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Direct Analysis Method – Application and Examples

December 9, 2021

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Course Description – Submitted for AIA CE Credit

Direct Analysis Method – Application and Examples
December 9, 2021

The direct analysis method first appeared in the 2005 AISC *Specification for Structural Steel Buildings* as an alternate way to design for stability. Transitioning from other stability methods or approaching stability design considerations for the first time can be intimidating. This webinar discusses the direct analysis method detailed in 2016 AISC *Specification*, Chapter C, Design for Stability, with a series of design examples. Topics will include a comparison of direct analysis method to the effective length method, second-order effects, and how to incorporate stability analysis in computer structural analysis models. Participants will gain the tools necessary to apply direct analysis in everyday practice.

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Learning Objectives – Submitted for AIA CE Credit

- Describe how loads are factored when using the direct analysis method.
- Explain how to consider geometric imperfections in an analysis model.
- Explain how to reduce member stiffness appropriately using the direct analysis procedure.
- Describe steps to take to ensure that a second order analysis is performed correctly.

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Direct Analysis Method – Application and Examples

David Landis, P.E.
Managing Principal
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Kansas City, Missouri

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Direct Analysis Method
Application and Examples

- What is it and why use it?
- How does it compare to the effective length method?
- Second-order effects
- Applying the Direct Analysis Method
- Examples

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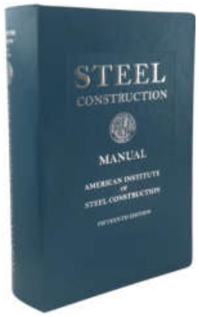
Direct Analysis Method



AISC 360-16

Direct Analysis

CHAPTER C

DESIGN FOR STABILITY

This chapter addresses requirements for the design of structures for stability. The direct analysis method is presented herein.

The chapter is organized as follows:

C1. General Stability Requirements
C2. Calculation of Required Strengths
C3. Calculation of Available Strengths

> **User Note:** Alternative methods for the design of structures for stability are provided in Appendices 1 and 7. Appendix 1 provides alternatives that allow for considering member imperfections and/or inelasticity directly within the analysis and may be particularly useful for more complex structures. Appendix 7 provides the effective length method and a first-order elastic method.

C1. 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: (a) flexural, shear and axial member deformations, and all other component and connection deformations that contribute to the displacements of the structure; (b) second-order effects (including P-Δ and P-δ effects); (c) geometric imperfections; (d) stiffness reductions due to inelasticity, including the effect of partial yielding of the cross section which may be accentuated by the presence of residual stresses; and (e) uncertainty in system, member, and connection strength and stiffness. All load-dependent effects shall be calculated at a level of loading corresponding to LRFD load combinations or 1.6 times ASD load combinations.

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

> **User Note:** See Commentary Section C1 and Table C-C1.1 for an explanation of how requirements (a) through (e) of Section C1 are satisfied in the methods of design listed in Sections C1.1 and C1.2.

1. Direct Analysis Method of Design

The direct analysis method of design is permitted for all structures, and can be based on either elastic or inelastic analysis. For design by elastic analysis, required strengths shall be calculated in accordance with Section C2 and the calculation of available strengths in accordance with Section C3. For design by advanced analysis, the provisions of Section 1.1 and Sections 1.2 or 1.3 of Appendix 1 shall be satisfied.

Specification for Structural Steel Buildings, July 7, 2016
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Sect. C2.] CALCULATION OF REQUIRED STRENGTHS **16.1**-23

2. Alternative Methods of Design Alternatives: ELM, FOM

The effective length method and the first-order analysis method, both defined in Appendix 7, are based on elastic analysis and are permitted as alternatives to the direct analysis method for structures that satisfy the limitations specified in that appendix.

C2. CALCULATION OF REQUIRED STRENGTHS

For the direct analysis method of design, the required strengths of components of the structure shall be determined from an elastic analysis conforming to Section C2.1. The analysis shall include consideration of initial imperfections in accordance with Section C2.2 and adjustments to stiffness in accordance with Section C2.3.

1. General Analysis Requirements

The analysis of the structure shall conform to the following requirements:

(a) 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 incorporate reductions in all stiffnesses that are considered to contribute to the stability of the structure, as specified in Section C2.3.

(b) The analysis shall be a second-order analysis that considers both P-Δ and P-δ effects, except that it is permissible to neglect the effect of P-δ on the response of the structure when the following conditions are satisfied: (1) the structure supports gravity loads primarily through nominally vertical columns, walls or frames; (2) 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, with stiffnesses adjusted as specified in Section C2.3) in all stories is equal to or less than 1.7; and (3) no more than one-third of the total gravity load on the structure is supported by columns that are part of moment-resisting frames in the direction of translation being considered. It is necessary in all cases to consider P-δ effects in the evaluation of individual members subject to compression and flexure.

> **User Note:** A P-Δ-only second-order analysis (one that neglects the effects of P-δ on the response of the structure) is permitted under the conditions listed. In this case, the requirement for considering P-δ effects in the evaluation of individual members can be satisfied by applying the B_1 multiplier defined in Appendix 8 to the required flexural strength of the member.

Use of the approximate method of second-order analysis provided in Appendix 8 is permitted.

(c) The analysis shall consider all gravity and other applied loads that may influence the stability of the structure.

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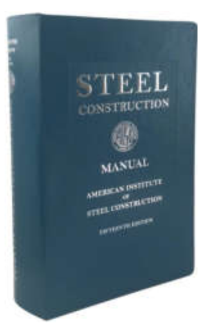
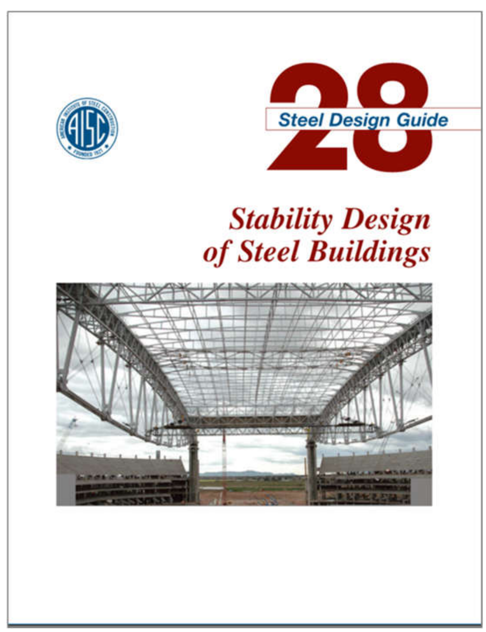
Direct Analysis Method



AISC 360-16
- Chapter C Design for Stability
- Chapter C commentary



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Why use the Direct Analysis Method?

- Primary method
- Versatile
- Applicable to all types of structural systems
- Captures internal structure forces more accurately
- Correct design of beams and connections providing column rotational restraint and stability
- No need to calculate *K*-factors and *K*-factor adjustments
- Applicable for all ranges of second-order effects ($\Delta_{2nd\ order} / \Delta_{1st\ order}$)
- Effective length method is limited (vertical columns, $\Delta_{2nd\ order}/\Delta_{1st\ order} < 1.5$)

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Second-Order Effects – What are they?

- Equilibrium satisfied on deformed geometry
- P-Δ effect (system)
- P-δ effect (member)

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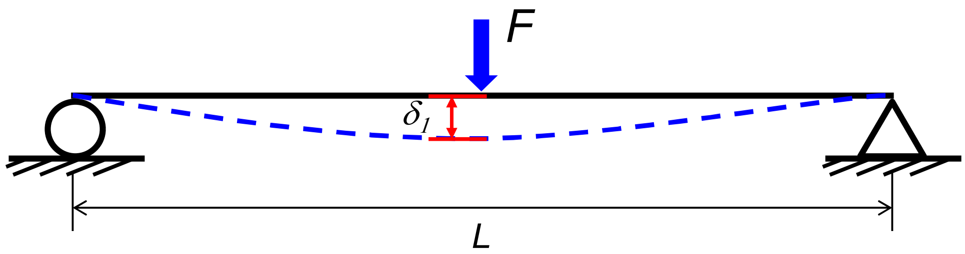
P-δ effect – What is it?

- Equilibrium satisfied on deformed geometry
- Member-level effect
- Axial load acting on member curvature produces additional moment

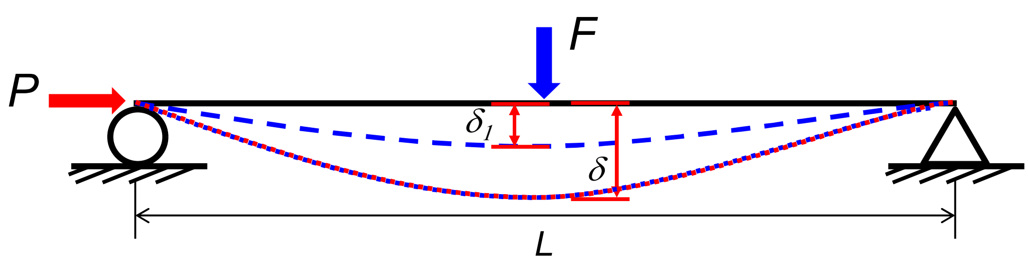


$M = FL/4$

13

P-δ effect – What is it?

- Equilibrium satisfied on deformed geometry
- Member-level effect
- Axial load acting on member curvature produces additional moment



$M = FL/4 + P\delta$

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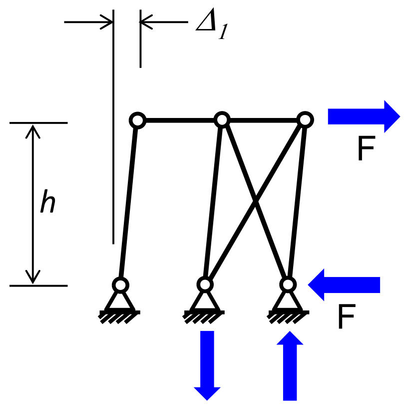
P-Δ effect – What is it?

- Equilibrium satisfied on deformed geometry
- System-level effect
- Gravity load acting on frame displacement produces thrust on system



$M_{OT} = Fh$

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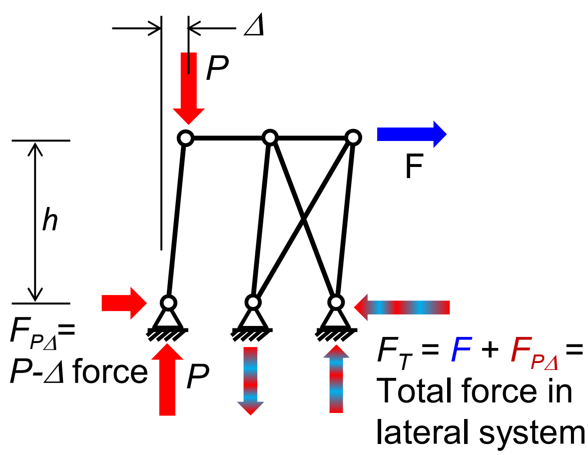
P-Δ effect – What is it?

