	0.0000
	ND2	0.0000	0.0089	0.0000	0.0089	0.0000	0.0001
	NL1	0.0086	-0.0000	0.0086	-0.0000	0.0001	0.0000
	NL2	0.0000	0.0029	0.0000	0.0029	0.0000	0.0000
	NR1	0.0025	-0.0000	0.0025	-0.0000	0.0000	0.0000
	NR2	0.0000	0.0008	0.0000	0.0008	0.0000	0.0000

Location (ft): (0.000, 60.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Lp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Sp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	W1	0.7325	0.0000	0.0895	0.0000	0.0005	0.0000
	W2	-0.0000	0.4846	-0.0000	0.0893	0.0000	0.0005
	W3	0.5494	0.0276	0.0671	0.0048	0.0004	0.0000
	W4	0.5494	-0.0276	0.0671	-0.0048	0.0004	0.0000
	W5	-0.0000	0.2696	-0.0000	0.0508	0.0000	0.0003
	W6	0.0000	0.4573	0.0000	0.0831	0.0000	0.0005
	W7	0.5494	0.3634	0.0671	0.0669	0.0004	0.0004
	W8	0.5494	-0.3634	0.0671	-0.0669	0.0004	0.0004
	W9	0.4120	0.3637	0.0503	0.0659	0.0003	0.0004
	W10	0.4120	0.1815	0.0503	0.0346	0.0003	0.0002
	W11	0.4120	-0.1815	0.0503	-0.0346	0.0003	0.0002
	W12	0.4120	-0.3637	0.0503	-0.0659	0.0003	0.0004
	E1	2.4324	0.0487	0.3099	0.0085	0.0018	0.0000
	E2	2.4324	-0.0479	0.3099	-0.0085	0.0018	0.0000
	E3	-0.0000	0.8591	-0.0000	0.1634	0.0000	0.0009
	E4	0.0000	1.0274	0.0000	0.1931	0.0000	0.0011
	ND1	0.0829	0.0000	0.0082	0.0000	0.0000	0.0000
	ND2	-0.0000	0.0310	-0.0000	0.0049	0.0000	0.0000
	NL1	0.0238	0.0000	0.0011	0.0000	0.0000	0.0000
	NL2	-0.0000	0.0085	-0.0000	0.0007	0.0000	0.0000
	NR1	0.0142	0.0000	0.0042	0.0000	0.0000	0.0000
	NR2	-0.0000	0.0062	-0.0000	0.0023	0.0000	0.0000
4th	D	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Lp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Sp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000

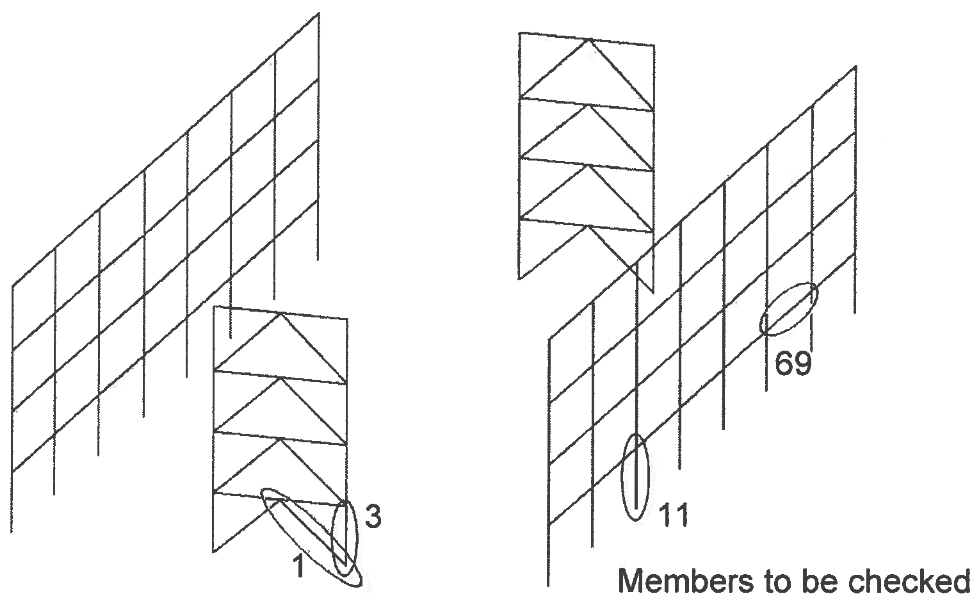
Drift

Story	LdC	Displacement		Story Drift		Drift Ratio	
	W1	0.6430	0.0000	0.1690	0.0000	0.0010	0.0000
	W2	-0.0000	0.3953	-0.0000	0.1370	0.0000	0.0008
	W3	0.4823	0.0229	0.1268	0.0077	0.0008	0.0000
	W4	0.4823	-0.0229	0.1268	-0.0077	0.0008	0.0000
	W5	-0.0000	0.2188	-0.0000	0.0768	0.0000	0.0005
	W6	0.0000	0.3742	0.0000	0.1287	0.0000	0.0008
	W7	0.4823	0.2965	0.1268	0.1028	0.0008	0.0006
	W8	0.4823	-0.2965	0.1268	-0.1028	0.0008	0.0006
	W9	0.3617	0.2978	0.0951	0.1023	0.0006	0.0006
	W10	0.3617	0.1469	0.0951	0.0518	0.0006	0.0003
	W11	0.3617	-0.1469	0.0951	-0.0518	0.0006	0.0003
	W12	0.3617	-0.2978	0.0951	-0.1023	0.0006	0.0006
	E1	2.1225	0.0402	0.6130	0.0147	0.0038	0.0001
	E2	2.1225	-0.0394	0.6130	-0.0147	0.0038	0.0001
	E3	-0.0000	0.6956	-0.0000	0.2673	0.0000	0.0016
	E4	0.0000	0.8343	0.0000	0.3187	0.0000	0.0020
	ND1	0.0747	0.0000	0.0183	-0.0000	0.0001	0.0000
	ND2	-0.0000	0.0261	-0.0000	0.0085	0.0000	0.0001
	NL1	0.0226	0.0000	0.0050	-0.0000	0.0000	0.0000
	NL2	-0.0000	0.0077	-0.0000	0.0022	0.0000	0.0000
	NR1	0.0100	0.0000	0.0038	0.0000	0.0000	0.0000
	NR2	-0.0000	0.0039	-0.0000	0.0019	0.0000	0.0000
3rd	D	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Lp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Sp	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	W1	0.4740	0.0000	0.2516	0.0000	0.0016	0.0000
	W2	-0.0000	0.2583	-0.0000	0.1314	0.0000	0.0008
	W3	0.3555	0.0152	0.1887	0.0076	0.0012	0.0000
	W4	0.3555	-0.0152	0.1887	-0.0076	0.0012	0.0000
	W5	-0.0000	0.1420	-0.0000	0.0728	0.0000	0.0004
	W6	0.0000	0.2455	0.0000	0.1243	0.0000	0.0008
	W7	0.3555	0.1937	0.1887	0.0985	0.0012	0.0006
	W8	0.3555	-0.1937	0.1887	-0.0985	0.0012	0.0006
	W9	0.2666	0.1955	0.1415	0.0989	0.0009	0.0006
	W10	0.2666	0.0951	0.1415	0.0489	0.0009	0.0003
	W11	0.2666	-0.0951	0.1415	-0.0489	0.0009	0.0003
	W12	0.2666	-0.1955	0.1415	-0.0989	0.0009	0.0006
	E1	1.5094	0.0254	0.8446	0.0135	0.0052	0.0001
	E2	1.5094	-0.0246	0.8446	-0.0135	0.0052	0.0001
	E3	-0.0000	0.4284	-0.0000	0.2360	0.0000	0.0015
	E4	0.0000	0.5156	0.0000	0.2830	0.0000	0.0017
	ND1	0.0563	0.0000	0.0294	0.0000	0.0002	0.0000
	ND2	-0.0000	0.0176	-0.0000	0.0088	0.0000	0.0001
	NL1	0.0177	0.0000	0.0091	0.0000	0.0001	0.0000

Drift

Story	LdC	Displacement		Story Drift		Drift Ratio	
	NL2	-0.0000	0.0055	-0.0000	0.0026	0.0000	0.0000
	NR1	0.0062	0.0000	0.0037	0.0000	0.0000	0.0000
	NR2	-0.0000	0.0020	-0.0000	0.0012	0.0000	0.0000
2nd	D	-0.0000	0.0000	-0.0000	0.0000	0.0000	0.0000
	Lp	-0.0000	-0.0000	-0.0000	-0.0000	0.0000	0.0000
	Sp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	W1	0.2224	0.0000	0.2224	0.0000	0.0014	0.0000
	W2	-0.0000	0.1270	-0.0000	0.1270	0.0000	0.0008
	W3	0.1668	0.0076	0.1668	0.0076	0.0010	0.0000
	W4	0.1668	-0.0076	0.1668	-0.0076	0.0010	0.0000
	W5	-0.0000	0.0692	-0.0000	0.0692	0.0000	0.0004
	W6	0.0000	0.1212	0.0000	0.1212	0.0000	0.0007
	W7	0.1668	0.0952	0.1668	0.0952	0.0010	0.0006
	W8	0.1668	-0.0952	0.1668	-0.0952	0.0010	0.0006
	W9	0.1251	0.0966	0.1251	0.0966	0.0008	0.0006
	W10	0.1251	0.0462	0.1251	0.0462	0.0008	0.0003
	W11	0.1251	-0.0462	0.1251	-0.0462	0.0008	0.0003
	W12	0.1251	-0.0966	0.1251	-0.0966	0.0008	0.0006
	E1	0.6648	0.0119	0.6648	0.0119	0.0041	0.0001
	E2	0.6648	-0.0112	0.6648	-0.0112	0.0041	0.0001
	E3	-0.0000	0.1924	-0.0000	0.1924	0.0000	0.0012
	E4	0.0000	0.2326	0.0000	0.2326	0.0000	0.0014
	ND1	0.0269	0.0000	0.0269	0.0000	0.0002	0.0000
	ND2	-0.0000	0.0089	-0.0000	0.0089	0.0000	0.0001
	NL1	0.0086	0.0000	0.0086	0.0000	0.0001	0.0000
	NL2	-0.0000	0.0029	-0.0000	0.0029	0.0000	0.0000
	NR1	0.0025	0.0000	0.0025	0.0000	0.0000	0.0000
	NR2	-0.0000	0.0008	-0.0000	0.0008	0.0000	0.0000

Building Example 2
4-Story Commercial



There's always a solution in steel

Building Example 2
4-Story Commercial

- Column in Braced Frame, #3, W12x53
  - Controlling load combination

$$D + 0.75L - 0.525E4$$

  - Analysis results by load case

$$D = 168 \text{ kips}$$
$$L = 52.7 \text{ kips}$$
$$E4 = -146 \text{ kips}$$

There's always a solution in steel

Building Example 2
4-Story Commercial

- Column in Braced Frame, #3, W12x53
  - Sway amplification

$$B_2 = 1.07$$

  - Member amplification
    - No amplification since no moments
  - Member force

$$P_a = P_D + 0.75P_L - B_2(0.525P_{E4})$$
$$P_a = 168 + 0.75(52.7) - 1.07(0.525)(-146) = 290 \text{ kips}$$

There's always a solution in steel

Building Example 2
4-Story Commercial

- Column in Braced Frame, #3, W12x53
  - Determine member strength

$$K = 1$$

$$L = 13.5 \text{ ft}$$

$$\frac{P_n}{\Omega} = 431 \text{ kips} > 290 \text{ kips}$$

  - W12x53 is adequate

Member Code Check

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

02/19/09 08:23:44
Steel Code: AISC360-05 ASD

COLUMN INFORMATION:

Story Level = 2nd
Fy (ksi) = 50.00
Column Size = W12X53

Frame Number = 3
Column Number = 3

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	13.50	13.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.000 D + 0.700 E4

Shear	Top	Vmajor (kip)	-0.10
		Vminor (kip)	-0.00
Shear	Bot.	Vmajor (kip)	-0.10
		Vminor (kip)	-0.00

SHEAR CHECK:

Vax (kip) =	-0.10	Vnx/1.50 (kip) =	83.49	Vax/(Vnx/1.50) =	0.001
Vay (kip) =	-0.00	Vny/1.67 (kip) =	206.59	Vay/(Vny/1.67) =	0.000

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.000 D + 0.750 Lp - 0.525 E4

Axial		Load (kip)	289.00
Moment	Top	Mmajor (kip-ft)	-1.04
		Mminor (kip-ft)	-0.00
Moment	Bot.	Mmajor (kip-ft)	0.00
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip) =	289.00	Pn/1.67 (kip) =	341.72
Max (kip-ft) =	-1.04	Mnx/1.67 (kip-ft) =	194.36
May (kip-ft) =	-0.00	Mny/1.67 (kip-ft) =	72.60
Cmx =	0.60	Cmy =	0.60
B1x =	1.00	B1y =	1.06
B2x =	1.07	B2y =	1.00
Baxial =	1.06	@angle (degrees) =	90.00
Cbx =	1.67		

INTERACTION EQUATION:

Pa/(Pn/1.67)) = 0.846

Eq H1-1a: 0.846 + 8/9(0.005 + 0.000) = 0.850

Member Forces

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

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STEEL COLUMN INFORMATION:

Column Number: 3 **Frame Number: 3**

Level	Top: 2nd	Column Line (0.00,45.00)
	Bot: Base	
Fy (ksi) = 50.00		Column Size = W12X53
Elastic Modulus (ksi) = 29000.00		
Orientation (deg) = 90.00		Length (ft) = 13.50

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Pin
	Minor Axis:	Fix	Pin
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	0.00	0.00
	Minor:	0.00	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)
Member Force Output:		At Face of Joint	
P-Delta:	No		
Ground Level:	Base		

LOAD CASES:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW
E1	E	EQ_IBC06_X_+E_F
E2	E	EQ_IBC06_X_-E_F
E3	E	EQ_IBC06_Y_+E_F
E4	E	EQ_IBC06_Y_-E_F
ND1	N	NL_AISC360_DL_X
ND2	N	NL_AISC360_DL_Y
NL1	N	NL_AISC360_LL_X
NL2	N	NL_AISC360_LL_Y

NR1	N	NL_AISC360_Rf_X
NR2	N	NL_AISC360_Rf_Y

MEMBER FORCES:

LdC	@	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
D	T	168.49	0.00	0.00	-0.00	-0.00	0.00
	B	168.49	0.00	0.00	-0.00	-0.00	0.00
Lp	T	52.67	0.00	0.00	-0.00	-0.00	0.00
	B	52.67	0.00	0.00	-0.00	-0.00	0.00
Sp	T	14.66	0.00	0.00	-0.00	-0.00	-0.00
	B	14.66	0.00	0.00	-0.00	-0.00	-0.00
W1	T	-0.00	0.00	-0.58	-0.00	0.04	-0.00
	B	-0.00	0.00	0.00	-0.00	0.04	-0.00
W2	T	-66.05	0.00	0.00	-0.00	-0.00	0.00
	B	-66.05	0.00	0.00	-0.00	-0.00	0.00
W3	T	-3.73	-0.01	-0.43	0.00	0.03	-0.00
	B	-3.73	-0.00	0.00	0.00	0.03	-0.00
W4	T	3.73	0.01	-0.43	-0.00	0.03	0.00
	B	3.73	0.00	0.00	-0.00	0.03	0.00
W5	T	-36.89	0.05	0.00	-0.00	-0.00	0.00
	B	-36.89	0.00	0.00	-0.00	-0.00	0.00
W6	T	-62.18	-0.05	-0.00	0.00	0.00	-0.00
	B	-62.18	-0.00	0.00	0.00	0.00	-0.00
W7	T	-49.54	0.00	-0.43	-0.00	0.03	0.00
	B	-49.54	0.00	0.00	-0.00	0.03	0.00
W8	T	49.54	-0.00	-0.43	0.00	0.03	-0.00
	B	49.54	-0.00	0.00	0.00	0.03	-0.00
W9	T	-49.43	-0.05	-0.33	0.00	0.02	-0.00
	B	-49.43	-0.00	0.00	0.00	0.02	-0.00
W10	T	-24.87	0.05	-0.32	-0.00	0.02	0.00
	B	-24.87	0.00	0.00	-0.00	0.02	0.00
W11	T	24.87	-0.05	-0.33	0.00	0.02	-0.00
	B	24.87	-0.00	0.00	0.00	0.02	-0.00
W12	T	49.43	0.05	-0.32	-0.00	0.02	0.00
	B	49.43	0.00	0.00	-0.00	0.02	0.00
E1	T	-6.80	0.05	-2.79	-0.00	0.21	-0.00
	B	-6.80	0.00	0.00	-0.00	0.21	-0.00
E2	T	6.79	-0.09	-2.79	0.01	0.21	0.00
	B	6.79	-0.00	0.00	0.01	0.21	0.00
E3	T	-122.02	1.61	-0.00	-0.12	0.00	0.00
	B	-122.02	0.00	0.00	-0.12	0.00	0.00
E4	T	-145.71	1.85	0.00	-0.14	-0.00	-0.00
	B	-145.71	0.00	0.00	-0.14	-0.00	-0.00
ND1	T	-0.00	-0.00	-0.06	0.00	0.00	-0.00
	B	-0.00	-0.00	0.00	0.00	0.00	-0.00

Member Forces

LdC	@	P	Mmajor	Mminor	Vmajor	Vminor	Tors
ND2	T	-4.17	-0.01	0.00	0.00	-0.00	0.00
	B	-4.17	-0.00	0.00	0.00	-0.00	0.00
NL1	T	-0.00	-0.00	-0.02	0.00	0.00	-0.00
	B	-0.00	-0.00	0.00	0.00	0.00	-0.00
NL2	T	-1.12	-0.01	0.00	0.00	-0.00	0.00
	B	-1.12	-0.00	0.00	0.00	-0.00	0.00
NR1	T	-0.00	0.00	-0.01	-0.00	0.00	-0.00
	B	-0.00	0.00	0.00	-0.00	0.00	-0.00
NR2	T	-0.88	0.01	-0.00	-0.00	0.00	0.00
	B	-0.88	0.00	0.00	-0.00	0.00	0.00

Building Example 2
4-Story Commercial

- Brace in Braced Frame, #1, HSS 6x6x1/2
  - Controlling load combination

$$D-0.7E4$$

  - Analysis results by load case

$$D=13.0 \text{ kips}$$

$$E4=-121 \text{ kips}$$

There's always a solution in steel

Building Example 2
4-Story Commercial

- Brace in Braced Frame, #1, HSS 6x6x1/2
  - Sway amplification

$$B_2=1.05$$

  - Member amplification
    - No amplification since no moments
  - Member force

$$P_a=P_D-0.7P_{E4}$$

$$P_a=13.0-1.05(0.7)(-121)=102 \text{ kips}$$

There's always a solution in steel

Building Example 2
4-Story Commercial

- Brace in Braced Frame, #1, HSS 6x6x1/2
  - Determine member strength

$$K = 1$$

$$L = 20.18 \text{ ft}$$

$$\frac{P_n}{\Omega} = 121 \text{ kips} > 102 \text{ kips}$$

  - HSS 6x6x1/2 is adequate

Member Code Check

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

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Steel Code: AISC360-05 ASD

BRACE INFORMATION:

Story Level = 2nd   Frame Number = 3   Brace Number = 1
Fy (ksi) = 46.00
Brace Size = HSS6X6X1/2

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	20.18	20.18
K	1.00	1.00

CONTROLLING BRACE FORCES - SHEAR

Load Combination: 1.000 D + 1.000 ND1

Shear	Top	Vmajor (kip)	-0.00
		Vminor (kip)	0.00
Shear	Bot.	Vmajor (kip)	-0.00
		Vminor (kip)	0.00

SHEAR CHECK:

Vax (kip) =	-0.00	Vnx/1.67 (kip) =	92.22	Vax/(Vnx/1.67) =	0.000
Vay (kip) =	0.00	Vny/1.67 (kip) =	92.22	Vay/(Vny/1.67) =	0.000

CONTROLLING BRACE FORCES - FLEXURE

Load Combination: 1.000 D - 0.700 E4

Axial		Load (kip)	101.95
Moment	Top	Mmajor (kip-ft)	0.00
		Mminor (kip-ft)	0.00
Moment	Bot.	Mmajor (kip-ft)	0.00
		Mminor (kip-ft)	0.00

CALCULATED PARAMETERS:

Pa (kip) =	101.95	Pn/1.67 (kip) =	121.09
Max (kip-ft) =	0.00	Mnx/1.67 (kip-ft) =	45.45
May (kip-ft) =	0.00	Mny/1.67 (kip-ft) =	45.45
Cmx =	1.00	Cmy =	1.00
B1x =	2.95	B1y =	2.95
B2x =	1.05	B2y =	1.00
Baxial =	1.05	@angle (degrees) =	-90.00
Cbx =	1.00		

INTERACTION EQUATION:

Pa/(Pn/1.67)) = 0.842
Eq H1-1a: 0.842 + 8/9(0.000 + 0.000) = 0.842

Member Forces

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

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STEEL BRACE INFORMATION:

Brace Number: 1 **Frame Number: 3**

Story Top: 2nd I-End (ft): (0.00,60.00)
Bot: Base J-End (ft): (0.00,45.00)
Fy (ksi) = 46.00 Brace Size = HSS6X6X1/2
Length (ft) = 20.18
Elastic Modulus (ksi) = 29000.00

INPUT PARAMETERS:

		Top	**Bottom**
Fixity	Major Axis:	Pin	Pin
	Minor Axis:	Pin	Pin
	Torsion:	Pin	Pin
Member Force Output:		At Centerline of Joint	

P-Delta: No
Ground Level: Base

LOAD CASES:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW
E1	E	EQ_IBC06_X_+E_F
E2	E	EQ_IBC06_X_-E_F
E3	E	EQ_IBC06_Y_+E_F
E4	E	EQ_IBC06_Y_-E_F
ND1	N	NL_AISC360_DL_X
ND2	N	NL_AISC360_DL_Y
NL1	N	NL_AISC360_LL_X
NL2	N	NL_AISC360_LL_Y
NR1	N	NL_AISC360_Rf_X
NR2	N	NL_AISC360_Rf_Y

Member Forces

MEMBER FORCES:

LdC	@	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
D	T	12.97	0.00	-0.00	-0.00	-0.00	-0.00
	B	12.97	-0.00	0.00	-0.00	-0.00	-0.00
Lp	T	6.16	0.00	-0.00	-0.00	-0.00	-0.00
	B	6.16	-0.00	0.00	-0.00	-0.00	-0.00
Sp	T	0.09	0.00	-0.00	-0.00	0.00	-0.00
	B	0.09	-0.00	0.00	-0.00	0.00	-0.00
W1	T	-0.00	0.00	-0.00	-0.00	0.00	-0.00
	B	-0.00	-0.00	0.00	-0.00	0.00	-0.00
W2	T	-65.76	0.00	-0.00	-0.00	0.00	-0.00
	B	-65.76	-0.00	0.00	-0.00	0.00	-0.00
W3	T	-3.95	0.00	-0.00	-0.00	0.00	-0.00
	B	-3.95	-0.00	0.00	-0.00	0.00	-0.00
W4	T	3.95	0.00	-0.00	0.00	0.00	-0.00
	B	3.95	-0.00	0.00	0.00	0.00	-0.00
W5	T	-35.85	0.00	-0.00	-0.00	0.00	-0.00
	B	-35.85	-0.00	0.00	-0.00	0.00	-0.00
W6	T	-62.79	0.00	-0.00	-0.00	-0.00	-0.00
	B	-62.79	-0.00	0.00	-0.00	-0.00	-0.00
W7	T	-49.32	0.00	-0.00	-0.00	0.00	-0.00
	B	-49.32	-0.00	0.00	-0.00	0.00	-0.00
W8	T	49.32	0.00	-0.00	0.00	0.00	-0.00
	B	49.32	-0.00	0.00	0.00	0.00	-0.00
W9	T	-50.06	0.00	-0.00	-0.00	0.00	-0.00
	B	-50.06	-0.00	0.00	-0.00	0.00	-0.00
W10	T	-23.92	0.00	-0.00	-0.00	0.00	-0.00
	B	-23.92	-0.00	0.00	-0.00	0.00	-0.00
W11	T	23.92	0.00	-0.00	0.00	0.00	-0.00
	B	23.92	-0.00	0.00	0.00	0.00	-0.00
W12	T	50.06	0.00	-0.00	0.00	0.00	-0.00
	B	50.06	-0.00	0.00	0.00	0.00	-0.00
E1	T	-6.17	0.00	-0.00	-0.00	0.00	-0.00
	B	-6.17	-0.00	0.00	-0.00	0.00	-0.00
E2	T	5.78	0.00	-0.00	0.00	0.00	-0.00
	B	5.78	-0.00	0.00	0.00	0.00	-0.00
E3	T	-99.65	0.00	-0.00	-0.00	0.00	-0.00
	B	-99.65	-0.00	0.00	-0.00	0.00	-0.00
E4	T	-120.49	0.00	-0.00	-0.00	-0.00	-0.00
	B	-120.49	-0.00	0.00	-0.00	-0.00	-0.00
ND1	T	-0.02	0.00	-0.00	-0.00	0.00	-0.00
	B	-0.02	-0.00	0.00	-0.00	0.00	-0.00
ND2	T	-4.60	0.00	-0.00	-0.00	0.00	-0.00
	B	-4.60	-0.00	0.00	-0.00	0.00	-0.00
NL1	T	-0.01	0.00	-0.00	-0.00	0.00	-0.00

Member Forces

LdC	@	P	Mmajor	Mminor	Vmajor	Vminor	Tors
	B	-0.01	-0.00	0.00	-0.00	0.00	-0.00
NL2	T	-1.49	0.00	-0.00	-0.00	0.00	-0.00
	B	-1.49	-0.00	0.00	-0.00	0.00	-0.00
NR1	T	-0.00	0.00	-0.00	-0.00	0.00	-0.00
	B	-0.00	-0.00	0.00	-0.00	0.00	-0.00
NR2	T	-0.40	0.00	-0.00	-0.00	0.00	-0.00
	B	-0.40	-0.00	0.00	-0.00	0.00	-0.00

Building Example 2
4-Story Commercial

- Column in Moment Frame, #11, W14x99
  - Controlling load combination

$$D - 0.7E2$$

  - Analysis results by load case

D	$P = 226$ kips	$M_T = 3.75$ ft-kips	$M_B = -1.86$ ft-kips
$E2$	$P = 0$ kips	$M_T = -105$ ft-kips	$M_B = 216$ ft-kips