- Equilibrium satisfied on deformed geometry
- System-level effect
- Gravity load acting on frame displacement produces thrust on system



$M_{OT} = Fh + P\Delta$

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Second-Order Effects – What are they?

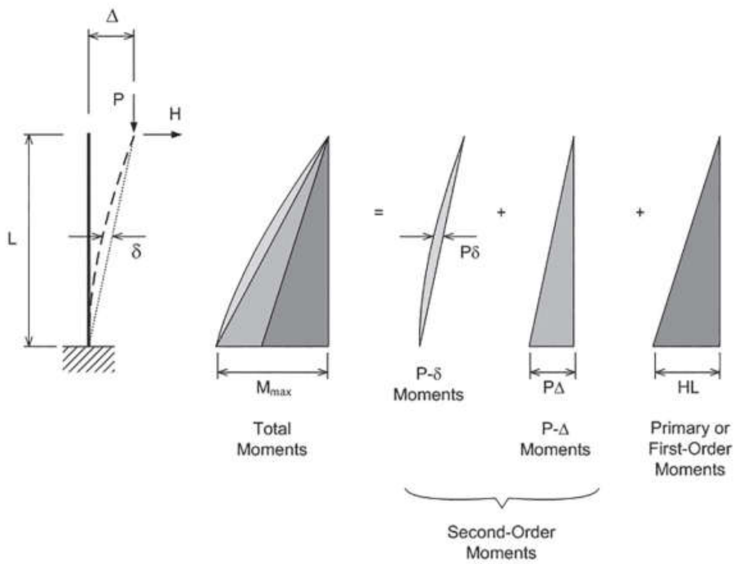

Figure from AISC Design Guide 28

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Second-Order Effects – What are they?

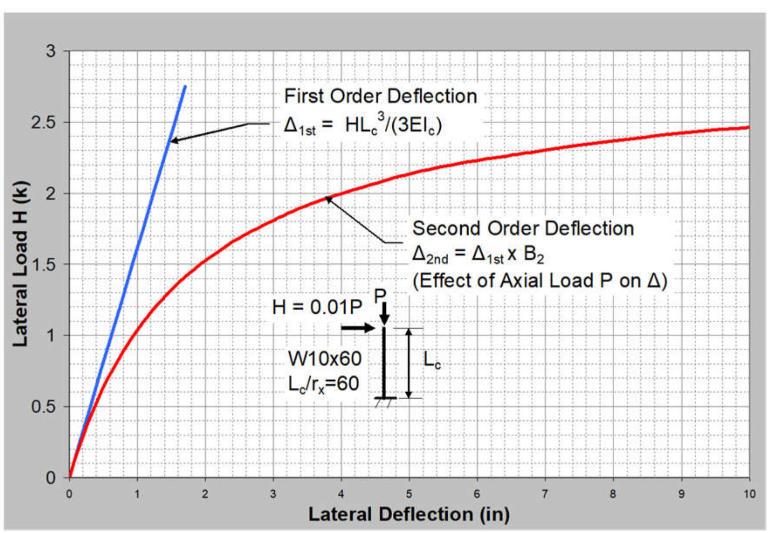

Figure from AISC Design Guide 28

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Direct Analysis Method versus Effective Length Method

	Effective Length Method (ELM)	Direct Analysis Method (DM)
Type of analysis	Rigorous Second-Order or Approx. Second-Order (B_1 & B_2)	Rigorous Second-Order or Approx. Second-Order (B_1 & B_2)
Member stiffness	Nominal EI & EA	Reduced EI & EA
Notional lateral loads	$0.002Y_i$ minimum	$0.002Y_i$ Minimum if $\Delta_{2nd}/\Delta_{1st} \leq 1.7$ Additive if $\Delta_{2nd}/\Delta_{1st} > 1.7$
Column effective length	Side-sway buckling analysis – determine K	$K = 1$
Limitations	Nominally vertical columns, $\Delta_{2nd}/\Delta_{1st} \leq 1.5$	No limitations

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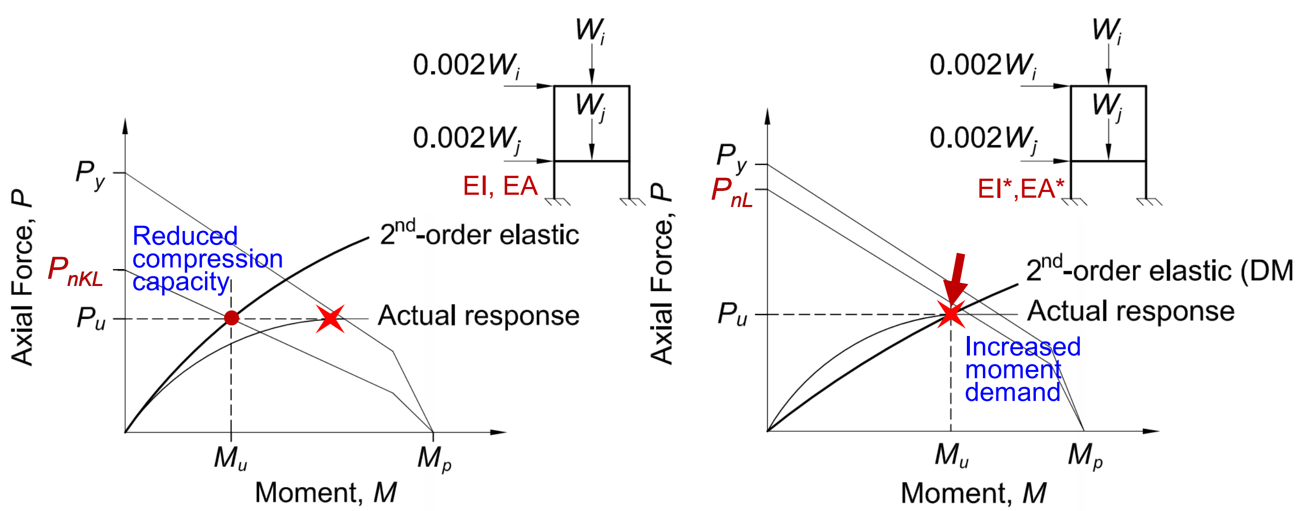
Direct Analysis Method vs. Effective Length Method



Effective Length Method Direct Analysis Method

Figures from AISC 360-16 Commentary

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Direct Analysis Method vs. Effective Length Method

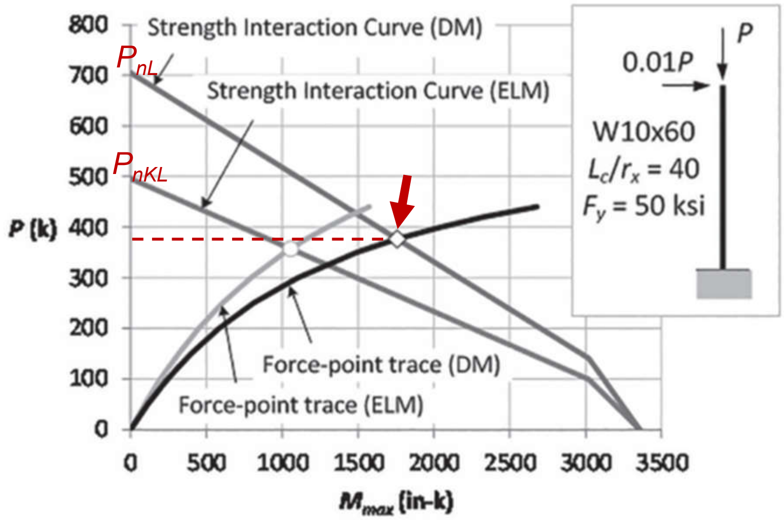

Figure from AISC Design Guide 28

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Direct Analysis Method Application

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both P-Δ and P-δ
- K=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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Direct Analysis Method Application

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both $P\text{-}\Delta$ and $P\text{-}\delta$
- K=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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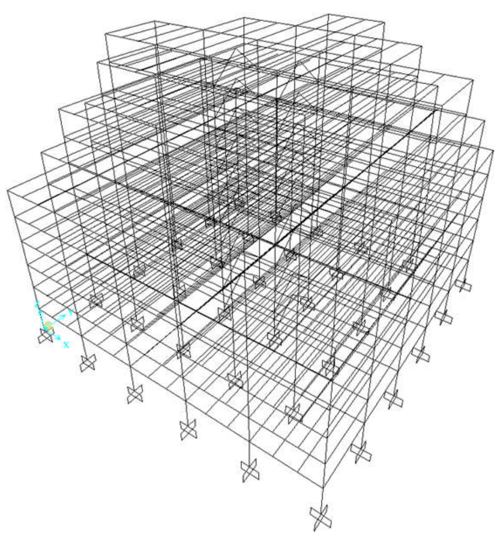
Direct Analysis Method Application

Accurately model frame behavior



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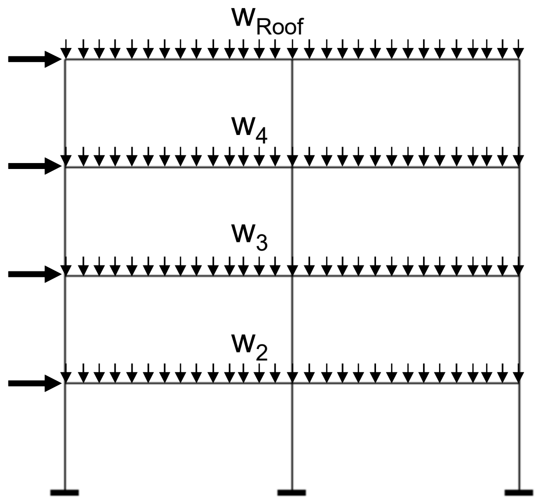
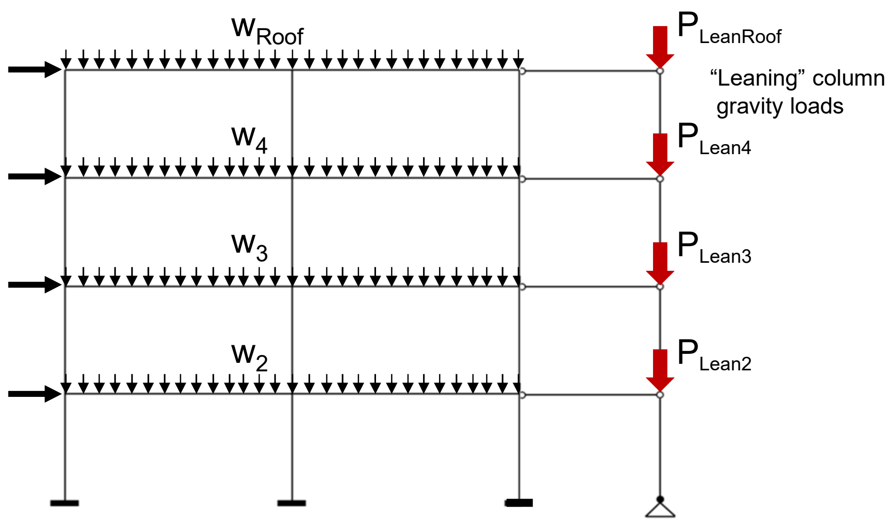
Direct Analysis Method Application