There's always a solution in steel

Building Example 2
4-Story Commercial

- Column in Moment Frame, #11, W14x99
  - Sway amplification

$$B_2 = 1.19$$

  - Member amplification

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

$$C_m = 0.6 - 0.4\left(\frac{1.86}{3.75}\right) = 0.40$$

$$\alpha P_r = 1.6(226 + 0) = 362 \text{ kips}$$

$$P_{e1} = \frac{\pi^2 (29{,}000)(1{,}110)}{(13.5(12))^2} = 12{,}100 \text{ kips}$$

There's always a solution in steel

Building Example 2
4-Story Commercial

- Column in Moment Frame, #11, W14x99
  - Member amplification

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} = \frac{0.4}{1 - \frac{362}{12{,}100}} = 1.03(0.4) = 0.41 < 1.0$$

  - Member force

$$P_a = P_D - B_2\left(0.7 P_{E2}\right)$$

$$P_a = 226 - 1.19(0.7)(0) = 226 \text{ kips}$$

Building Example 2
4-Story Commercial

- Column in Moment Frame, #11, W14x99
  - Member moment

$$M_a = B_1 M_D - B_2\left(0.7 M_{E2}\right)$$

$$M_a = 1.0(-1.86) - 1.19(0.7)(216) = -182 \text{ ft-kips}$$

  - Determine member strength

$$K = 1.0,\ L = 13.5 \text{ ft},\ \frac{P_n}{\Omega} = 759 \text{ kips}$$

Building Example 2
4-Story Commercial

- Column in Moment Frame, #11, W14x99
  - Determine member strength

$$L_p = 13.5 \text{ ft}, \ \frac{M_n}{\Omega} = 430 \text{ ft-kips}$$

  - Interaction

$$\frac{226}{759} + \frac{8}{9}\left(\frac{182}{430}\right) = 0.30 + 0.38 = 0.68 < 1.0$$

  - W14x99 is adequate

There's always a solution in steel

Member Code Check

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

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Steel Code: AISC360-05 ASD

COLUMN INFORMATION:

Story Level = 2nd   Frame Number = 2   Column Number = 11
Fy (ksi) = 50.00
Column Size = W14X99

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	13.50	13.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.000 D - 0.700 E2

Shear	Top	Vmajor (kip)	-18.42
		Vminor (kip)	0.03
Shear	Bot.	Vmajor (kip)	-18.42
		Vminor (kip)	0.03

SHEAR CHECK:

Vax (kip) =	-18.42	Vnx/1.50 (kip) =	137.74	Vax/(Vnx/1.50) =	0.134
Vay (kip) =	0.03	Vny/1.67 (kip) =	409.15	Vay/(Vny/1.67) =	0.000

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.000 D - 0.700 E2

Axial		Load (kip)	225.84
Moment	Top	Mmajor (kip-ft)	91.49
		Mminor (kip-ft)	-0.06
Moment	Bot.	Mmajor (kip-ft)	-182.88
		Mminor (kip-ft)	0.33

CALCULATED PARAMETERS:

Pa (kip) =	225.84	Pn/1.67 (kip) =	758.26
Max (kip-ft) =	-182.88	Mnx/1.67 (kip-ft) =	429.45
May (kip-ft) =	0.33	Mny/1.67 (kip-ft) =	207.02
Cmx =	0.40	Cmy =	0.52
B1x =	1.00	B1y =	1.00
B2x =	1.19	B2y =	1.00
Baxial =	1.19	@angle (degrees) =	0.00
Cbx =	2.17		

INTERACTION EQUATION:

Pa/(Pn/1.67)) = 0.298
Eq H1-1a: 0.298 + 8/9(0.426 + 0.002) = 0.678

Member Forces

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

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STEEL COLUMN INFORMATION:

Column Number: 11 **Frame Number: 2**

Level Top: 2nd Column Line (60.00,0.00)
Bot: Base

Fy (ksi) = 50.00 Column Size = W14X99

Elastic Modulus (ksi) = 29000.00

Orientation (deg) = 0.00 Length (ft) = 13.50

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	11.80	0.00
	Minor:	0.00	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)
Member Force Output:		At Face of Joint	
P-Delta:	No		
Ground Level:	Base		

LOAD CASES:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW
E1	E	EQ_IBC06_X_+E_F
E2	E	EQ_IBC06_X_-E_F
E3	E	EQ_IBC06_Y_+E_F
E4	E	EQ_IBC06_Y_-E_F
ND1	N	NL_AISC360_DL_X
ND2	N	NL_AISC360_DL_Y
NL1	N	NL_AISC360_LL_X
NL2	N	NL_AISC360_LL_Y

NR1	N	NL_AISC360_Rf_X
NR2	N	NL_AISC360_Rf_Y

MEMBER FORCES:

LdC	@	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
D	T	225.88	3.75	0.00	-0.45	-0.00	0.00
	B	225.88	-1.86	-0.00	-0.45	-0.00	0.00
Lp	T	68.54	1.61	0.00	-0.19	-0.00	0.00
	B	68.54	-0.80	-0.00	-0.19	-0.00	0.00
Sp	T	18.47	-0.02	0.00	0.00	-0.00	-0.00
	B	18.47	0.01	-0.00	0.00	-0.00	-0.00
W1	T	0.02	-37.17	0.00	8.80	-0.00	-0.00
	B	0.02	72.92	0.00	8.80	-0.00	-0.00
W2	T	0.00	-0.00	-3.42	0.00	1.21	0.00
	B	0.00	0.00	12.90	0.00	1.21	0.00
W3	T	0.01	-27.08	-0.09	6.42	0.03	-0.00
	B	0.01	53.23	0.34	6.42	0.03	-0.00
W4	T	0.01	-28.67	0.09	6.78	-0.03	0.00
	B	0.01	56.15	-0.34	6.78	-0.03	0.00
W5	T	0.00	-2.71	-2.24	0.61	0.80	0.00
	B	0.00	4.98	8.53	0.61	0.80	0.00
W6	T	-0.00	2.71	-2.88	-0.61	1.02	-0.00
	B	-0.00	-4.98	10.82	-0.61	1.02	-0.00
W7	T	0.01	-27.87	-2.56	6.60	0.91	0.00
	B	0.01	54.69	9.68	6.60	0.91	0.00
W8	T	0.01	-27.87	2.56	6.60	-0.91	-0.00
	B	0.01	54.69	-9.68	6.60	-0.91	-0.00
W9	T	0.01	-18.28	-2.23	4.35	0.79	-0.00
	B	0.01	36.18	8.37	4.35	0.79	-0.00
W10	T	0.01	-23.53	-1.61	5.54	0.57	0.00
	B	0.01	45.85	6.15	5.54	0.57	0.00
W11	T	0.01	-18.28	1.61	4.35	-0.57	-0.00
	B	0.01	36.18	-6.15	4.35	-0.57	-0.00
W12	T	0.01	-23.53	2.23	5.54	-0.79	0.00
	B	0.01	45.85	-8.37	5.54	-0.79	0.00
E1	T	0.05	-102.69	-0.12	25.15	0.05	-0.00
	B	0.05	212.12	0.51	25.15	0.05	-0.00
E2	T	0.06	-104.91	0.09	25.67	-0.04	0.00
	B	0.06	216.45	-0.47	25.67	-0.04	0.00
E3	T	0.00	-1.94	-3.85	0.46	1.76	0.00
	B	0.00	3.78	19.92	0.46	1.76	0.00
E4	T	-0.00	1.94	-4.21	-0.46	1.91	-0.00
	B	-0.00	-3.78	21.62	-0.46	1.91	-0.00
ND1	T	0.00	-4.56	-0.00	1.07	0.00	-0.00
	B	0.00	8.85	0.00	1.07	0.00	-0.00

Member Forces

LdC	@	P	Mmajor	Mminor	Vmajor	Vminor	Tors
ND2	T	0.00	-0.00	-0.25	0.00	0.09	0.00
	B	0.00	0.00	0.91	0.00	0.09	0.00
NL1	T	0.00	-1.49	-0.00	0.35	0.00	-0.00
	B	0.00	2.85	0.00	0.35	0.00	-0.00
NL2	T	0.00	-0.00	-0.09	0.00	0.03	0.00
	B	0.00	0.00	0.30	0.00	0.03	0.00
NR1	T	0.00	-0.36	-0.00	0.09	0.00	-0.00
	B	0.00	0.80	0.00	0.09	0.00	-0.00
NR2	T	0.00	-0.00	-0.01	0.00	0.01	0.00
	B	0.00	0.00	0.07	0.00	0.01	0.00

Building Example 2
4-Story Commercial

- Beam in Moment Frame, #69, W24x55
  - Controlling load combination

$$D + 0.75L + 0.525E2$$

  - Analysis results by load case

D	$M_T = -136$ ft-kips	$M_B = -139$ ft-kips
L	$M_T = -82.7$ ft-kips	$M_B = -83.8$ ft-kips
$E2$	$M_T = 133$ ft-kips	$M_B = -133$ ft-kips

Building Example 2
4-Story Commercial

- Beam in Moment Frame, #69, W24x55
  - Sway amplification

$$B_2 = 1.26$$

  - Member amplification
    - No amplification since no axial load
  - Member moment

$$M_a = B_1\left(M_D + 0.75L\right) + +B_2\left(0.525M_{E2}\right)$$

$$M_a = 1.0\left(-139 + 0.75(-83.8)\right) + 1.26\left(0.525(-133)\right) = -290 \text{ ft-kips}$$

Building Example 2
4-Story Commercial

- Beam in Moment Frame, #69, W24x55
  - Member strength

$$\frac{M_n}{\Omega} = 334 \text{ ft-kips} < 290 \text{ ft-kips}$$

  - W24x55 is adequate

There's always a solution in steel

Member Code Check

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

02/19/09 08:23:44
Steel Code: AISC360-05 ASD

BEAM INFORMATION:

Story Level = 2nd   Frame Number = 2   Beam Number = 69
Fy (ksi) = 50.00
Beam Size = W24X55

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	10.00
Lu for Bending (ft)	30.00	10.00
K	1.00	1.00
Top Flange Continuously Braced	Yes	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.000 D + 0.750 Lp + 0.525 E2

Segment distance (ft) i - end	20.00
j - end	30.00

SHEAR CHECK:

Vax (kip) =	-39.21	Vnx/1.67 (kip) =	167.46	Vax/(Vnx/1.67) =	0.234
Vay (kip) =	-0.00	Vny/1.67 (kip) =	127.19	Vay/(Vny/1.67) =	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.000 D + 0.750 Lp + 0.525 E2

Segment distance (ft) i - end	20.00
j - end	30.00

CALCULATED PARAMETERS:

Pa (kip) =	0.00	Pn/1.67 (kip) =	264.14
Max (kip-ft) =	-289.58	Mnx/1.67 (kip-ft) =	334.33
May (kip-ft) =	-0.00	Mny/1.67 (kip-ft) =	33.13
B1x =	1.00	B1y =	1.00
B2x =	1.26	B2y =	1.00
Baxial =	1.26	@angle (degrees) =	0.00
Cbx =	2.10		

INTERACTION EQUATION:

Pa/(Pn/1.67)) = 0.000
Eq H1-1b: 0.000 + 0.866 + 0.000 = 0.866

Member Forces

RAM Frame v13.0
8810 AISC
DataBase: 8810 AISC
Building Code: IBC

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STEEL BEAM INFORMATION:

Beam Number: 69 **Frame Number: 2**
Level: 2nd I-End (150.00,0.00) J-End (180.00,0.00)
Fy (ksi) = 50.00 Beam Size = W24X55
Length (ft) = 30.00
Elastic Modulus (ksi) = 29000.00

INPUT PARAMETERS:

		I-End	**J-End**
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Rigid End Zone (in):		0.00	0.00 (Ignore)
Member Force Output:		At Face of Joint	
P-Delta:	No		
Ground Level:	Base		

LOAD CASES:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW
E1	E	EQ_IBC06_X_+E_F
E2	E	EQ_IBC06_X_-E_F
E3	E	EQ_IBC06_Y_+E_F
E4	E	EQ_IBC06_Y_-E_F
ND1	N	NL_AISC360_DL_X
ND2	N	NL_AISC360_DL_Y
NL1	N	NL_AISC360_LL_X
NL2	N	NL_AISC360_LL_Y
NR1	N	NL_AISC360_Rf_X
NR2	N	NL_AISC360_Rf_Y

Member Forces

MEMBER FORCES:

LdC	@	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
D	i	0.00	-136.33	0.00	24.03	-0.00	-0.00
	j	0.00	-139.07	-0.00	-24.22	-0.00	-0.00
Lp	i	-0.00	-82.69	0.00	13.46	0.00	-0.00
	j	-0.00	-83.76	0.00	-13.54	0.00	-0.00
Sp	i	0.00	-0.04	-0.00	0.00	0.00	0.00
	j	0.00	0.07	-0.00	0.00	0.00	0.00
W1	i	0.00	41.67	0.00	-2.90	0.00	0.00
	j	0.00	-41.77	0.00	-2.90	0.00	0.00
W2	i	0.00	0.00	0.00	-0.00	0.00	-0.00
	j	0.00	-0.00	0.00	-0.00	0.00	-0.00
W3	i	-0.00	30.50	0.00	-2.12	0.00	0.00
	j	-0.00	-30.57	0.00	-2.12	0.00	0.00
W4	i	0.00	32.01	0.00	-2.22	0.00	-0.00
	j	0.00	-32.08	0.00	-2.22	0.00	-0.00
W5	i	0.00	2.57	0.00	-0.18	0.00	-0.00
	j	0.00	-2.58	0.00	-0.18	0.00	-0.00
W6	i	0.00	-2.57	0.00	0.18	-0.00	0.00
	j	0.00	2.58	-0.00	0.18	-0.00	0.00
W7	i	0.00	31.25	0.00	-2.17	-0.00	-0.00
	j	0.00	-31.32	-0.00	-2.17	-0.00	-0.00
W8	i	-0.00	31.25	0.00	-2.17	-0.00	0.00
	j	-0.00	-31.32	-0.00	-2.17	-0.00	0.00
W9	i	-0.00	20.94	0.00	-1.46	0.00	0.00
	j	-0.00	-20.99	0.00	-1.46	0.00	0.00
W10	i	-0.00	25.93	-0.00	-1.80	0.00	-0.00
	j	-0.00	-25.99	-0.00	-1.80	0.00	-0.00
W11	i	0.00	20.94	0.00	-1.46	-0.00	0.00
	j	0.00	-20.99	-0.00	-1.46	-0.00	0.00
W12	i	0.00	25.93	0.00	-1.80	0.00	-0.00
	j	0.00	-25.99	0.00	-1.80	0.00	-0.00
E1	i	-0.00	130.20	0.00	-9.05	0.00	0.00
	j	-0.00	-130.51	0.00	-9.05	0.00	0.00
E2	i	0.00	132.65	-0.00	-9.22	0.00	-0.00
	j	0.00	-132.96	-0.00	-9.22	0.00	-0.00
E3	i	0.00	2.13	-0.00	-0.15	0.00	-0.00
	j	0.00	-2.14	-0.00	-0.15	0.00	-0.00
E4	i	0.00	-2.13	-0.00	0.15	0.00	0.00
	j	0.00	2.14	0.00	0.15	0.00	0.00
ND1	i	0.00	4.97	0.00	-0.35	0.00	0.00
	j	0.00	-4.98	0.00	-0.35	0.00	0.00
ND2	i	0.00	0.00	0.00	-0.00	-0.00	-0.00
	j	0.00	-0.00	-0.00	-0.00	-0.00	-0.00
NL1	i	0.00	1.57	0.00	-0.11	0.00	0.00

Member Forces

LdC	@	P	Mmajor	Mminor	Vmajor	Vminor	Tors
	j	0.00	-1.57	0.00	-0.11	0.00	0.00
NL2	i	0.00	0.00	0.00	-0.00	-0.00	-0.00
	j	0.00	-0.00	-0.00	-0.00	-0.00	-0.00
NR1	i	-0.00	0.53	0.00	-0.04	0.00	0.00
	j	-0.00	-0.53	0.00	-0.04	0.00	0.00
NR2	i	0.00	0.00	0.00	-0.00	-0.00	-0.00
	j	0.00	-0.00	-0.00	-0.00	-0.00	-0.00

STABILITY AND ANALYSIS PROVISIONS OF THE 2005 AISC SPECIFICATION

R. SHANKAR NAIR

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ABSTRACT

The provisions regarding analysis and, especially, stability in the 2005 AISC Specification for Structural Steel Buildings represent a significant departure from earlier editions. The changes were intended to reflect the current state of knowledge and also to make the specification more transparent to the user. The new provisions spell out the general safety- and reliability-based requirements that must be satisfied by all structural designs, giving designers the freedom to select or devise their own methods of analysis and design within these constraints, and also provide "prescriptive" methods for those who prefer that approach.

This paper discusses the logical basis of the new Specification requirements for stability, and outlines the three alternative prescriptive methods that are specified. The most versatile and powerful of these methods is the Direct Analysis Method. An Appendix to this paper offers a model specification reformulated around the Direct Analysis Method alone, making it easier to understand and use. This represents the direction in which the AISC Specification appears to be evolving; the stability section of the next edition is likely to resemble this model specification.

INTRODUCTION

In today's engineering practice, there is no such thing as a "normal" or "standard" structural analysis: Advanced analysis methods that were regarded as research tools a few years ago have entered some design offices while other practices are still using the same (except bigger and faster) analysis tools they had a generation ago. This is especially true in the area of stability, where direct, rigorous second-order analysis is routine in some practices but not in others. This range in analysis options is especially important in the area of stability because of the close interrelationship between stability design and analysis.

The provisions regarding analysis and, especially, stability in the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005) represent a significant departure from earlier editions. The new specification recognizes the wide range of analyses in common use. It spells out the general safety- and reliability-based requirements that must be satisfied by all structural designs, giving designers the freedom to select or devise their own methods of analysis and design within these constraints, and also provides "prescriptive" methods for those (possibly a large majority of designers) who prefer that approach.

This paper discusses the logical basis of the new specification requirements for stability, and outlines the three alternative prescriptive methods that are specified. The most versatile and powerful of these methods is the Direct Analysis Method. An appendix to this paper offers a model specification reformulated around the Direct Analysis Method alone, making it easier to understand and use. This represents the direction in which the AISC Specification appears to be evolving; the stability section of the next edition is likely to resemble this model specification.

GENERAL REQUIREMENTS

The chapter of the *Specification* on Design Requirements (Chapter B) specifies that the design of structural components must be consistent with the assumptions made in the structural analysis used to determine the required strengths of the components. There are no other constraints on the method of analysis.

The chapter on Stability Analysis and Design (Chapter C) specifies that the design of the structure for stability must consider all of the following:

- Flexural, shear and axial deformations of members.
- All other component and connection deformations that contribute to displacements of the structure.
- P-Δ effects, which are the effects of loads acting on the displaced location of points of intersection of members in the structure. (In typical building structures, this is the effect of loads acting on the laterally displaced location of floors and roofs.)
- P-δ effects, which are the effects of loads acting on the deformed shape of individual members.
- Geometric imperfections, such as initial out-of-plumbness.
- The reduction in member stiffness due to inelasticity (including residual stress effects) and, in particular, the effect of this stiffness reduction on the stability of the structure.

When the required strengths of members have been determined from an analysis that considers all the above effects, the members can be designed using the provisions for design of individual members (provided in Chapters D, E, F, G, H and I).

The *Specification* states explicitly that any method of analysis and design that considers all the specified effects is permissible, and then presents certain specific approaches that account for the last four of the listed effects (P-Δ effects, P-δ effects, geometric imperfections, inelasticity).

DIRECT ANALYSIS METHOD

The most generally-applicable method of accounting for P-Δ and P-δ effects, geometric imperfections and inelasticity is the "Direct Analysis Method" (presented in Appendix 7 of the *Specification*). It is applicable to all types of structural systems; the provisions of the Direct Analysis Method do not distinguish between braced frames, moment-resisting frames, shear wall systems, and combinations of these and other structure types. In the Direct Analysis Method:

- P-Δ and P-δ effects are accounted for through second-order analysis (either explicit second-order analysis or second-order analysis by amplified first-order analysis, for which a procedure is presented in the *Specification*).
- Geometric imperfections are accounted for either by direct inclusion of imperfections in the analysis model or by the application of "notional loads" (which are a proportion of the gravity load, applied laterally).
- Stiffness reductions due to inelasticity are accounted for by reducing the flexural and axial stiffnesses of members by specified amounts or, at the designer's option, by a combination of reduced member stiffness and additional notional loads.

When the required strengths of members have been determined from an analysis conforming to the above requirements, individual members can be designed using an effective length factor of unity in calculating the nominal strengths of members subject to compression.

The *Specification* provides enough direction to allow application of the Direct Analysis Method in "cook book" fashion. But it also lays out the logical basis for the provisions in a way that offers designers the option of tailoring the method to particular situations. For instance, it is spelled out that the specified 0.002 notional load coefficient to account for geometric imperfections is based on a maximum initial story out-of-plumbness ratio of 1/500; a different notional load can be used if the known or anticipated out-of-plumbness is different; the imperfections can even be modeled explicitly instead of applying notional loads.

In time, if not immediately, the Direct Analysis Method will almost certainly become the "standard" method of stability design of steel building structures.

INDIRECT METHODS

For structures in which second-order effects are not very large (where the ratio of second-order drift to first-order drift is below a specified threshold), the *Specification* offers two alternatives to the Direct Analysis Method.

Effective Length Method. In this method, the structure is analyzed using the nominal geometry and nominal elastic stiffness of all members; required member strengths are determined from a second-order analysis (either explicit second-order analysis or second-order analysis by amplified first-order analysis); all gravity-only load combinations include a minimum lateral load at each frame level of 0.002 of the gravity load applied at that level. Effective length factors (K) or buckling stresses for calculating the nominal strengths of compression members must be determined from a sidesway buckling analysis, except that K=1 may be used for braced frames or where the ratio of second-order drift to first-order drift is less than 1.1.

First-Order Analysis Method. This method is applicable only when the required compressive strength is less than half the yield strength in all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure. In this method, the structure is analyzed using the nominal geometry and nominal elastic stiffness of all members; required member strengths are determined from a first-order analysis; all load combinations include an additional lateral load at each frame level of a magnitude based on the gravity load applied at that level and the lateral stiffness of the structure. The nominal strengths of compression members may be determined assuming K=1; beam-column moments must be adjusted (using a formula that is provided) to account for non-sway amplification.

	Direct Analysis Method	**Effective Length Method**	**First-Order Analysis Method**
Specification reference	Appendix 7	Section C.2.2a	Section C.2.2b
Limits on applicability?	No	Yes	Yes
Type of analysis	Second-Order	Second-Order	First-Order
Member stiffness	Reduced EI & EA	Nominal EI & EA	Nominal EI & EA
Notional lateral load?	Yes	Yes	Additional lateral load
Column effective length	K=1	Sidesway buckling analysis	K=1

TABLE 1
COMPARISON OF ANALYSIS AND DESIGN OPTIONS

The alternative analysis methods and corresponding stability design requirements in the AISC *Specification* are summarized in Table 1.

METHODS OF SECOND-ORDER ANALYSIS

As noted in the discussion of alternative analysis-design approaches, the Direct Analysis Method and one of the two indirect methods require a second-order analysis of the structure. The second-order analysis can take the form of an explicit second-order analysis that includes both P-

Δ and P-δ effects. Alternatively, the second-order analysis can consist of amplified first-order analysis, for which a detailed procedure is provided in the *Specification*. (This is the "B1-B2" procedure familiar to designers from previous editions of the *Specification*.)

Since stability is an inherently nonlinear phenomenon, it is essential that all second-order analyses be carried out at the LRFD load level. To obtain the proper level of reliability when ASD is used, the analysis must be conducted under 1.6 times the ASD load combinations and the results must then be divided by 1.6 to obtain the forces and moments for member design by ASD. (The 1.6 load multiplier must also be used, in ASD, when checking the ratio of second-order drift to first-order drift, as required under certain provisions.)

Source of Additional Information

This outline of the analysis provisions in the 2005 AISC *Specification* is intended primarily as an introduction to these provisions and to show the logical progression of the provisions from general requirements applicable to all structures to specific procedures that designers may choose to use for the design of typical structures. More information on the rational basis of the new *Specification* provisions can be found in the Commentary to the *Specification* and the references listed therein.

FURTHER DEVELOPMENTS

The most versatile and powerful of the three alternative methods of stability analysis and design in the 2005 AISC *Specification* is the Direct Analysis Method. An appendix to this paper offers a model specification reformulated around the Direct Analysis Method alone, making it easier to understand and use. This represents the direction in which the AISC Specification appears to be evolving; the stability section of the next edition is likely to resemble this model specification. A second appendix explains the substantive differences between this model specification and the present AISC *Specification*.

Reference

AISC (2005), *Specification for Structural Steel Buildings*, ANSI/AISC 360-05, American Institute of Steel Construction, Inc., Chicago

APPENDIX
SPECIFICATION FOR STABILITY DESIGN BY DIRECT ANALYSIS

As discussed in the paper to which this is an appendix, the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005) offers three alternatives for the design of structures for stability. The main body of the *Specification*, in Chapter C, prescribes two methods: the Effective Length Method in Section C2.2a and the First-Order Analysis Method in Section C2.2b. Appendix 7 presents the Direct Analysis Method. The Effective Length and First-Order Analysis Methods are of limited applicability; the Direct Analysis Method is applicable to all structures.

Of the three methods, the Effective Length Method will be most familiar to users of previous editions of the *Specification* and that is why it was placed in the main body of the current edition. The Direct Analysis Method (now in an Appendix) is, however, the most powerful and versatile of the available methods and, as noted, it is applicable to all structures, unlike the other approaches. There is little doubt that in time the Direct Analysis Method will become the "standard" method of design for stability.

In this appendix, the stability provisions of the 2005 AISC *Specification* are rewritten around the Direct Analysis Method. The material is presented in the language and format of the AISC *Specification*, including "User Notes" and the italicizing of terms listed in the glossary. The focus on a single method has offered the opportunity to expand some of the provisions beyond what is in the current *Specification*, both to improve clarity and to address issues that have arisen from use of the document. Where this involved substantive changes, they are explained in a second appendix.

What follows is not an approved AISC specification. In the author's judgment, however, a design that conformed to the following "model" specification would also conform to the stability provisions of the 2005 AISC *Specification* (AISC, 2005). This reformulation represents the direction in which the AISC *Specification* appears to be evolving. The stability section (currently Chapter C) of the next edition is likely to resemble this model specification, except that it will almost certainly permit variations of today's Effective Length and First Order Analysis Methods as alternate approaches, specified in appendices.