Accurately model frame behavior



Direct Analysis Method Application

Accurately model frame behavior



Direct Analysis Method Application

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both *P-Δ* and *P-δ*
- *K*=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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Direct Analysis Method Application

Factor loads (even for ASD!) (spec C2.1(d))

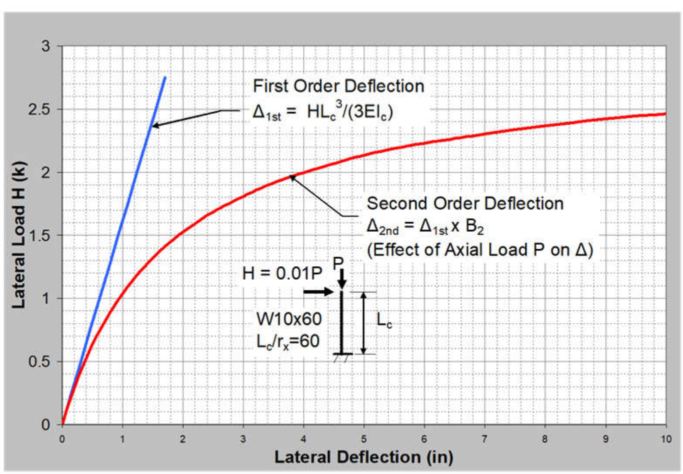

Figure from AISC Design Guide 28

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Direct Analysis Method Application

Factor loads (even for ASD)   (spec C2.1(d))

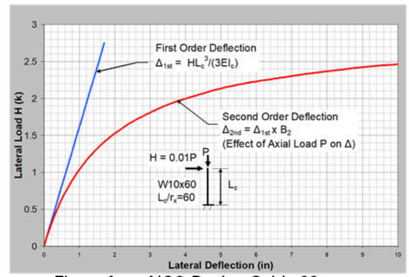

Figure from AISC Design Guide 28

- LRFD load combinations
- 1.6*ASD load combinations
  (then divide resulting forces by 1.6)
  (alternative approach: further reduce stiffness to $0.5EA$ and $0.5\tau_b{}'EI$ instead of factoring ASD loads – refer to C2.1 commentary)
- Include all loads that affect stability
  – Include “leaning” columns and all other destabilizing loads

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Direct Analysis Method Application

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both *P-Δ* and *P-δ*
- *K*=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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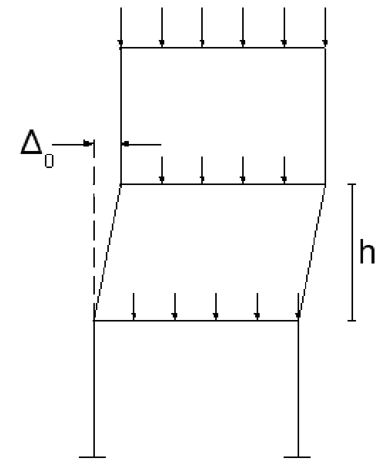
Buildings are not built perfect!

- Geometric imperfections affect column behavior
  - member out-of-straightness (δ_0)
  - story out-of-plumbness (Δ_0)
- Only δ_0 is included in column strength curves



Local story out-of-plumbness

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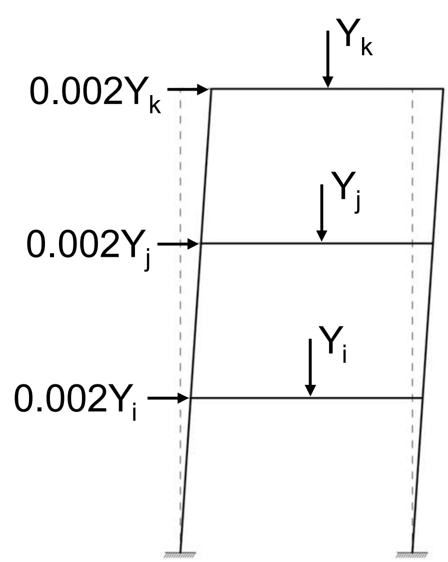
What is the Purpose of Notional Loads?

- Account for geometric imperfections, non-ideal conditions: h/500 → 0.002
- Lateral loads applied at each framing level
- Specified in terms of gravity loads at that level
- Applied in direction that adds to destabilizing effects
- Need not be applied if structure is modeled in an assumed out-of-plumb state



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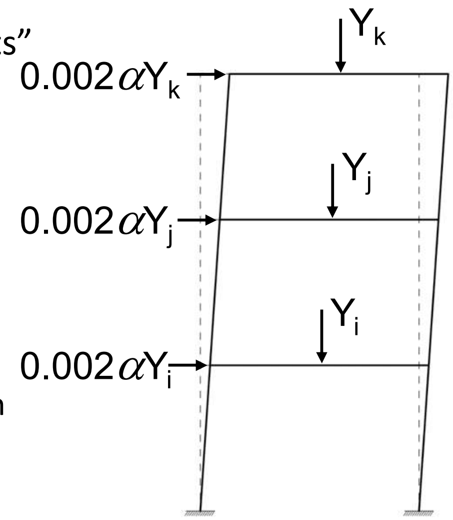
Direct Analysis Method Application

Consider initial imperfections (spec C2.2)

- Apply "notional loads" or "notional displacements"
- Notional Loads: (spec C2.2b)
  - $N_i = 0.002\alpha Y_i$
  - α = 1.0 (LRFD), 1.6 (ASD)
  - Y_i = gravity load applied at level *i*
  - N_i added to other lateral loads

    If $\Delta_{2nd\text{-}order}/\Delta_{1st\text{-}order}$ < 1.7 (reduced stiffness), or,
    If $\Delta_{2nd\text{-}order}/\Delta_{1st\text{-}order}$ < 1.5 (nominal stiffness), then it is permissible to omit N_i in combinations with other lateral loads



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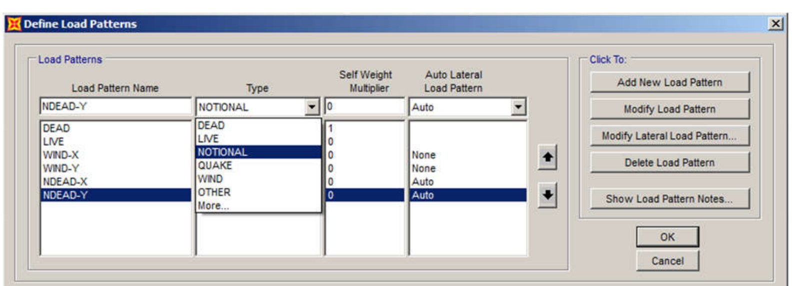
Direct Analysis Method Application

Consider initial imperfections (apply notional loads)

- Define Notional Loads and "auto" generate notional loads



(SAP2000 shown)

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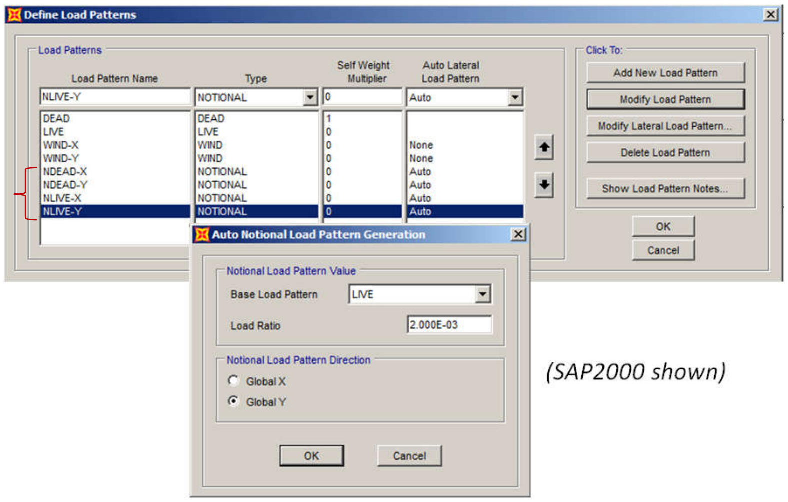
Direct Analysis Method Application

Consider initial imperfections (apply notional loads)

- Define Notional Loads and “auto” generate notional loads



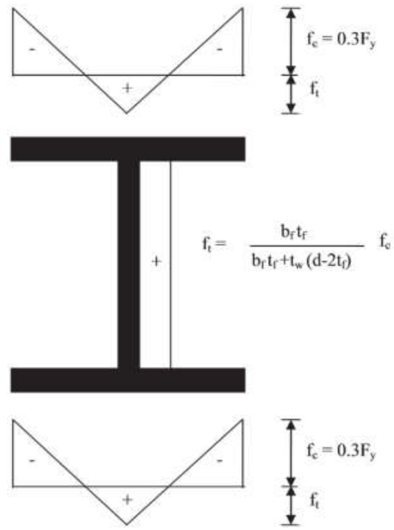
(SAP2000 shown)

Direct Analysis Method Application

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both P-Δ and P-δ
- K=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

Residual Stresses affect behavior of compression members

- Consequence of differential cooling rates during manufacturing
- Results in earlier initiation of yielding, thus affecting compressive strength
- Lowers member flexural strength and buckling resistance



Typical residual stress distribution

Figure from AISC Design Guide 28

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Direct Analysis Method Application

Reduce all stiffness that contributes to stability (spec C2.3)

- Axial and flexural stiffness reductions
- $EA^* = 0.8EA$
- $EI^* = 0.8\tau_b EI,\ \tau_b \leq 1.0$

$\tau_b = 1.0$ when $\alpha P_r / P_{ns} \leq 0.5$

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

$\alpha = 1.0$ for LRFD; $\alpha = 1.6$ for ASD

P_r = required axial compressive strength using LRFD or ASD combo's

$P_{ns} = F_y A_g$ for nonslender sections; $P_{ns} = F_y A_e$ for slender sections

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Direct Analysis Method Application

Reduce all stiffness that contributes to stability (spec C2.3)

$\tau_b = 1.0$ when $\alpha P_r / P_{ns} \leq 0.5$

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

$\alpha = 1.0$ for LRFD; $\alpha = 1.6$ for ASD

P_r = required axial compressive strength using LRFD or ASD combo's

$P_{ns} = F_y A_g$ for nonslender sections; $P_{ns} = F_y A_e$ for slender sections

- Permissible τ_b simplification: (spec C2.3(c))

$\tau_b = 1.0$ can be used if add'l notional loads $N_{iaddl} = 0.001\alpha Y_i$ are applied to all load combinations, including lateral load combo's