STABILITY ANALYSIS AND DESIGN

This specification addresses requirements for the analysis and design of structures for stability. It is organized as follows:

1. General Stability Requirements
2. Calculation of Required Strengths
3. Design of Components

1. GENERAL STABILITY REQUIREMENTS

Stability shall be provided for the structure as a whole and for each of its elements. The effects of all of the following on the stability of the structure and its elements shall be considered: (1) flexural, shear and axial member deformations, and all other component and *connection* deformations that contribute to displacements of the structure; (2) *second-order effects* (including *P-Δ* and *P-δ effects*) calculated at a level of loading corresponding to *LRFD load combinations* or 1.6 times *ASD load combinations*; (3) geometric imperfections; and (4) stiffness reductions due to inelasticity.

Any rational method of analysis and design that considers all of the listed effects is permitted. Calculation of *required strengths* in accordance with Section 2 and design of components in accordance with Section 3 is permitted for all structures.

2. CALCULATION OF REQUIRED STRENGTHS

The *required strengths* of components of the structure shall be determined from an analysis conforming to Section 2.1. The analysis shall include consideration of initial imperfections in accordance with Section 2.2 and adjustments to stiffness in accordance with Section 2.3.

2.1. Method of Analysis

The analysis of the structure shall conform to the following requirements:

(1) The analysis shall be an elastic *second-order analysis* that considers both *P-Δ* and *P-δ effects*, except as provided in (2), below.

User Note: The *second-order analysis* may consist of either a rigorous second-order analysis or a first-order analysis amplified to account for second-order effects.

(2) Methods of analysis that neglect the effects of P-δ on nodal displacements in the structure are permitted where the axial *load*s satisfy Equation 2-1 in all members whose flexural stiffnesses are considered to contribute to the stability of the structure.

User Note: Notwithstanding this exclusion in the analysis, *P-δ* effects must still be considered in the design of individual members.

$$\alpha P_r < 0.15 P_{eL} \qquad (2\text{-}1)$$

where

P_r = required axial compressive strength under *LRFD* or *ASD load combinations*, kips (N)
P_{eL} = Euler buckling load, $\pi^2 EI/L^2$, evaluated in the plane of bending, kips (N)

and

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

(3) The analysis shall consider flexural, shear and axial member deformations, and all other component and *connection* deformations that contribute to displacements of the structure. The analysis shall use reduced stiffnesses for all components whose stiffnesses are considered to contribute to the stability of the structure, as specified in Section 2.3.

(4) For design by LRFD, the *second-order analysis* shall be carried out under *LRFD load combinations*. For design by ASD, the *second-order analysis* shall be carried out under 1.6 times the *ASD load combinations*, and the results shall be divided by 1.6 to obtain the *required strengths* of components.

2.2. Consideration of Initial Imperfections

The effect of initial imperfections on the stability of the structure shall be taken into account either by direct modeling of imperfections in the analysis as specified in Section 2.2a or by the application of *notional loads* as specified in Section 2.2b.

User Note: The imperfections considered in this section are imperfections in the locations of points of intersection of members in the unloaded structure. In typical building structures, the important imperfection of this type is the out-of-plumbness of columns. Initial out-of-straightness of individual members is not addressed in this section; it is accounted for by other aspects of the analysis and design procedure and need not be considered explicitly as long as it is within customary limits.

2.2a. Direct Modeling of Imperfections

In all cases, it is permissible to account for the effect of initial imperfections by including the imperfections in the analysis. The structure shall be analyzed with points of intersection of members displaced from their nominal locations by the maximum amount considered in the design. The pattern of initial displacements shall be such that it provides the greatest destabilizing effect.

User Note: Initial displacements similar in configuration to both displacements due to loading and anticipated buckling modes should be considered in the modeling of imperfections.

In the analysis of structures to which Sections 2.2b and 2.2b(4) are applicable, subject to load combinations that would require no application of notional loads under Section 2.2b(4), it is permissible to neglect the effect of initial imperfections.

2.2b. Use of Notional Loads to Represent Imperfections

For building structures that support gravity loads primarily through nominally-vertical columns, walls or frames, it is permissible to use *notional loads* to represent the effect of initial imperfections in accordance with the requirements of this section.

User Note: The notional load concept is applicable to all types of structures, but the specific requirements in 2.2b(1) through 2.2b(4) are applicable only for the particular class of structure identified above.

(1) *Notional loads* shall be applied as *lateral loads* at all levels, independently in two orthogonal directions. The *notional loads* shall be additive to other *lateral loads* and shall be applied in all *load combinations*, except as indicated in (4), below. The magnitude of the *notional loads* shall be:

$$N_i = 0.002Y_i \tag{2-2}$$

where

N_i = *notional load* applied at level *i*, kips (N)

Y_i = *gravity load* from the *LRFD load combination* or 1.6 times the *ASD load combination* applied at level *i*, kips (N)

User Note: The notional loads do not cause a net horizontal reaction on the foundation but may, in some cases, cause horizontal reactions on individual foundation components. A horizontal force of ΣN_i, opposite in direction to the notional loads, may be applied in the analysis at the bases of all columns to yield the correct reactions at the foundation.

(2) The *notional load* at any level, N_i, shall be distributed over the level in the same manner as the *gravity load* at that level. The *notional loads* shall be applied independently in opposite directions.

(3) The *notional load* coefficient of 0.002 in Equation 2-2 is based on a nominal initial story out-of-plumbness ratio of 1/500. Where the use of a different maximum out-of-plumbness is justified, it is permissible to adjust the notional load coefficient proportionally.

(4) For frames in which the ratio of maximum second-order drift to maximum first-order drift (both determined for *LRFD load combinations* or 1.6 times *ASD load combinations*) in all stories is equal to or less than 1.7, it is permissible to apply the notional load, N_i, as a minimum lateral load (such that the total lateral load in any *load combination* at any level is not less than N_i) and not in combination with other lateral loads. The specified drift ratio threshold of 1.7 is based on analyses using stiffnesses adjusted as indicated in Section 2.3. If the drift ratio is determined from analyses using nominal, unreduced stiffnesses, the drift ratio threshold for applying the notional loads as minimum lateral loads shall be taken as 1.5.

2.3. Adjustments to Stiffness

The analysis of the structure to determine the *required strengths* of components shall use reduced stiffnesses, as follows:

(1) A factor of 0.8 shall be applied to all axial, shear and flexural stiffnesses that are considered to contribute to the stability of the structure. It is permissible to apply this reduction factor to all stiffnesses in the structure.

User Note: Applying the stiffness reduction to some members and not others can, in some cases, result in artificial distortion of the structure under load. This can be avoided by applying the reduction to all members, including those that do not contribute to the stability of the structure.

(2) An additional factor, τ_b, shall be applied to the flexural stiffnesses of all members whose flexural stiffnesses are considered to contribute to the stability of the structure, where:

$\tau_b = 1.0$ for $\alpha P_r/P_y \leq 0.5$
$= 4[\alpha P_r/P_y\,(1-\alpha P_r/P_y)]$ for $\alpha P_r/P_y > 0.5$
P_r = required axial compressive strength under *LRFD* or *ASD load combinations*, kips (N)
P_y = axial *yield strength*, kips (N)

and

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

(3) In structures to which Section 2.2b is applicable, in lieu of using $\tau_b < 1.0$ where $\alpha P_r/P_y > 0.5$, it is permissible to use $\tau_b = 1.0$ for all members if a *notional load* of $0.001Y_i$ (as defined in Section 2.2b(1)) is applied at all levels, independently in two orthogonal directions, in all *load combinations*. These *notional loads* shall be added to those, if any, used to account for imperfections and shall not be subject to the limits of Section 2.2b(4).

3. DESIGN OF COMPONENTS

When *required strengths* have been determined in accordance with Section 2, members and connections shall be designed to satisfy the provisions of Chapters D, E, F, G, H, I and J, as applicable, of the 2005 AISC *Specification for Structural Steel Buildings*, with no further consideration of overall structure stability. The *effective length factor*, *K*, of all members shall be taken as unity unless a smaller value can be justified by rational analysis.

Bracing intended to define the unbraced lengths of members shall have sufficient stiffness and strength to control member movement at the braced points. Methods of satisfying this requirement are provided in Appendix 6, Stability Bracing for Columns and Beams, of the 2005 AISC *Specification.*

User Note: The requirements of Appendix 6 of the 2005 AISC *Specification* are not applicable to bracing that is included in the analysis of the overall structure as part of the overall load-resisting system.

A Comparison of Frame Stability Analysis Methods in ANSI/AISC 360-05

CHARLES J. CARTER and LOUIS F. GESCHWINDNER

ANSI/AISC 360-05 *Specification for Structural Steel Buildings* (AISC, 2005a), hereafter referred to as the AISC *Specification*, includes three prescriptive approaches for stability analysis and design. Table 2-1 in the 13th Edition AISC *Steel Construction Manual* (AISC, 2005b), hereafter referred to as the AISC *Manual*, provides a comparison of the methods and design options associated with each. A fourth approach, referred to as the Simplified Method, is also presented in the AISC *Manual* (see page 2-12) and on the AISC *Basic Design Values* cards. These four methods are illustrated in this paper in order to give the reader a general understanding of the differences between them:

1. The Second-Order Analysis Method (Section C2.2a)
2. The First-Order Analysis Method (Section C2.2b)
3. The Direct Analysis Method (Appendix 7)
4. The Simplified Method (Manual page 2-12; AISC *Basic Design Values* cards)

Two simple unbraced frames are used in this paper. The one-bay frame shown in Figure 1 has a rigid roof element spanning between a flagpole column (Column A) and leaning column (Column B). Drift is not limited for this frame, which results in a higher ratio of second-order drift to first-order drift, and allows illustration of the detailed requirements in each method for the calculation of K-factors, notional loads, and required and available strengths. The three-bay frame shown in Figure 2 has rigid roof elements spanning between two flagpole columns (Columns D and E) and two leaning columns (Columns C and F). This frame is used with a drift limit of $L/400$ to illustrate the simplifying effect a drift limit can have on the analysis requirements in each method.

Although these example frames are not realistic frames, the results obtained are representative of the impact of second-order elastic and inelastic effects on strength requirements in real frames, particularly when the number of moment connections is reduced. The loads shown in Figures 1 and 2 are from the controlling load and resistance factor design (LRFD) load combination and the corresponding designs are performed using LRFD. The process is essentially identical for allowable strength design (ASD), where ASD load combinations are used with $\alpha = 1.6$ as a multiplier, when required in each method, to account for the second-order effects at the ultimate load level.

When it is required to include second-order effects, the B_1-B_2 amplification is used with a first-order analysis throughout this paper. A direct second-order analysis is straightforward and could have been used instead of the B_1-B_2 amplification.

THE ONE-BAY FRAME

A trial shape is selected using a first-order analysis without consideration of drift limits or second-order effects. Thereafter, that trial shape is used as the basis for comparison of the four methods discussed earlier.

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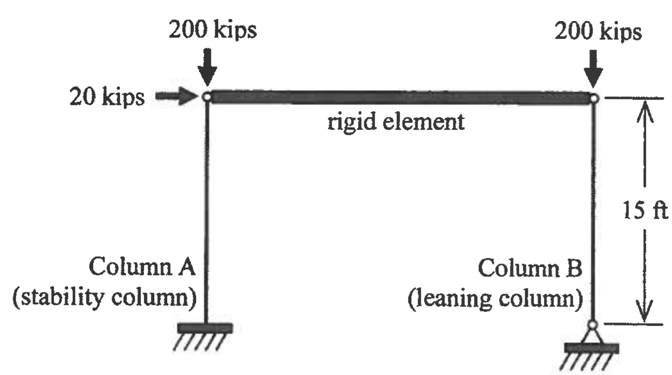

Fig. 1. One-bay unbraced frame used in examples.

Selection of Trial Shape Based Upon Strength Consideration Only

Based upon the loading shown in Figure 1, the first-order axial force, strong-axis moment, and design parameters for Column A are:

$$P_u = 200 \text{ kips} \qquad M_{ux} = (20 \text{ kips})(15 \text{ ft})$$
$$K_x = 2.0 \qquad = 300 \text{ kip-ft}$$
$$K_y = 1.0 \qquad C_b = 1.67$$
$$L_x = L_y = 15 \text{ ft} \qquad L_b = 15 \text{ ft}$$

Note that $K_x = 2.0$, the theoretical value for a column with a fixed base and top that is free to rotate and translate, is used rather than the value of 2.1 recommended for design in the AISC *Specification* Commentary Table C-C2.2. The value of 2.0 is used because it is consistent with the formulation of the lateral stiffness calculation below. Note also that the impact of the leaning column on K_x is ignored in selecting the trial size, although it will be considered in subsequent sections when K_x cannot be taken equal to 1 for Column A. Out of the plane of the frame, K_y is taken as 1.0.

A simple rule of thumb for trial beam-column selection is to use an equivalent axial force equal to P_u plus $24/d$ times M_u, where d is the nominal depth of the column (Geschwindner, Disque and Bjorhovde, 1994). Using d = 14 in. for a W14, the equivalent axial force is 714 kips and an ASTM A992 W14×90 is selected as the trial shape.

The lateral stiffness of the frame depends on Column A only and is:

$$\begin{aligned} k &= 3EI/L^3 \\ &= 3(29{,}000 \text{ ksi})(999 \text{ in.}^4)/(15 \text{ ft} \times 12 \text{ in./ft})^3 \\ &= 14.9 \text{ kips/in.} \end{aligned}$$

The corresponding first-order drift of the frame is:

$$\begin{aligned} \Delta_{1st} &= (20 \text{ kips})/(14.9 \text{ kips/in.}) \\ &= 1.34 \text{ in.} \end{aligned}$$

Note that this is a very flexible frame with Δ_{1st}/L = 1.34/ (15 ft × 12 in./ft) = 1/134.