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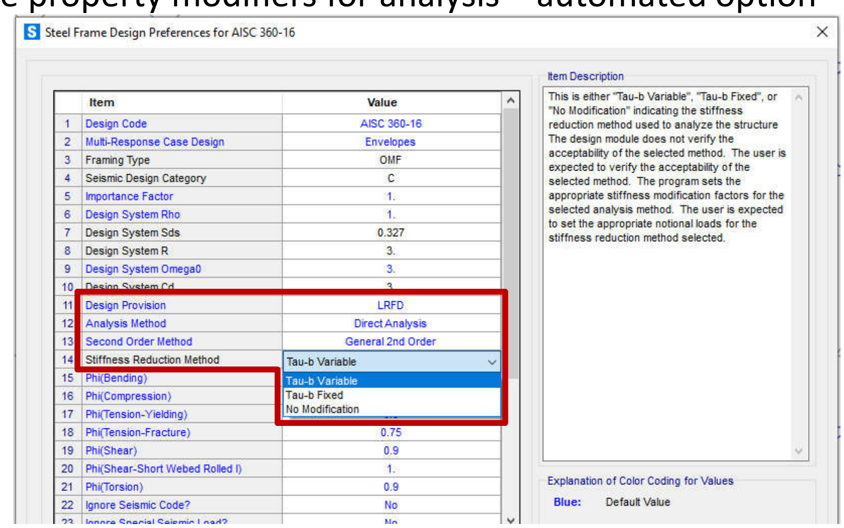
Direct Analysis Method Application

Reduce all stiffness that contributes to stability

- Define property modifiers for analysis – automated option



(SAP2000 shown)

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Direct Analysis Method Application

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both P-Δ and P-δ
- K=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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Direct Analysis Method Application

Second-order analysis – include both P-Δ and P-δ (spec C2.1)

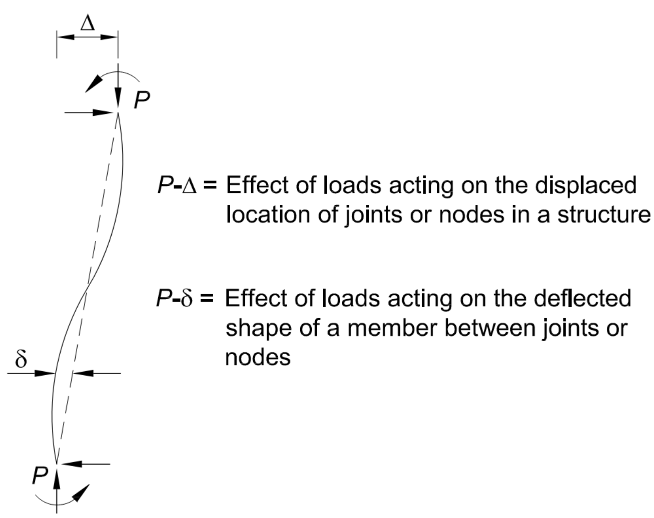

Figure from AISC 360-16 Commentary

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Direct Analysis Method Application

Second-order analysis – include both P-Δ and P-δ (spec C2.1)

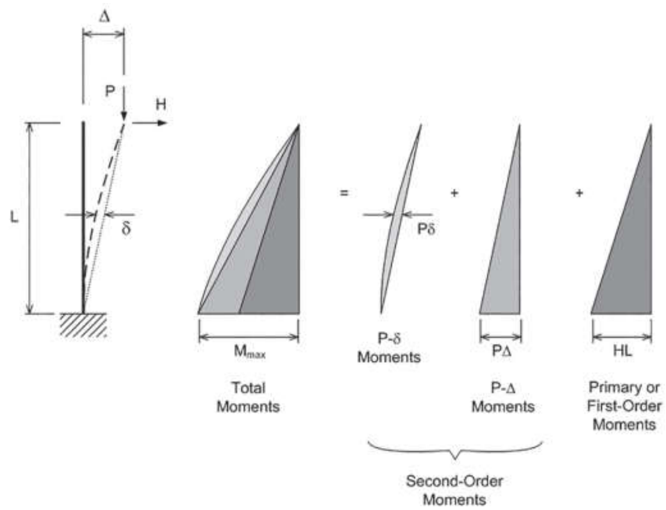

Figure from AISC Design Guide 28

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Direct Analysis Method Application

Second-order analysis – include both P-Δ and P-δ

- Know your software second-order analysis method:
  - Iterative incremental analysis method
  - Noniterative geometric stiffness method
  - Approximate second-order

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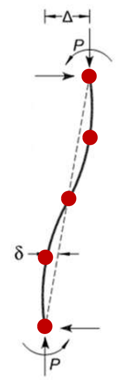
Direct Analysis Method Application

Software P-Δ and P-δ analysis capability notes:

- Iterative incremental analysis method
  - Most general and versatile method
  - Captures P-Δ directly
  - Captures P-δ effects directly by subdividing frame elements into multiple elements
  - No applicability limits



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Direct Analysis Method Application

Software P-Δ and P-δ analysis capability notes:

- Noniterative geometric stiffness method
  - Captures P-Δ directly
  - Generally, ***not*** able to directly capture P-δ effects through the analysis
  - Use B_1 amplifiers from Appendix 8 to amplify member moments from the P-Δ analysis to approximate the P-δ effect on member moments
  - Analysis method limits (C2.1(b)):
    - Nominally vertical columns
    - $\Delta_2/\Delta_1 \leq 1.7$ (using reduced stiffness)
    - $P_{mf}/P_{story} \leq 1/3$
    - Recommended B_1<1.2 for members having significant effect on overall structural response (C2.1 and Appendix 8 commentaries)

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Direct Analysis Method Application

Software P-Δ and P-δ analysis capability notes:

- Approximate second-order analysis method (Appendix 8)
  - First order analysis only – not able to directly capture P-Δ or P-δ
  - Use B_1 and B_2 amplifiers from Appendix 8 to approximate second-order effects
  - B_1 approximates P-δ effects
  - B_2 approximates P-Δ effects
  - Analysis limits (App. 8.1):
    - Nominally vertical columns
    - Recommended B_1<1.2 for members having significant effect on overall structural response (Appendix 8 commentary)

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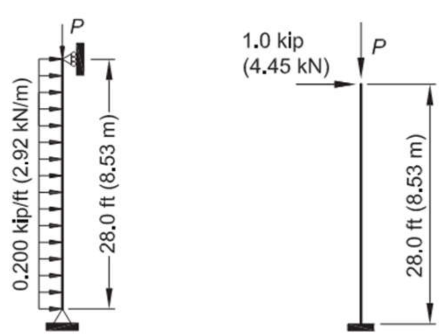
Direct Analysis Method Application

Software P-Δ and P-δ analysis capability notes:

- Software second-order analysis benchmark tests
  - Analysis benchmark problems in C2.1 commentary
  - Further discussion in Design Guide 28 Appendix D



Figures from AISC 360-16 Commentary

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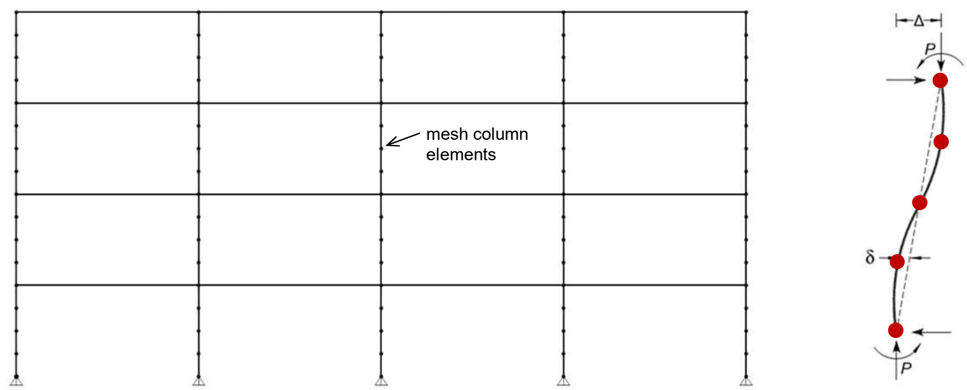
Direct Analysis Method Application

Second-order analysis – include both *P-Δ* and *P-δ* (spec C2.1)

- Internally subdivide compression elements to capture *P-δ* effects



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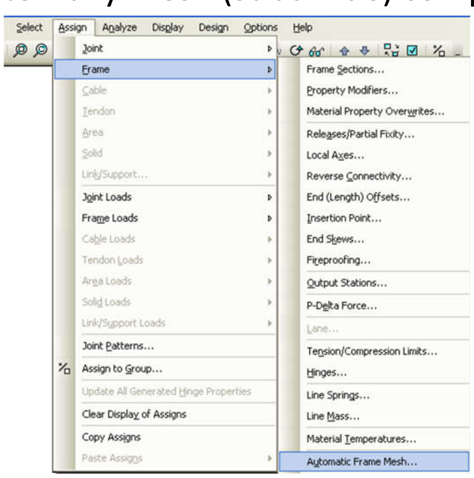
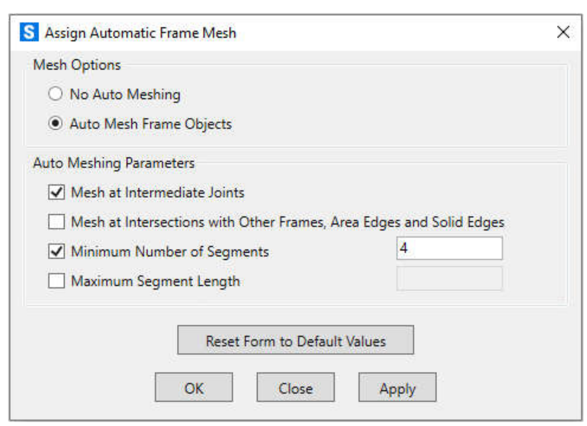
Direct Analysis Method Application

Second-order analysis – include both *P-Δ* and *P-δ*

- Internally mesh (subdivide) compression elements to capture *P-δ* effects





(SAP2000 shown)

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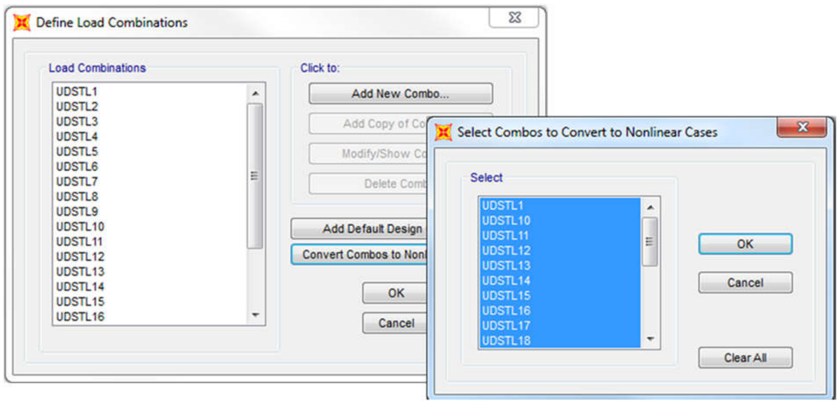
Direct Analysis Method Application

Second-order analysis – include both *P-Δ* and *P-δ*

- Generate ***nonlinear analysis cases*** for iterative second-order P-Delta analyses



(SAP2000 shown)

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Direct Analysis Method Application

Second-order analysis – include both *P-Δ* and *P-δ*

- Automated stiffness reduction factors to *EA* and *EI* are assigned only after design check is run (*SAP2000, ETABS*)
- Iterate as necessary
- Check $\Delta_{2nd\ order}/\Delta_{1st\ order}$ ratio
  - If $\Delta_{2nd\ order}/\Delta_{1st\ order} \leq 1.7$ (reduced stiffness) or 1.5 (nominal stiffness), then N_i not required in lateral combinations (N_i only *required* in gravity combinations)
  - If $\Delta_{2nd\ order}/\Delta_{1st\ order} > 1.7$ (reduced stiffness) or 1.5 (nominal stiffness), then include N_i in ***all*** load combinations
  - Simplification: include N_i in all load combinations

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Direct Analysis Method Application

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both P-Δ and P-δ
- K=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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Direct Analysis Method Application

K=1 for member design (spec C3)

- $K = 1 \rightarrow L_c = L$
- Effective length = actual unbraced length
- No more K-factors or K-factor adjustments!

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Direct Analysis Method Application

Member design

- For ASD, divide resulting analysis forces by 1.6 (spec C2.1(d))
  - P, M, V = Analysis{1.6*ASD}/1.6
- Caution: Rerun analysis and recheck designs if member sizes or loads change

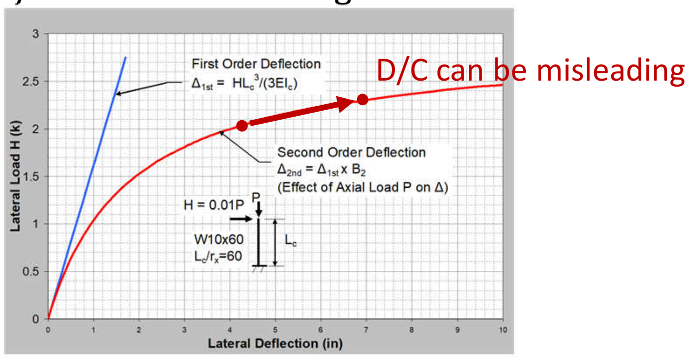

Figure from AISC Design Guide 28

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Direct Analysis Method Application

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both *P-Δ* and *P-δ*
- *K*=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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Direct Analysis Method Application

Use nominal (unreduced) stiffness for building periods and serviceability checks

- Reduced stiffness is ONLY used in the strength analyses
- Determine building periods using nominal (unreduced) stiffness
- All serviceability checks use the unreduced stiffness
  - Check drifts for wind and seismic using nominal (unreduced) stiffness
  - Check vibration using nominal (unreduced) stiffness

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Direct Analysis Method Application Summary

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analyses – include both P-Δ and P-δ
- K=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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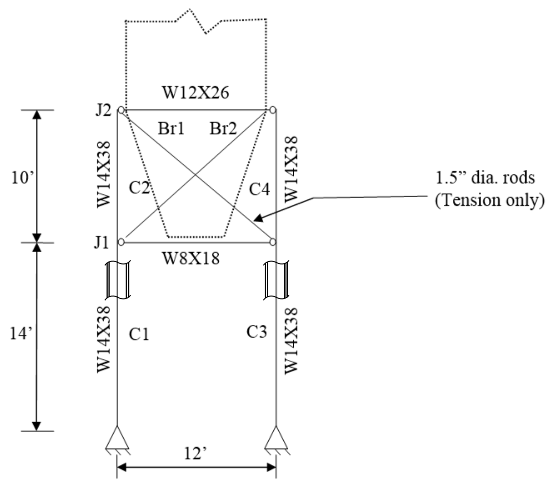
Direct Analysis Method Application

Examples

Example 1: Grain Storage Bin

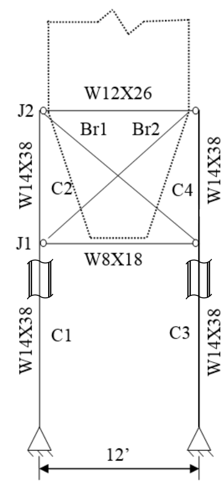
Representative of an elevated structure where stability effects are accentuated by the position of most weight at top



Using LFRD, check adequacy of the given steel frame for the given loads

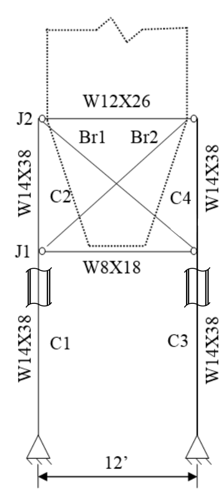
Example 1: Grain Storage Bin
Loads, material properties, definitions, and design requirements

- Bin sits on top of frame shown producing the following nominal loads:
  - Grain load: Vertical load, P_G = 60 kips at top of each column
  - Dead load: Vertical load, P_D = 5 kips at top of each column
  - Wind load: Total Horizontal Force = 11.2 kips with centroid 9 ft above top of frame
    - Horizontal load, W_H = 5.6 kips at top of each column (ΣW_H = 11.2 kips)
    - Vertical load, W_V = 11.2 x 9/12 = +/-8.4 kips at top of each column
- A992 steel for wide flange shapes, A36 steel rods
- Use Δ_o/H = 1/500 = 0.002 initial out-of-plumbness
- No interstory drift requirement under nominal wind and gravity loads



Example 1: Grain Storage Bin
Connection types

- All columns are oriented for strong axis bending in the plane shown. The columns are braced out-of-plane at each joint.
- All lateral load resistance in the upper tier is provided by the tension only rod bracing.
- All lateral load resistance in the lower tier is provided by the flexural resistance of the columns.
- Tension rods are assumed as pinned connections using a standard clevis and pin
- Horizontal beams within the braced frame portion have bolted double angle shear connections.



Example 1: Grain Storage Bin
Notional Loads and Load combinations

NDead, NGrain: Notional lateral loads = 0.002D and 0.002Grain
(the grain load is handled as a dead load by engineering judgment)

Assume the following LRFD load combinations:

Comb1 = 1.4(D + Grain) + 1.4(NDead + NGrain)
Comb2 = 1.4(D + Grain) – 1.4(NDead + NGrain)
Comb3 = 1.2(D + Grain) + 1.0W
Comb4 = 1.2(D + Grain) – 1.0W
Comb5 = 0.9D + 1.0W
Comb6 = 0.9D – 1.0W

Notional lateral loads combined only with gravity loads

Because of symmetry Comb1 and Comb2, and Comb3 and Comb4 will produce the same results. By inspection, Comb5 and Comb6 are not critical.

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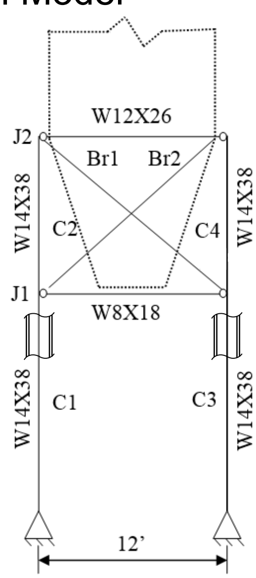
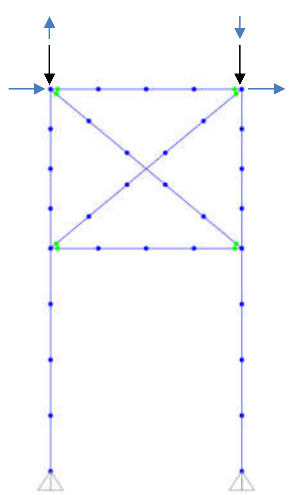
Example 1: Grain Storage Bin
FEM Model





Analysis model:
- Add notional loads to gravity cases
- Mesh frame members
- Reduce member stiffness
  - $0.8EA$
  - $0.8\tau_b EI$
    - First assume $\tau_b = 1.0$, then check later during design checks
- Run linear (first-order) analysis first
- Generate nonlinear P-Delta LRFD factored analysis cases
- Run iterative second-order analyses

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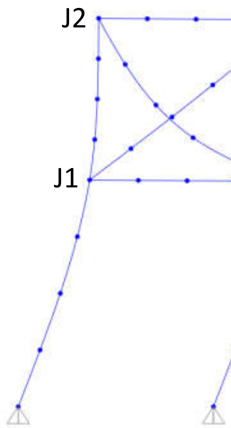
Example 1: Grain Storage Bin
Second-order effects check

Verify if the ratio of second-order to first-order story drift ≤ 1.7 (w/ reduced properties) at each level of the frame for all load combinations



Story	Combination	Story Drift 1st order	Story Drift 2nd order	$\Delta_{2nd}/\Delta_{1st}$
J2-J1	Comb1	0.052	0.056	1.08
J2-J1	Comb3	0.294	0.332	1.13
J1	Comb1	0.187	0.236	1.26
J1	Comb3	2.169	2.636	1.22

Since $\Delta_{2nd}/\Delta_{1st}$ ≤ 1.7 (w/ reduced properties), Notional loads can be applied in gravity-load combinations only; not required in combination with lateral loads.