Design by Second-Order Analysis (Section C2.2a)

Design by second-order analysis is essentially the traditional effective length method with an additional requirement for a minimum lateral load. It is permitted when the ratio of second-order drift, Δ_{2nd}, to first-order drift, Δ_{1st}, is equal to or less than 1.5, and requires the use of:

1. A direct second-order analysis or a first-order analysis with B_1-B_2 amplification.
2. The nominal frame geometry with a minimum lateral load (a "notional load") $N_i = 0.002Y_i$, where Y_i is the total gravity load on level i from LRFD load combinations (or 1.6 times ASD load combinations). This notional load is specified to capture the effects of initial out-of-plumbness up to the AISC *Code of Standard Practice* maximum value of 1:500. In this method, N_i is not applied when the actual lateral load is larger than the calculated notional load.
3. The nominal stiffnesses EA and EI.
4. LRFD load combinations, or ASD load combinations multiplied by 1.6. This multiplier on ASD load combinations ensures that the drift level is consistent for LRFD and ASD when determining second-order effects. The forces and moments obtained in this analysis are then divided by 1.6 for ASD member design.

When the ratio of second-order drift to first-order drift, which is given by B_2, is equal to or less than 1.1, K = 1.0 can be used in the design of moment frames. Otherwise, for moment frames, K is determined from a sidesway buckling analysis. Section C2.2a(4) indicates that for braced frames, K = 1.0.

For the example frame given in Figure 1, the minimum lateral load based upon the total gravity load, Y_i, is:

$$\begin{aligned} Y_i &= 200 \text{ kips} + 200 \text{ kips} \\ &= 400 \text{ kips} \end{aligned}$$

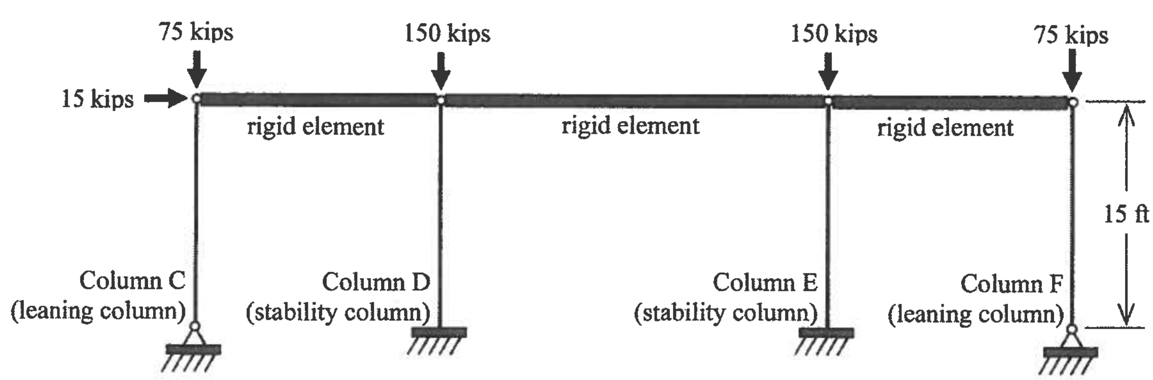

Fig. 2. Three-bay unbraced frame used in examples.

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

Because this notional load is less than the actual lateral load, it need not be applied. For a load combination that did not include a lateral load, the notional load would need to be included in the analysis.

For Column A, using first-order analysis and B_1-B_2 amplification:

$P_{nt} = 200$ kips, $P_{lt} = 0$ kips
$M_{nt} = 0$ kip-ft, $M_{lt} = 300$ kip-ft

For P-δ amplification, since there are no moments associated with the no-translation case, there is no need to calculate B_1. For P-Δ amplification, the first-order drift ratio is determined from the calculated drift of 1.34 in. Thus,

$$\Delta_{1st}/L = (1.34 \text{ in.})/(15 \text{ ft} \times 12 \text{ in./ft})$$
$$= 0.00744$$

For moment frames, $R_m = 0.85$ and from Equation C2-6b with $\Delta_H = \Delta_{1st}$ and $\Sigma H = 20$ kips,

$$\Sigma P_{e2} = R_m \Sigma H/(\Delta_{1st}/L)$$
$$= 0.85\,(20 \text{ kips})/(0.00744)$$
$$= 2{,}280 \text{ kips}$$

For design by LRFD, $\alpha = 1.0$ and ΣP_{nt} is the sum of the gravity loads. Thus,

$$\alpha \Sigma P_{nt}/\Sigma P_{e2} = 1.0\,(200 \text{ kips} + 200 \text{ kips})/2{,}280 \text{ kips}$$
$$= 0.175$$

From Equation C2-3, the amplification is:

$$B_2 = \frac{1}{1 - \dfrac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1$$
$$= \frac{1}{1 - 0.175} \geq 1$$
$$= 1.21 \geq 1.0$$
$$= 1.21$$

Because $B_2 = 1.21$, the second-order drift is less than 1.5 times the first-order drift. Thus, the use of this method is permitted. Because $B_2 > 1.1$, K cannot be taken as 1.0 for column design in the moment frame with this method. Thus, K must be calculated, including the leaning-column effect. Several approaches are available in the AISC *Specification* Commentary to include this effect. A simple approach that uses the ratio of the load on the leaning columns to the load on the stabilizing columns had been provided in previous Commentaries and is used here (Lim and McNamara, 1972):

$$\Sigma P_{leaning}/\Sigma P_{stability} = (200 \text{ kips})/(200 \text{ kips})$$
$$= 1$$

$$K_x^* = K_x(1 + \Sigma P_{leaning} / \Sigma P_{stability})^{1/2}$$
$$= 2.0(1 + 1)^{1/2}$$
$$= 2.83$$

The amplified axial force (Equation C2-1b) and associated design parameters for this method are:

$$P_r = P_{nt} + B_2 P_{lt}$$
$$= 200 \text{ kips} + 1.21(0 \text{ kips})$$
$$= 200 \text{ kips}$$

$$K_x^* = 2.83,\ K_y = 1.0$$

$$L_x = L_y = 15 \text{ ft}$$

The amplified moment (Equation C2-1a) and associated design parameters for this method are:

$$M_{rx} = B_1 M_{nt} + B_2 M_{lt}$$
$$= (0 \text{ kip-ft}) + 1.21\,(300 \text{ kip-ft})$$
$$= 363 \text{ kip-ft}$$

$$C_b = 1.67$$

$$L_b = 15 \text{ ft}$$

Based upon these design parameters, the axial and strong-axis available flexural strengths of the ASTM A992 W14×90 are:

$$P_c = \phi_c P_n$$
$$= 721 \text{ kips}$$

$$M_{cx} = \phi_b M_{nx}$$
$$= 573 \text{ kip-ft}$$

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\frac{P_r}{P_c} = \frac{200 \text{ kips}}{721 \text{ kips}}$$
$$= 0.277$$

Thus, because $P_r/P_c \geq 0.2$, Equation H1-1a is applicable.

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right) = 0.277 + \frac{8}{9}\left(\frac{363 \text{ kip-ft}}{573 \text{ kip-ft}}\right)$$
$$= 0.840$$

The W14×90 is adequate because $0.840 \leq 1.0$.

Design by First-Order Analysis (Section C2.2b)

The first-order analysis method is permitted when:

1. The ratio of second-order drift, Δ_{2nd}, to first-order drift, Δ_{1st}, is equal to or less than 1.5.

2. The column axial force $\alpha P_r \leq 0.5 P_y$, where $\alpha = 1.0$ for LRFD, 1.6 for ASD.

This method requires the use of:

1. A first-order analysis.
2. The nominal frame geometry with an additional lateral load $N_i = 2.1(\Delta/L)Y_i \geq 0.0042Y_i$, applied in all load cases.
3. The nominal stiffnesses EA and EI.
4. B_1 as a multiplier on the total moment in beam-columns.
5. LRFD load combinations, or ASD load combinations multiplied by 1.6. This multiplier on ASD load combinations ensures that the drift level is consistent for LRFD and ASD when determining the notional loads. The forces and moments obtained in this analysis are then divided by 1.6 for ASD member design.

For all frames designed with this method, $K = 1.0$.

For the example frame given in Figure 1, the additional lateral load is based on the first-order drift ratio, Δ/L, and the total gravity load, Y_i. Thus, with $\Delta = \Delta_{1st}$,

$$\begin{aligned}\Delta_{1st}/L &= (1.34 \text{ in.})/(15 \text{ ft} \times 12 \text{ in./ft})\\ &= 0.00744\end{aligned}$$

$$\begin{aligned}Y_i &= 200 \text{ kips} + 200 \text{ kips}\\ &= 400 \text{ kips}\end{aligned}$$

$$\begin{aligned}N_i &= 2.1(\Delta_{1st}/L)Y_i \geq 0.0042Y_i\\ &= 2.1(0.00744)(400 \text{ kips}) \geq 0.0042(400 \text{ kips})\\ &= 6.25 \text{ kips} \geq 1.68 \text{ kips}\\ &= 6.25 \text{ kips}\end{aligned}$$

It was previously determined in the illustration of design by second-order analysis example that the second-order drift is less than 1.5 times the first-order drift. Additionally,

$$\begin{aligned}\alpha P_r &= 1.0(200 \text{ kips})\\ &= 200 \text{ kips}\end{aligned}$$

And for a W14×90,

$$\begin{aligned}0.5P_y &= 0.5F_yA_g\\ &= 0.5(50 \text{ ksi})(26.5 \text{ in.}^2)\\ &= 663 \text{ kips}\end{aligned}$$

Because $\Delta_{2nd} < 1.5\Delta_{1st}$ and $\alpha P_r < 0.5P_y$, the use of this method is permitted.

The loading for this method is the same as that shown in Figure 1, except for the addition of a notional load of 6.25 kips coincident with the lateral load of 20 kips shown, resulting in a column moment, M_u, of 394 kip-ft.

This moment must be amplified by B_1 as determined from Equation C2-2. The Euler buckling load is calculated with $K_1 = 1.0$. Thus,

$$\begin{aligned}P_{e1} &= \pi^2EI/(K_1L)^2\\ &= \pi^2(29{,}000 \text{ ksi})(999 \text{ in.}^4)/(1.0 \times 15 \text{ ft} \times 12 \text{ in./ft})^2\\ &= 8{,}830 \text{ kips}\end{aligned}$$

The moment on one end of the column is zero, so the moment gradient term is:

$$\begin{aligned}C_m &= 0.6 - 0.4(M_1/M_2)\\ &= 0.6 - 0.4(0/394 \text{ kip-ft})\\ &= 0.6\end{aligned}$$

From Equation C2-2,

$$\begin{aligned}\alpha P_r/P_{e1} &= 1.0(200 \text{ kips})/(8{,}830 \text{ kips})\\ &= 0.0227\end{aligned}$$

$$\begin{aligned}B_1 &= \frac{C_m}{1-\dfrac{\alpha P_r}{P_{e1}}} \geq 1\\ &= \frac{0.6}{1-0.0227} \geq 1.0\\ &= 0.614 \geq 1.0\\ &= 1.0\end{aligned}$$

The axial force and associated design parameters for this method are:

$$\begin{aligned}P_r &= 200 \text{ kips}\\ K_x &= K_y = 1.0\\ L_x &= L_y = 15 \text{ ft}\end{aligned}$$

The amplified moment and associated design parameters for this method are:

$$\begin{aligned}M_{rx} &= B_1M_u\\ &= 1.0\,(394 \text{ kip-ft})\\ &= 394 \text{ kip-ft}\end{aligned}$$

$$C_b = 1.67$$

$$L_b = 15 \text{ ft}$$

Based on these design parameters, the axial and strong-axis available flexural strengths of the ASTM A992 W14×90 are:

$$\begin{aligned}P_c &= \phi_cP_n = 1{,}000 \text{ kips}\\ M_{cx} &= \phi_bM_{nx} = 573 \text{ kip-ft}\end{aligned}$$

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\begin{aligned}\frac{P_r}{P_c} &= \frac{200 \text{ kips}}{1{,}000 \text{ kips}}\\ &= 0.200\end{aligned}$$

Thus, because $P_r/P_c \geq 0.2$, Equation H1-1a is applicable.

$$\frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right)=0.200+\frac{8}{9}\left(\frac{394 \text{ kip-ft}}{573 \text{ kip-ft}}\right)$$
$$=0.811$$

The W14×90 is adequate since $0.811 \leq 1.0$.

Design by Direct Analysis (Appendix 7)

The Direct Analysis Method is permitted for any ratio of second-order drift, Δ_{2nd}, to first-order drift, Δ_{1st}, and required when this ratio exceeds 1.5. It requires the use of:

1. A direct second-order analysis or a first-order analysis with B_1-B_2 amplification.
2. The nominal frame geometry with an additional lateral load of $N_i = 0.002Y_i$, where Y_i is the total gravity load on level i from LRFD load combinations, or 1.6 times ASD load combinations.
3. The reduced stiffnesses EA^* and EI^* (including in B_1-B_2 amplification, if used).
4. LRFD load combinations, or ASD load combinations multiplied by 1.6. This multiplier ensures that the drift level is consistent for LRFD and ASD when determining second-order effects. The forces and moments obtained in this analysis are then divided by 1.6 for ASD member design.

The following exceptions apply as alternatives in item 2:

a. If the out-of-plumb geometry of the structures is used, the notional loads can be omitted.

b. When the ratio of second-order drift to first-order drift is equal to or less than 1.5, the notional load can be applied as a minimum lateral load, not an additional lateral load. Note that the unreduced stiffnesses, EA and EI, are used in this comparison.

c. When the actual out-of-plumbness is known, it is permitted to adjust the notional loads proportionally.