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Example 1: Grain Storage Bin
Property modifiers for strength analysis only (spec C2.3)

Axial stiffness = 0.8*EA*

Flexural stiffness = $0.8\tau_b EI$

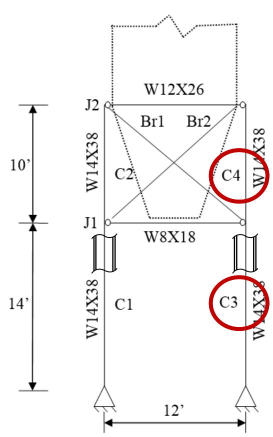
For Columns C3 and C4 in Comb3:

$$\frac{\alpha P_r}{P_{ns}} = \frac{1P_r}{F_y A_g} = \frac{113^{\text{k}}}{50\text{ ksi} * 11.2\text{ in}^2} = 0.2 < 0.5 \qquad \text{(C2-2a)}$$

$\therefore \tau_b = 1.0$

By inspection, for columns C1 and C2, $\tau_b = 1.0$ also

Our earlier assumption that $\tau_b = 1$ is now confirmed



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Example 1: Grain Storage Bin
Second-order analysis results and strength checks

Load Combination		C1	C2	C3	C4	Br1	Br2
Comb1	P_r	-91.8	-91.8	-93.8	-92.8	0.0	1.6
	M_r	45.2	45.2	44.6	44.6	2.4	2.4
	$øP_n$	213.4	324.1	213.4	324.1	79.5	79.5
	$øM_n$	2767.5	2767.5	2767.5	2767.5	0	0
	Interaction*	0.45	0.30	0.45	0.30	0.000	0.044
Comb2	P_r	-93.8	-92.8	-91.8	-91.8	1.6	0.0
	M_r	44.6	44.6	45.2	45.2	2.4	2.4
	$øP_n$	213.4	324.1	213.4	324.1	79.5	79.5
	$øM_n$	2767.5	2767.5	2767.5	2767.5	0	0
	Interaction*	0.45	0.30	0.45	0.30	0.04	0.00
Comb3	P_r	-44.8	-70.3	-112.9	-112.7	0.0	40.1
	M_r	1161.4	1161.4	1136.6	1136.6	2.0	2.0
	$øP_n$	213.4	324.1	213.4	324.1	79.5	79.5
	$øM_n$	2767.5	2767.5	2767.5	2767.5	0	0
	Interaction*	0.58	0.59	0.89	0.71	0.00	0.62
Comb4	P_r	-112.9	-112.7	-44.8	-70.3	40.1	0.0
	M_r	1136.6	1136.6	1161.4	1161.4	2.0	2.0
	$øP_n$	213.4	324.1	213.4	324.1	79.5	79.5
	$øM_n$	2767.5	2767.5	2767.5	2767.5	0	0
	Interaction*	0.89	0.71	0.58	0.59	0.62	0.00

K = 1 for all members in strength calculations (spec C3)

*Chapter H interaction Equations (H1-1a), (H1-1b)

(a) When $\dfrac{P_r}{P_c} \geq 0.2$

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

(b) When $\dfrac{P_r}{P_c} < 0.2$

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

Demand/Capacity < 1, OK

Member Strength Check – Column C3 (Comb3)

Calculations for Column C3:

- K = 1; $KL_x = L_x$ = 14 ft; $KL_y = L_y$ = 14 ft
- L_y/r_y = 14x12/1.55 = 108
- F_e = 24.4 ksi (Eqn E3-4, K=1)
- ϕF_{cr} = 19.1 ksi (Eqn E3-2)
- ϕP_n = 19.1 ksi x 11.2 in^2 = 213 kips (Eqn E3-1)
- C_b = 1.67 (linear moment diagram with zero moment at one end)
- L_b = 14 ft, $\phi M_n = C_b$ x moment from Table 3-10 ≤ ϕM_p
- ϕM_n = 1.67 x 162 kip-ft = 271 k-ft > ϕM_p = 231 k-ft
- ϕM_n = 231 k-ft

Member Strength Check – Column C3 (Comb3)

Calculations for Column C3, continued:

- P_r = 112.9 kips and M_r = 1136.6 kip-in = 94.7 kip-ft
- $P_r/\phi P_n$ = 112.9/213 = 0.53 > 0.2; use interaction eqn H1-1a:

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

112.9/213 + 8/9 (94.7/231) = 0.89 < 1 → OK

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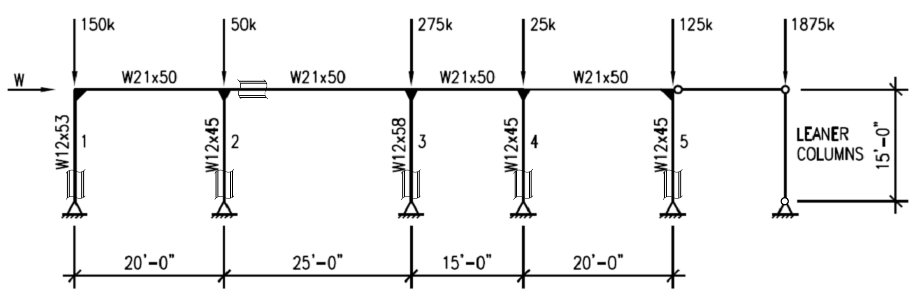
Example 2: Unsymmetrical Moment Frame Building

Check each column for conformance to 2016 AISC Specification using LRFD and the Direct Analysis Method.



This problem was originally worked by Baker (1997) and later by Geschwindner (2002) to demonstrate the challenges in determining the effective length factor accurately for an ELM solution by the 1999 LRFD Specification.

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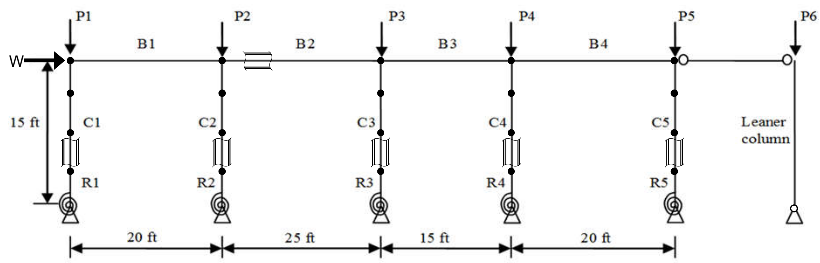
Example 2: Unsymmetrical Moment Frame Building
Material properties, definitions, and design requirements

- All columns are subjected to strong axis bending in the plane shown
- Assume all column bases have a rotational spring stiffness β = 6*EI*/10*L* (derived for "pin base" at foundation using *G*=10)
- Drift (Δ/*H*) limit under 10-yr wind load = 1/400
- A992 steel



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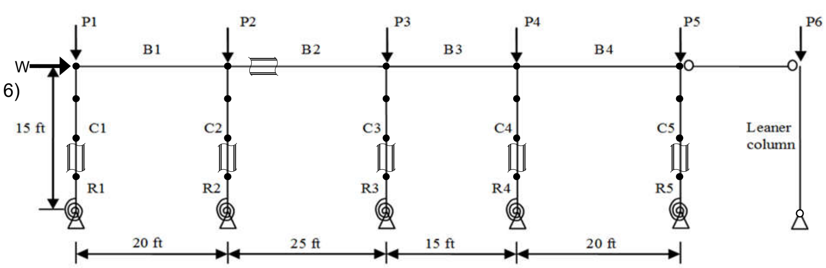
Example 2: Unsymmetrical Moment Frame Building
Loads



Load	Factored Gravity Load (kips) (1.2D + 1.6L)	Unfactored Dead Load D (kips)	Unfactored Live Load L (kips)
P1	150	75	37.5
P2	50	25	12.5
P3	275	137.5	68.75
P4	25	12.5	6.25
P5	125	62.5	31.25
P6	1,875	937.5	468.75

Rotational Spring Stiffness (β = 6*EI*/10*L*) at Foundation

Support	Stiffness (k-in/rad)
R1	41,083
R2	33,640
R3	45,917
R4	33,640
R5	33,640

Notional loads = $N_i = 0.002Y_i$

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Example 2: Unsymmetrical Moment Frame Building
Analysis

- Notional Lateral Loads $N_i = 0.002Y_i$
- Property modifiers for the analysis only
  - Axial stiffness = $0.8EA$
  - Flexural stiffness = $0.8\tau_b EI$
  - Assume $\tau_b = 1.0$ (check assumption later)
- Perform a second-order elastic analysis including P-Δ and P-δ effects, using reduced member stiffness and subdivided columns

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Example 2: Unsymmetrical Moment Frame Building
Load combinations and second order effect

- ASCE 7 load combinations:

Comb2a = $1.2D + 1.6L + 1.2NDead + 1.6NLive$	Notional lateral loads combined with all loads
Comb2b = $1.2D + 1.6L - 1.2NDead - 1.6NLive$	
Comb4a = $1.2D + 1.0L + 1.2NDead + 1.0NLive + 1.0W$	
Comb4b = $1.2D + 1.0L - 1.2NDead - 1.0NLive - 1.0W$	

$NDead = 0.002D$ notional lateral load,

$NLive = 0.002L$ notional lateral load

- The check $\Delta_{2nd}/\Delta_{1st}$ vs. 1.7 is determined using the reduced stiffness
- From the second-order analysis results, $\Delta_{2nd}/\Delta_{1st} = 1.89 > 1.7$ (Comb2a & 2b)
- Therefore, the notional lateral loads are *applied additively to **all** load combinations.* (spec C2.2b(a), C2.2b(d))

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Example 2: Unsymmetrical Moment Frame Building
Second-order analysis results

Load Combination		C1	C2	C3	C4	C5
COMB2a	P_r (kips)	-153	-53	-275	-30	-126
	$M_{r,bot}$ (k-in)	82	68	98	74	62
	$M_{r,top}$ (k-in)	-255	-219	-338	-284	-152
COMB4a	P_r (kips)	-121	-50	-228	-27	-115
	$M_{r,bot}$ (k-in)	361	317	425	323	296
	$M_{r,top}$ (k-in)	-1044	-1074	-1358	-1151	-845
COMB4b	P_r (kips)	-136	-42	-237	-25	-103
	$M_{r,bot}$ (k-in)	-365	-328	-428	-325	-310
	$M_{r,top}$ (k-in)	1017	1140	1302	1117	937

- Check original assumption of τ_b = 1.0: $(\tau_b = 1.0 \text{ when } \alpha P_r/P_{ns} \leq 0.5)$ (C2-2a)
- Check column with the highest compressive stress: Column C3
  - P_r = 275 kips and A = 17.0 in^2
  - $P_{ns} = F_y A_g$ = 50 ksi x 17.0 in^2 = 850 kips
  - $\alpha P_r/P_{ns}$ = 1x275 kips/850 kips = 0.33 < 0.5; Therefore, confirmed that τ_b = 1.0

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Example 2: Unsymmetrical Moment Frame Building
Strength checks

- K = 1 for all members in strength calculations
- Representative calculations for Column C3 (W12x58):
- Governing combination is Comb4a where P_r = 228 kips (compression) and M_r = -1,358 k-in (M_{top} = -1,358k-in, and M_{bot} = 425 k-in)
- K = 1; $KL = L$ = 15ft x 12 = 180 in
- KL/r_y = 180/2.51 = 71.71 < $4.71\sqrt{(E/F_y)}$ = 113.4
- $F_e = \pi^2 E/(KL/r_y)^2$ = 55.65 ksi (Eqn E3-4, K=1)
- $F_{cr} = [0.658^{(Fy/Fe)}]F_y$ = 34.33 ksi (Eqn E3-2)
- ϕP_n = 0.9 x 34.33 ksi x 17.0 in^2 = 525 kips

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Example 2: Unsymmetrical Moment Frame Building
Strength checks

- W12x58 column, L_b = 15 ft
- M_r at top = -1,358 k-in
- M_r at bottom = 425 k-in
- $C_b = \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C} = 2.11$ (Eqn F1-1)
- ϕM_n = 3,888 k-in using C_b= 2.11 (Eqn F2-2)

- Interaction Equation (H1-1a):
  228/525 + (8/9)(1,358/3,888) = 0.75 < 1 OK

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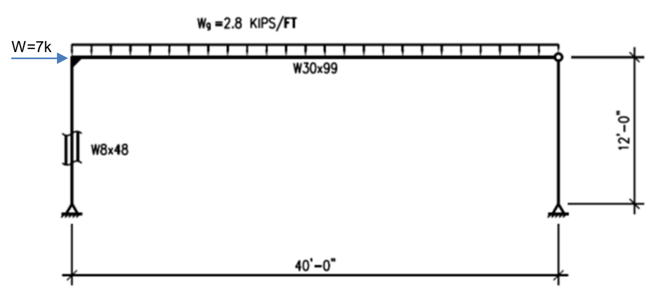
Example 3: Market Shed Building – Simple Moment Frame

Using ASD, check existing frame for dead, live and wind load combinations



This problem is taken from LeMessurier (1977)

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Example 3: Market Shed Building – Simple Moment Frame

Loads, material properties, definitions, and design requirements

- Frames @ 35 ft on center
- Columns braced out of plane at the roof level
- A992 steel
- Δ_o/H = 1/500 = 0.002 out-of-plumbness
- Limit lateral deflection Δ = 1" under a 50-yr wind load and total gravity loads ($D+L$) using second-order analysis

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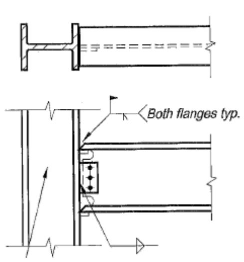
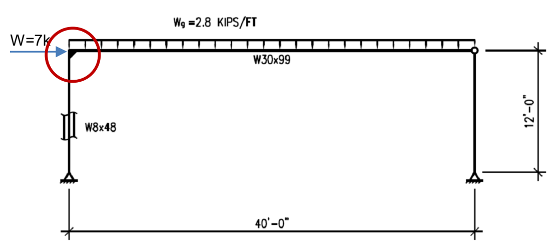
Example 3: Market Shed Building – Simple Moment Frame

Connection types

- All lateral load resistance is provided by the moment connection between the left-hand column and the roof beam
- Assume that this moment connection is a field welded complete penetration beam flange to column flange welded connection with a shear tab bolted splice.





- The right-hand column to beam connection is a bolted simple shear connection, assumed to be pinned

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Example 3: Market Shed Building – Simple Moment Frame

Loads

- Dead load = 0.7 k/ft uniform line load
- Live load = 2.1 k/ft uniform line load
- Wind load = 7.0 kips (ASCE 7-16)
- Self-weight = 4.71 kips
- Notional lateral loads $N_i = 0.002\alpha Y_i$, α=1.6 for ASD:
  – *NDead* = 0.002 x α x (0.7 k/ft x 40 ft + 4.71 kips) = 0.0654 α kips
  – *NLive* = 0.002 x α x 2.1 k/ft x 40 ft = 0.168 α kips

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Example 3: Market Shed Building – Simple Moment Frame

ASD load combinations (spec C2.1(d)):

Member design forces are obtained by analyzing the structure for 1.6 times ASD load combinations and then dividing the analysis results by 1.6.

Comb1a = 1.6(*D* + *SelfWt* + *NDead*)
Comb1b = 1.6(*D* + *SelfWt* – *NDead*)
Comb3a = 1.6(*D* + *SelfWt* + *NDead* + L_r + *NLive*)
Comb3b = 1.6(*D* + *SelfWt* - *NDead* + L_r – *NLive*)
Comb5a = 1.6(*D* + *SelfWt* + 0.6*W*)
Comb5b = 1.6(*D* + *SelfWt* - 0.6*W*)
Comb6a = 1.6(*D* + *SelfWt* + 0.75L_r + 0.75*0.6*W*)
Comb6b = 1.6(*D* + *SelfWt* + 0.75L_r - 0.75*0.6*W*)
Comb7a = 1.6(0.6*D* + 0.6*SelfWt* + 0.6*W*)
Comb7b = 1.6(0.6*D* + 0.6*SelfWt* – 0.6*W*)

Notional lateral loads combined only with gravity loads

NDead and *NLive* are minimum lateral loads assumed to apply to the gravity-only load combinations only. This assumption will be checked later.

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Example 3: Market Shed Building – Simple Moment Frame

Property modifiers for factored analysis

- Section properties are reduced for strength analysis:
  - Axial stiffness = 0.8*EA*
  - Flexural stiffness = $0.8\tau_b EI$
  - Assume τ_b=1.0 (check assumption later)

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Example 3: Market Shed Building – Simple Moment Frame

Analysis

- Column elements are meshed to capture the P-δ effects
- Direct Analysis is performed using the *reduced* stiffness at *1.6 times* the ASD load combination level using second-order analyses
- Check lateral drift ratio for application of notional lateral loads (using reduced stiffness)
  - $\Delta_{2nd\ order}/\Delta_{1st\ order} < 1.7$ (using reduced stiffness)
  - Therefore, permissible to apply notional lateral loads only in the gravity-only load combinations

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Example 3: Market Shed Building – Simple Moment Frame

Serviceability drift limits

- Serviceability check done using nominal (unreduced) stiffness and unfactored service loads
- Second-order drift = 2.83” > 1” (using nominal stiffness)
  No Good – Frame must be stiffened
- W36x150 beam and W18X97 column required for drift control (determined from trial-and-error analysis)

Rerun factored strength analysis with updated member sizes and with reduced stiffness properties

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Example 3: Market Shed Building – Simple Moment Frame

Second-order analysis results (with revised member sizes and reduced stiffness)

ASD Load Combination Level (after dividing results by 1.6)

Load Combination		Direct Analysis Method	
		COL_1	BEAM
Comb1	P_r (kips)	-17.0	0.1
	M_r (k-in)	-23.3	2052.6
Comb3	P_r (kips)	-58.6	**0.7**
	M_r (k-in)	-194.2	**7177.2**
Comb5a	P_r (kips)	-15.7	2.2
	M_r (k-in)	-628.1	2365.1
Comb5b	P_r (kips)	-18.3	-2.1
	M_r (k-in)	602.0	1740.7
Comb6a	P_r (kips)	**-47.3**	2.0
	M_r (k-in)	**-581.5**	6109.4
Comb6b	P_r (kips)	-49.3	-1.3
	M_r (k-in)	369.6	5637.6
Comb7a	P_r (kips)	-8.9	2.1
	M_r (k-in)	-615.6	1550.3
Comb7b	P_r (kips)	-11.5	-2.1
	M_r (k-in)	606.3	921.8

$\alpha P_r / P_{ns}$ = 1.6(58.6)/($A_g F_y$) = 0.07 < 0.5, thus confirmed τ_b = 1.0

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Example 3: Market Shed Building – Simple Moment Frame

Strength checks (with revised member sizes)

- *K* = 1 for all members in the capacity calculations
- Capacity calculations are performed using nominal section properties
- Capacity calculations are not presented here but use the same process of applying the capacity calculations and interaction equations
- The new sizes easily work because drift controlled the design of the frame

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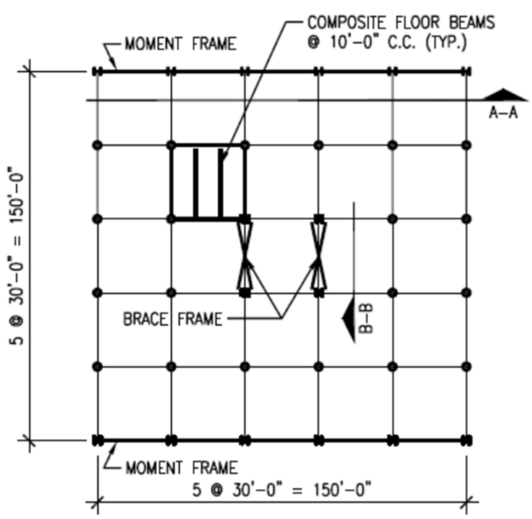
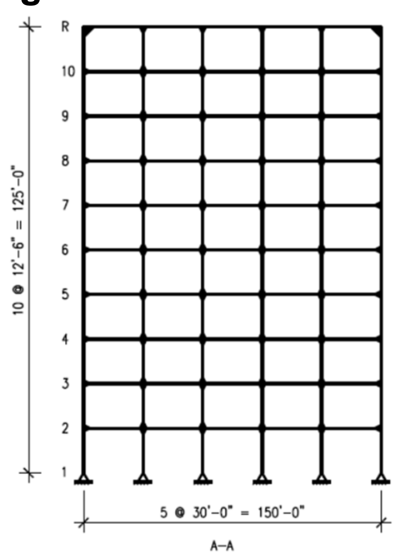
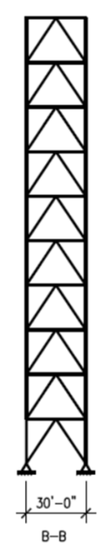
Example 4: 10-story office building



PLAN



MOMENT FRAME



BRACED FRAME

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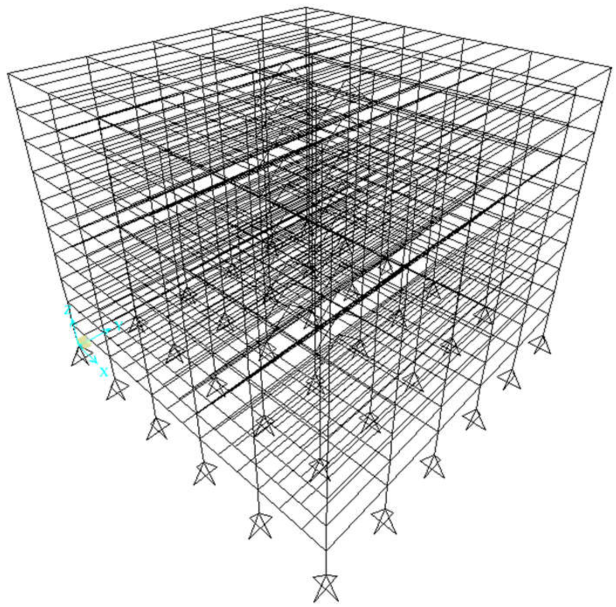
Example 4: 10-story office building

3-D Model



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Example 4: 10-story office building

Gravity Loads

Floor
- Structure weight = 65 psf (plus the vertical framing)
- Superimposed dead loads = 25 psf
- Live Load = 100 psf (reducible)

Roof
- Same dead loads as Floor
- Live Load = 30 psf (unreduced)

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Example 4: 10-story office building

Live Load Reduction

- Applied according to IBC 2018 Section 1607.11

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

- K_{LL} = Live load element factor
  = 4 for columns – interior, exterior w/o cantilever slabs
  = 2 for beams – interior, edge w/o cantilever slabs

- For beams of moment frames,
  L = 100 psf x [0.25 + 15 / (2 x 15 x 30)$^{0.5}$] = 75 psf

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Example 4: 10-story office building