For all frames designed with this method, $K = 1.0$.

It was previously determined in the illustration of design by second-order analysis example that the second-order drift is less than 1.5 times the first-order drift (note that this check is properly made using the unreduced stiffnesses, EA and EI).

Thus, the notional load can be applied as a minimum lateral load, and that minimum is:

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

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

Because this notional load is less than the actual lateral load, it need not be applied. For a load combination that does not include a lateral load, the notional load would need to be included in the analysis.

For Column A, using first-order analysis and B_1-B_2 amplification:

$P_{nt} = 200$ kips, $\quad P_{lt} = 0$ kips
$M_{nt} = 0$ kip-ft, $\quad M_{lt} = 300$ kip-ft

To determine the second-order amplification, the reduced stiffness, EI^*, must be calculated.

$$\alpha P_r = 1.0(200 \text{ kips})$$
$$= 200 \text{ kips}$$

and

$$0.5P_y = 0.5F_yA_g$$
$$= 0.5(50 \text{ ksi})(26.5 \text{ in.}^2)$$
$$= 663 \text{ kips}$$

Thus, because $\alpha P_r < 0.5P_y$, $\tau_b = 1.0$ and

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

For P-δ amplification, since there are no moments associated with the no-translation case, there is no need to calculate B_1. For P-Δ amplification, the reduced stiffness EI^* must be used to determine the first-order drift. Because $EI^* = 0.8EI$, the first-order drift based upon EI^* is 25% larger than that calculated previously. Thus,

$$\Delta_{1st} = 1.25(1.34 \text{ in.})$$
$$= 1.68 \text{ in.}$$

The first-order drift ratio is determined from the amplified drift of 1.68 in.

$$\Delta_{1st}/L = (1.68 \text{ in.})/(15 \text{ ft} \times 12 \text{ in./ft})$$
$$= 0.00933$$

For moment frames, $R_M = 0.85$ and from Equation C2-6b with $\Delta_H = \Delta_{1st}$ and $\Sigma H = 20$ kips,

$$\Sigma P_{e2} = R_M \frac{\Sigma H}{\left(\Delta_{1st}/L\right)}$$
$$= 0.85\frac{20 \text{ kips}}{(0.00933)}$$
$$= 1{,}820 \text{ kips}$$

For design by LRFD, $\alpha = 1.0$ and ΣP_{nt} is the sum of the gravity loads. Thus,

$$\alpha\Sigma P_{nt}/\Sigma P_{e2} = 1.0(200 \text{ kips} + 200 \text{ kips})/1{,}820 \text{ kips} = 0.220$$

From Equation C2-3, the amplification is:

$$B_2 = \frac{1}{1-\dfrac{\alpha\Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1$$

$$= \frac{1}{(1-0.220)} \geq 1.0$$

$$= 1.28 \geq 1.0$$

$$= 1.28$$

It is worth noting that use of the reduced axial stiffness, $EA^* = 0.8EA$, in members that contribute to lateral stability is also required in this method. However, due to the characteristics of the structures chosen for this example, there are no axial deformations that impact the stability of the structure.

The amplified axial force (Equation C2-1b) and associated design parameters for this method are:

$$P_r = P_{nt} + B_2P_{lt}$$

$$= 200 \text{ kips} + 1.28(0 \text{ kips})$$

$$= 200 \text{ kips}$$

$$K_x = K_y = 1.0$$

$$L_x = L_y = 15 \text{ ft}$$

The amplified moment (Equation C2-1a) and associated design parameters for this method are:

$$M_{rx} = B_1M_{nt} + B_2M_{lt}$$

$$= (0 \text{ kip-ft}) + 1.28(300 \text{ kip-ft})$$

$$= 384 \text{ kip-ft}$$

$$C_b = 1.67$$

$$L_b = 15 \text{ ft}$$

Based upon these design parameters, the axial and strong-axis available flexural strengths of the ASTM A992 W14×90 are:

$$P_c = \phi_cP_n = 1{,}000 \text{ kips}$$

$$M_{cx} = \phi_bM_{nx} = 573 \text{ kip-ft}$$

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\frac{P_r}{P_c} = \frac{200 \text{ kips}}{1{,}000 \text{ kips}}$$

$$= 0.200$$

Thus, because $P_r/P_c \geq 0.2$, Equation H1-1a is applicable.

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right) = 0.200 + \frac{8}{9}\left(\frac{384 \text{ kip-ft}}{573 \text{ kip-ft}}\right)$$

$$= 0.796$$

The W14×90 is adequate since $0.796 \leq 1.0$.

The Simplified Method

This method is provided in the AISC *Basic Design Values* Cards and the 13th Edition *Steel Construction Manual* (AISC, 2005b), and excerpted as shown in Figure 3. This simplified method is derived from the effective length method (Design by Second-Order Analysis; Section C2.2a) using B_1-B_2 amplification with B_1 taken equal to B_2. Note that the user note in Section C2.1b says that B_1 may be taken equal to B_2 as long as B_1 is less than 1.05. However, it is also conservative to take B_1 equal to B_2 any time B_1 is less than B_2. Although it cannot universally be stated that B_1 is always equal to or less than B_2, this is the case for typical framing. It is left to engineering judgment to confirm that this criterion is satisfied when applying the simplified method.

This method is permitted when the ratio of second-order drift, Δ_{2nd}, to first-order drift, Δ_{1st}, is equal to or less than 1.5 as with the Design by Second-Order Analysis method. It allows the use of a first-order analysis based on nominal stiffnesses, EA and EI, with a minimum lateral load $N_i = 0.002Y_i$, where Y_i is the total gravity load on level i from LRFD load combinations or ASD load combinations. The 1.6 multiplier on ASD load combinations is not used at this point but its effect is included in the determination of the amplification multiplier upon entering the table.

The ratio of total story gravity load (times 1.0 in LRFD, 1.6 in ASD) to the story lateral load is used to enter the table in Figure 3. The second-order amplification multiplier is determined from the value in the table corresponding to the calculated load ratio and design story drift limit. While linear interpolation between tabular values is permitted, it is important to note that the tabular values have, in essence, only two significant digits. Accordingly, the value determined should not be calculated to more than one decimal place. The tabular value is used to amplify all forces and moments in the analysis.

When the ratio of second-order drift to first-order drift is equal to or less than 1.1, $K = 1.0$ can be used in the design of moment frames. Otherwise, for moment frames, K is determined from a sidesway buckling analysis. For braced frames, $K = 1.0$.

For the example frame given in Figure 1, the minimum lateral load is:

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

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

Because this notional load is less than the actual lateral load, it need not be applied. For a load combination that does not include a lateral load, the notional load would need to be included in the analysis.

The actual first-order drift of the trial frame corresponds to a drift ratio of $L/134$ and the load ratio is:

$$1.0 \times (200 \text{ kips} + 200 \text{ kips})/(20 \text{ kips}) = 20$$

Entering the table in the column for a load ratio of 20, the corresponding multiplier for a drift ratio of $H/134$ is 1.3 (determined by interpolation to one decimal place). This multiplier is less than 1.5; thus, $\Delta_{2nd} < 1.5\Delta_{1st}$ and the use of this method is permitted. However, because the multiplier is greater than 1.1, K cannot be taken as 1.0 for column design in the moment frame with this method. Thus, K must be calculated, including the leaning column effect. Using the same approach as previously discussed (Lim and McNamara, 1972):

$$\Sigma P_{leaning}/\Sigma P_{stability} = (200 \text{ kips})/(200 \text{ kips}) = 1$$

$$K_x^* = K_x(1 + \Sigma P_{leaning}/\Sigma P_{stability})^{½} = 2.0(1 + 1)^{½} = 2.83$$

The amplified axial force (with the full axial force amplified by B_2) and associated design parameters for this method are:

$$P_r = 1.3P_u = 1.3 \text{ (200 kips)} = 260 \text{ kips}$$

$$K_x^* = 2.83, K_y = 1.0$$

$$L_x = L_y = 15 \text{ ft}$$

The amplified moment (with the full moment amplified by B_2) and associated design parameters for this method are:

$$M_{rx} = 1.3M_u = 1.3(300 \text{ kip-ft}) = 390 \text{ kip-ft}$$

$$C_b = 1.67$$

$$L_b = 15 \text{ ft}$$

Based on these design parameters, the available axial compressive strength and strong-axis available flexural strength of the ASTM A992 W14×90 are:

$$P_c = \phi_c P_n = 721 \text{ kips}$$
$$M_{cx} = \phi_b M_{nx} = 573 \text{ kip-ft}$$

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\frac{P_r}{P_c} = \frac{260 \text{ kips}}{721 \text{ kips}} = 0.361$$

Simplified Method

Step 1. Perform first-order analysis. Use 0.2% of total story gravity load as minimum lateral load in all load combinations.
Step 2. Establish the design story drift limit and determine the lateral load required to produce it.
Step 3. Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For ASD, multiply by 1.6.
Step 4. Multiply first-order results by the tabular value. K=1, except for moment frames when the tabular value is greater than 1.1.

Design Story Drift Limit	Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)									
	0	5	10	20	30	40	50	60	80	100	120
H/100	1	1.1	1.1	1.3	1.4				When ratio exceeds 1.5, simplified method requires a stiffer structure.		
H/200	1	1	1.1	1.1	1.2	1.3	1.3	1.4			
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	1.4	1.5	
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.3	1.4
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.3

Fig. 3. Simplified method from AISC basic design values cards.

Thus, because $P_r/P_c \geq 0.2$, Equation H1-1a is applicable.

$$\frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right)=0.361+\frac{8}{9}\left(\frac{390\text{ kip-ft}}{573\text{ kip-ft}}\right)$$
$$=0.966$$

The W14×90 is adequate since $0.966 \leq 1.0$.

Summary for the One-Bay Frame

All methods illustrated in the foregoing sections produce similar designs. The results are tabulated here for comparison, where the result of the beam-column interaction equation is given for each method. A lower interaction equation result for the same column shape signifies a prediction of higher strength.

Method	Interaction Equation
Second-Order	0.840
First-Order	0.811
Direct Analysis	0.796
Simplified	0.966

In this example, the direct analysis method predicts the highest strength, while the simplified method predicts the lowest strength. This would be expected because the Direct Analysis Method was developed as the most accurate approach while the simplified method was developed to produce a quick yet conservative solution.

The designs compared here are based on strength with no consideration of drift limitation, except to the extent that the actual drift impacts the magnitude of the second-order effects. The usual drift limits of approximately $L/400$ will necessitate framing members and configurations with more lateral stiffness than this frame provides. Hence, the designer may find that a frame configured for drift first will often require no increase in member size for strength, including second-order effects. This will be explored further with the three-bay frame.

THE THREE-BAY FRAME

For the frame shown in Figure 2, a trial shape is selected using a first-order drift limit of $L/600$ under a service level lateral load of 10 kips. Thereafter, that trial shape is used as the basis for comparison of the four methods used previously for the one-bay frame.

Selection of Trial Shape Based on the Drift Limit Only

For the dimensions shown in Figure 2:

$$L/600 = (15\text{ ft} \times 12\text{ in./ft})/600$$
$$= 0.300\text{ in.}$$

The lateral stiffness of the frame depends on Columns D and E only, and based on a classical stiffness derivation with the given end conditions, it is calculated as follows:

$$k = 2 \times 3EI/L^3$$
$$= 2 \times 3(29{,}000\text{ ksi})(I)/(15\text{ ft} \times 12\text{ in./ft})^3$$
$$= 0.0298(I)$$

With the service level lateral load on the frame of 10 kips:

$$0.0298(I) \geq (10\text{ kips})/(0.300\text{ in.})$$

Thus, I_{req} = 1,120 in.^{4} and an ASTM A992 W14×109 is selected as the trial shape with I_x = 1,240 in.^{4}

The actual lateral stiffness of the frame is:

$$k = 2 \times 3EI/L^3$$
$$= 2 \times 3(29{,}000\text{ ksi})(1{,}240\text{ in.}^4)/(15\text{ ft} \times 12\text{ in./ft})^3$$
$$= 37.0\text{ kips/in.}$$

The corresponding first-order drift of the frame under the LRFD lateral load of 15 kips is:

$$\Delta_{1st} = (15\text{ kips})/(37.0\text{ kips/in.})$$
$$= 0.405\text{ in.}$$

The first-order axial force, strong-axis moment, and design parameters for Columns D and E are:

P_u = 150 kips M_{ux} = (15 kips)(15 ft)/2
K_x = 2.0 = 113 kip-ft
K_y = 1.0 C_b = 1.67
$L_x = L_y$ = 15 ft L_b = 15 ft

Note that K_x = 2.0, the theoretical value for a column with a fixed base and pinned top, is used rather than the value of 2.1 recommended for design in the AISC *Specification* Commentary Table C-C2.2. The value of 2.0 is used because it is consistent with the formulation of the lateral stiffness calculation that follows. Note also that the impact of the leaning column on K_x is ignored in selecting the trial size, although it will be considered in subsequent sections when K_x cannot be taken equal to 1.0 for Column A. Out of the plane of the frame, K_y is taken as 1.0.

Design by Second-Order Analysis (Section C2.2a)

For the example frame given in Figure 2, the minimum lateral load is:

$$Y_i = 75\text{ kips} + 150\text{ kips} + 150\text{ kips} + 75\text{ kips}$$
$$= 450\text{ kips}$$

$$N_i = 0.002\ Y_i$$
$$= 0.002(450\text{ kips})$$
$$= 0.90\text{ kips}$$

Because this notional load is less than the actual lateral load, it need not be applied.

For Columns D and E, using first-order analysis and B_1-B_2 amplification:

$$P_{nt} = 150 \text{ kips}, P_{lt} = 0 \text{ kips}$$
$$M_{nt} = 0 \text{ kip-ft}, M_{lt} = 113 \text{ kip-ft}$$

For P-δ amplification, because there are no moments associated with the no-translation case, there is no need to calculate B_1. For P-Δ amplification, the first-order drift ratio is determined from the calculated drift of 0.405 in. Thus,

$$\begin{aligned}\Delta_{1st}/L &= (0.405 \text{ in.})/(15 \text{ ft} \times 12 \text{ in./ft}) \\ &= 0.00225\end{aligned}$$

For moment frames, $R_m = 0.85$ and from Equation C2-6b with $\Delta_H = \Delta_{1st}$ and $\Sigma H = 15$ kips,

$$\begin{aligned}\Sigma P_{e2} &= R_M \frac{\Sigma H}{\left(\Delta_{1st} / L\right)} \\ &= 0.85 \frac{15 \text{ kips}}{(0.00225)} \\ &= 5{,}670 \text{ kips}\end{aligned}$$

For design by LRFD, $\alpha = 1.0$ and ΣP_{nt} is the sum of the gravity loads. Thus,

$$\begin{aligned}\alpha \Sigma P_{nt}/\Sigma P_{e2} &= 1.0(75 \text{ kips} + 150 \text{ kips} + 150 \text{ kips} + 75 \text{ kips})/5{,}670 \text{ kips} \\ &= 0.0794\end{aligned}$$

From Equation C2-3, the amplification is:

$$\begin{aligned}B_2 &= \frac{1}{\left(1 - \dfrac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}\right)} \geq 1 \\ &= \frac{1}{(1 - 0.0794)} \geq 1.0 \\ &= 1.09 \geq 1.0 \\ &= 1.09\end{aligned}$$

Because $B_2 = 1.09$, the second-order drift is less than 1.5 times the first-order drift. Thus, the use of this method is permitted. Because $B_2 < 1.1$, K can be taken as 1.0 for column design in the moment frame with this method.

The amplified axial force (Equation C2-1b) and associated design parameters for this method are:

$$\begin{aligned}P_r &= P_{nt} + B_2 P_{lt} \\ &= 150 \text{ kips} + 1.09(0 \text{ kips}) \\ &= 150 \text{ kips}\end{aligned}$$

$$K_x = K_y = 1.0$$

$$L_x = L_y = 15 \text{ ft}$$

The amplified moment (Equation C2-1a) and associated design parameters for this method are:

$$\begin{aligned}M_{rx} &= B_1 M_{nt} + B_2 M_{lt} \\ &= (0 \text{ kip-ft}) + 1.09\,(113 \text{ kip-ft}) \\ &= 123 \text{ kip-ft}\end{aligned}$$

$$C_b = 1.67$$

$$L_b = 15 \text{ ft}$$

Based on these design parameters, the available axial compressive strength and strong-axis available flexural strength of the ASTM A992 W14×109 are:

$$P_c = \phi_c P_n = 1{,}220 \text{ kips}$$
$$M_{cx} = \phi_b M_{nx} = 720 \text{ kip-ft}$$

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\begin{aligned}\frac{P_r}{P_c} &= \frac{150 \text{ kips}}{1{,}220 \text{ kips}} \\ &= 0.123\end{aligned}$$

Thus, because $P_r/P_c < 0.2$, Equation H1-1b is applicable.

$$\begin{aligned}\frac{P_r}{2P_c} + \frac{M_{rx}}{M_{cx}} &= \frac{0.123}{2} + \frac{123 \text{ kip-ft}}{720 \text{ kip-ft}} \\ &= 0.232\end{aligned}$$

The W14×109 is adequate because $0.232 \leq 1.0$.

Design by First-Order Analysis (Section C2.2b)

For the example frame given in Figure 2, the additional lateral load (with $\Delta = \Delta_{1st}$) is:

$$\begin{aligned}\Delta_{1st}/L &= (0.405 \text{ in.})/(15 \text{ ft} \times 12 \text{ in./ft}) \\ &= 0.00225\end{aligned}$$

$$\begin{aligned}Y_i &= 75 \text{ kips} + 150 \text{ kips} + 150 \text{ kips} + 75 \text{ kips} \\ &= 450 \text{ kips}\end{aligned}$$

$$\begin{aligned}N_i &= 2.1(\Delta_{1st}/L)Y_i \geq 0.0042Y_i \\ &= 2.1(0.00225)(450 \text{ kips}) \geq 0.0042(450 \text{ kips}) \\ &= 2.13 \text{ kips} \geq 1.89 \text{ kips} \\ &= 2.13 \text{ kips}\end{aligned}$$

It was previously determined in the illustration of design by second-order analysis example that the second-order drift is less than 1.5 times the first-order drift. Additionally,

$$\alpha P_r = 1.0(150 \text{ kips}) = 150 \text{ kips}$$

and for the ASTM A992 W14×109,

$$\begin{aligned}0.5P_y &= 0.5F_yA_g \\ &= 0.5(50 \text{ ksi})(32.0 \text{ in.}^2) \\ &= 800 \text{ kips}\end{aligned}$$

Because $\Delta_{2nd} < 1.5\Delta_{1st}$ and $\alpha P_r < 0.5P_y$, the use of this method is permitted.

The loading for this method is the same as shown in Figure 2, except for the addition of a notional load of 2.13 kips coincident with the lateral load of 15 kips shown, resulting in a moment M_u of 128 kip-ft in each column.

This moment must be amplified by B_1 as determined from Equation C2-2. The Euler buckling load is calculated with $K_1 = 1.0$. Thus,

$$\begin{aligned} P_{e1} &= \pi^2 EI / (K_1 L)^2 \\ &= \pi^2(29{,}000 \text{ ksi})(1{,}240 \text{ in.}^4)/(1.0 \times 15 \text{ ft} \times 12 \text{ in./ft})^2 \\ &= 11{,}000 \text{ kips} \end{aligned}$$

Because the moment on one end of the column is zero, the moment gradient term is:

$$\begin{aligned} C_m &= 0.6 - 0.4(M_1/M_2) \\ &= 0.6 - 0.4(0/128) \\ &= 0.6 \end{aligned}$$

From Equation C2-2,

$$\begin{aligned} \alpha P_r / P_{e1} &= 1.0(150 \text{ kips})/(11{,}000 \text{ kips}) \\ &= 0.0136 \end{aligned}$$

$$\begin{aligned} B_1 &= \frac{C_m}{1 - \dfrac{\alpha P_r}{P_{e1}}} \geq 1 \\ &= \frac{0.6}{1 - 0.0136} \geq 1.0 \\ &= 0.608 \geq 1.0 \\ &= 1.0 \end{aligned}$$

The axial force and associated design parameters for this method are:

$$\begin{aligned} P_r &= 150 \text{ kips} \\ K_x &= K_y = 1.0 \\ L_x &= L_y = 15 \text{ ft} \end{aligned}$$

The amplified moment and associated design parameters for this method are:

$$\begin{aligned} M_{rx} &= B_1 M_u \\ &= 1.0(128 \text{ kip-ft}) \\ &= 128 \text{ kip-ft} \\ C_b &= 1.67 \\ L_b &= 15 \text{ ft} \end{aligned}$$

Based on these design parameters, the available axial compressive strength and strong-axis available flexural strengths of the ASTM A992 W14×109 are:

$$\begin{aligned} P_c &= \phi_c P_n = 1{,}220 \text{ kips} \\ M_{cx} &= \phi_b M_{nx} = 720 \text{ kip-ft} \end{aligned}$$

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\begin{aligned} \frac{P_r}{P_c} &= \frac{150 \text{ kips}}{1{,}220 \text{ kips}} \\ &= 0.123 \end{aligned}$$

Thus, because $P_r/P_c < 0.2$, Equation H1-1b is applicable.

$$\begin{aligned} \frac{P_r}{2P_c} + \frac{M_{rx}}{M_{cx}} &= \frac{0.123}{2} + \frac{128 \text{ kip-ft}}{720 \text{ kip-ft}} \\ &= 0.239 \end{aligned}$$

The W14×109 is adequate because $0.239 \leq 1.0$.

Direct Analysis Method (Appendix 7)

It was previously determined in the illustration of design by second-order analysis example that the second-order drift is less than 1.5 times the first-order drift (note that this check is properly made using the unreduced stiffness EI). Thus, the notional load can be applied as minimum lateral load, and that minimum is:

$$\begin{aligned} Y_i &= 75 \text{ kips} + 150 \text{ kips} + 150 \text{ kips} + 75 \text{ kips} \\ &= 450 \text{ kips} \end{aligned}$$

$$\begin{aligned} N_i &= 0.002 Y_i \\ &= 0.002(450 \text{ kips}) \\ &= 0.9 \text{ kip} \end{aligned}$$

Because this notional load is less than the actual lateral load, it need not be applied.

For Columns D and E, using first-order analysis and B_1-B_2 amplification:

$$\begin{aligned} &P_{nt} = 150 \text{ kips}, P_{lt} = 0 \text{ kips} \\ &M_{nt} = 0 \text{ kip-ft}, M_{lt} = 113 \text{ kip-ft} \end{aligned}$$

To determine the second-order amplification, the reduced stiffness, EI^*, must be calculated.

$$\begin{aligned} \alpha P_r &= 1.0(150 \text{ kips}) \\ &= 150 \text{ kips} \end{aligned}$$

and for the ASTM A992 W14×109,

$$\begin{aligned} 0.5P_y &= 0.5F_y A_g \\ &= 0.5(50 \text{ ksi})(32.0 \text{ in.}^2) \\ &= 800 \text{ kips} \end{aligned}$$

Thus, because $\alpha P_r < 0.5P_y$, $\tau_b = 1.0$ and

$$\begin{aligned} EI^* &= 0.8\tau_b EI \\ &= 0.8EI \end{aligned}$$

For P-δ amplification, because there are no moments associated with the no-translation case, there is no need to calculate B_1. For P-Δ amplification, the reduced stiffness EI^* must be used to determine the first-order drift. Because

APPENDIX
EXPLANATION OF CHANGES

The model specification for stability analysis and design presented in the previous appendix is based on Chapter C and Appendix 7 of the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005). Where substantive technical changes have been made in AISC *Specification* provisions, they are explained in this appendix. The changes are conservative in that a design that conforms to the proposed model specification would also conform to the 2005 AISC *Specification.*

Type of Structure. Some of the provisions of the 2005 AISC *Specification*, specifically those related to the use of notional loads, are applicable only to conventional building structures that support gravity loads primarily through nominally-vertical columns, walls or frames. They are not applicable, for instance, to the analysis of laterally-unsupported compression chords of long-span trusses. This limitation is not noted in the *Specification*, except to the extent that the entire *Specification* is described in the Scope section of Chapter A as being intended for buildings and building-like structures, which would, typically, fall within the constraint.

The model specification is based on the very versatile Direct Analysis Method and is intended to be applicable to a broader range of structures than just conventional building frames. Therefore, those provisions that can only be used for typical building structures (Sections 2.2b and 2.3(3)) are clearly identified and alternatives usable with all structures are provided. [The notional load concept is broadly applicable, but the specific provisions in Sections 2.2b and 2.3(3) are intended only for the limited class of building structures.]

Load Level for Calculation of Second-Order Effects. The requirement that second-order effects be considered at a level of load corresponding to LRFD Load Combinations or 1.6 times ASD Load Combinations is set forth in the general requirements section of the present work. The 2005 AISC *Specification* has this requirement only in the sections on specific methods, which could be taken to imply, incorrectly, that it applied only to those methods and was not a general requirement for all designs.

Application of Notional Loads. Requirements regarding the distribution and direction of notional loads are specified in greater detail in the present work (Section 2.2b(2)). These requirements may have been implicit in the 2005 AISC *Specification*; they are now spelled out.

Drift Ratio Threshold for Applying Notional Loads as Minimum Lateral Loads. In the 2005 AISC *Specification*, the drift ratio (ratio of second-order drift to first-order drift) of 1.5 below which notional loads are applied as minimum lateral loads (rather than as additive loads) is based on analyses with nominal stiffnesses. In the model specification, the user is offered the option of determining the drift ratio from analyses with either nominal or reduced stiffnesses (reduced per Section 2.3). The threshold value for applying notional loads as minimum lateral loads is 1.5 in the former case, 1.7 in the latter case.

Interpretation of Minimum Lateral Load. Another change involves clarification of the conditions under which the minimum lateral load needs to be applied. The wording of the 2005 AISC *Specification* could be interpreted as requiring the application of notional loads only in "gravity-only" load combinations and not in combinations that include wind or other applied lateral loads, even if the applied lateral loads are less than the specified minimum. The present work makes it clear that every load combination must include lateral load at least equal to the minimum (see Section 2.2b(4)).

Adjustments to Stiffness. The 2005 AISC *Specification* requires analysis with reduced axial and flexural stiffnesses of members whose stiffnesses are considered to contribute to the lateral stability of the structure. It offers no explicit guidance, however, about member shear stiffnesses, diaphragm stiffnesses, column base rotational stiffnesses, etc. The present work takes the more conservative approach of applying the basic 0.8 reduction to all stiffnesses that contribute to the stability of the structure (see Section 2.3(1)).