Live Load Reduction – Interior Columns

LEVEL	Interior Column K_{LL} = 4 Tributary area of reducible load SF	ΣSF	LLR	With 100 psf design LL: P Live kips	ΣP Live kips	ΣP Live × LLR kips	With 75 psf LL: P Live kips	ΣP Live kips	ΣP Up Live kips	Correction in Load: P Up per Level (kips) for Column LLR
ROOF	0	0	1	0	0	0	0	0	0	0
LEVEL10	900	900	0.50	90	90	45	67.5	67.5	22.5	22.5
LEVEL9	900	1800	0.43	90	180	76.8	67.5	135	58.2	35.7
LEVEL8	900	2700	0.40	90	270	108	67.5	203	94.5	36.3
LEVEL7	900	3600	0.40	90	360	144	67.5	270	126	31.5
LEVEL6	900	4500	0.40	90	450	180	67.5	338	158	31.5
LEVEL5	900	5400	0.40	90	540	216	67.5	405	189	31.5
LEVEL4	900	6300	0.40	90	630	252	67.5	473	221	31.5
LEVEL3	900	7200	0.40	90	720	288	67.5	540	252	31.5
LEVEL2	900	8100	0.40	90	810	324	67.5	608	284	31.5

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Example 4: 10-story office building

Column Gravity Design – Interior Columns

Column Label:	B-2				Area Service Loads				Cumulative Factored Loads					Column		
No.	Fl. Label	Fl. Height (ft)	F_y of Col.	KLL	Load Type No.	Trib. Area (ft^2)	Load Type No.	Trib. Area (ft^2)	Dead Load (kips)	S-Dead Load (kips)	Reducible Live Load (kips)	Unreducible Live Load	Total Load (kips)	Column Size	Col. Cap. (kips)	Pu/ϕPn
															Col. Designer	
10	Roof	12.5	50	4	3	900	2	900	81.0	16.2	0.0	43.2	140.4	W14X30	189.8	0.740
9	10	12.5	50	4	1	900	2	900	163.1	32.4	72.0	43.2	310.7	W14X43	357.7	0.868
8	9	12.5	50	4	1	900	2	900	245.0	48.6	122.9	43.2	459.7	W14X61	612.4	0.751
7	8	12.5	50	4	1	900	2	900	327.0	64.8	172.8	43.2	607.8	W14X68	685.8	0.886
6	7	12.5	50	4	1	900	2	900	409.3	81.0	230.4	43.2	763.9	W14X82	826.5	0.924
5	6	12.5	50	4	1	900	2	900	491.6	97.2	288.0	43.2	920.0	W14X90	1057.5	0.870
4	5	12.5	50	4	1	900	2	900	574.1	113.4	345.6	43.2	1076.3	W14X99	1162.0	0.926
3	4	12.5	50	4	1	900	2	900	656.9	129.6	403.2	43.2	1232.9	W14X120	1412.2	0.873
2	3	12.5	50	4	1	900	2	900	739.9	145.8	460.8	43.2	1389.7	W14X132	1554.2	0.894
1	2	12.5	50	4	1	900	2	900	823.1	162.0	518.4	43.2	1546.7	W14X145	1732.0	0.893
												Sum:	1548.8			

Column Load Take Down Spreadsheet

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Example 4: 10-story office building

Wind Loads

- ASCE 7-16 wind loads
  - Basic wind speed, V = 114 mph
  - Exposure Type B
  - Occupancy Category = II
  - Importance Factor, I = 1.0
  - Wind directionality factor, K_d = 0.85
  - Topographic factor, K_{zt} = 1.0
  - Gust effect factor, G = 0.85
    - Note: 0.85 used for example simplicity, but G > 0.85 for this height building (Use nominal stiffness properties to determine building frequency)
- Auto generation option utilized in SAP
- Limit wind story drifts to h/400 using 10-year MRI wind speed

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Example 4: 10-story office building

Seismic Loads

- ASCE 7-16 seismic loads
- Occupancy Category II
- Importance Factor, I = 1.0
- S_{DS} = 0.327g; S_{D1} = 0.168g
- SDC = C
- Steel Systems Not Specifically Detailed for Seismic Resistance: R = 3; C_d = 3
- Equivalent Lateral Force Procedure

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Example 4: 10-story office building

Seismic Loads

- Approximate fundamental period: $T_a = C_t h_n^x$ with h_n = 125 ft
- For moment frame direction, C_t = 0.028, x = 0.8
- For braced frame direction, C_t = 0.02, x = 0.75
- For S_{D1} = 0.168 g, C_u = 1.564

- Upper limit on period $T \le C_u T_a$
  - T = 2.08 sec for moment frame
  - T = 1.17 sec for braced frame

- Use auto generation option in SAP
  (calculate period using nominal properties, not reduced properties)

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Example 4: 10-story office building

Notional Loads

- Y_i (Dead) = 110 psf total structure weight & superimposed dead loads
- Y_i (Floor Live) = 100 psf
- Y_i (Roof Live) = 30 psf
- *NDead* = 0.002 x 110 psf x 150 ft x 150 ft = 5 kips at each level
- *NLive* = 0.002 x 100 psf x 150 x 150 = 4.5 kips at each floor
- *NLiveR* = 0.002 x 30 psf x 150 x 150 = 1.4 kips at roof

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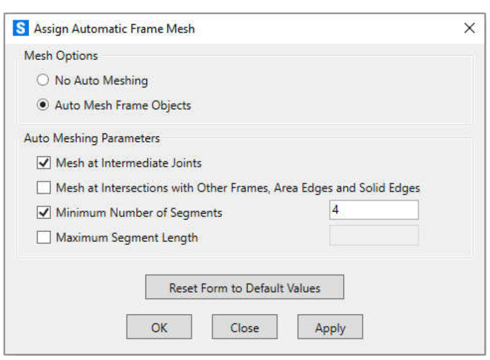
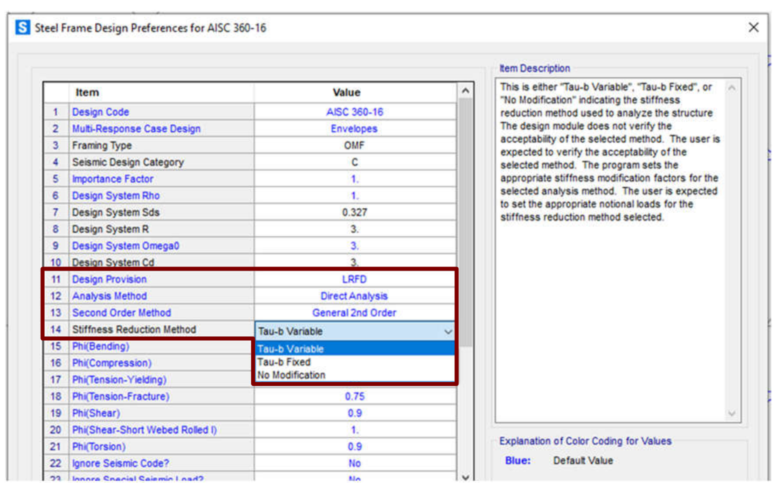
Example 4: 10-story office building

Member meshing and stiffness adjustments



Column Meshing



Stiffness Reduction & τ_b

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Example 4: 10-story office building

Nonlinear Analysis Cases

Combo1	$1.4D + 1.4NDead_x$
Combo2	$1.2D + 1.6L + 0.5L_r + 1.2NDead_x + 1.6NLive_x + 0.5NLiveR_x$
Combo3	$1.4D + 1.4NDead_y$
Combo4	$1.2D + 1.6L + 0.5L_r + 1.2NDead_y + 1.6NLive_y + 0.5NLiveR_y$
Combo5	$1.4D – 1.4NDead_x$
Combo6	$1.2D + 1.6L + 0.5L_r – 1.2NDead_x – 1.6NLive_x – 0.5NLiveR_x$
Combo7	$1.4D – 1.4NDead_y$
Combo8	$1.2D + 1.6L + 0.5L_r – 1.2NDead_y – 1.6NLive_y – 0.5NLiveR_y$
Combo9	$1.2D + 1.0W_x + 0.5L + 0.5L_r$
Combo10	$1.2D – 1.0W_x + 0.5L + 0.5L_r$
Combo11	$1.2D + 1.0W_y + 0.5L + 0.5L_r$
Combo12	$1.2D – 1.0W_y + 0.5L + 0.5L_r$
Combo13	$1.266D + 1.0E_x + 0.5L$
Combo14	$1.266D – 1.0E_x + 0.5L$
Combo15	$1.266D + 1.0E_y + 0.5L$
Combo16	$1.266D – 1.0E_y + 0.5L$
Combo17	$0.9D + 1.0W_x$
Combo18	$0.9D – 1.0W_x$
Combo19	$0.9D + 1.0W_y$
Combo20	$0.9D – 1.0W_y$
Combo21	$0.834D + 1.0E_x$
Combo22	$0.834D – 1.0E_x$
Combo23	$0.834D + 1.0E_y$
Combo24	$0.834D – 1.0E_y$

Notional lateral loads combined only with gravity loads

Note:
Torsional cases should also be considered.
For coupled or correlated systems, Nx & Ny should be applied simultaneously with appropriate directional correlation.

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Example 4: 10-story office building

Strength Analysis

- Perform an iterative second-order elastic analysis including P-Δ and P-δ effects using **reduced** member properties
- Property modifiers for the analysis
  - Axial stiffness = $0.8EA$
  - Flexural stiffness = $0.8\tau_b EI$

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Example 4: 10-story office building

Serviceability Analysis

- For serviceability checks, perform a second-order elastic analysis including *P*-Δ and *P*-δ effects using the **nominal** (unreduced) stiffness

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Example 4: 10-story office building

Drift Check – Braced Frame

Drift for Serviceability Limit State
Strength Controlled Braced Frame Design

Level	Deflection 10-yr wind, δ (in.)	Story Drift 10-yr wind, Δ (in.)	Drift Index
ROOF	0.825	0.079	h/1901
10	0.746	0.088	h/1709
9	0.658	0.089	h/1685
8	0.569	0.091	h/1650
7	0.478	0.091	h/1656
6	0.388	0.089	h/1690
5	0.299	0.085	h/1764
4	0.214	0.080	h/1877
3	0.134	0.073	h/2058
2	0.061	0.061	h/2451

all < h/400 → OK

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Example 4: 10-story office building

Drift Check – Moment Frame

Drift for Serviceability Limit State Strength Controlled Moment Frame Design			
Level	Deflection 10-yr wind, δ (in.)	Story Drift 10-yr wind, Δ (in.)	Drift Index
ROOF	3.43	0.13	h/1174
10	3.31	0.21	h/709
9	3.09	0.27	h/551
8	2.82	0.31	h/483
7	2.51	0.35	h/435
6	2.17	0.37	h/403
5	1.79	0.38	h/390
4	1.41	0.40	h/377
3	1.01	0.41	h/366
2	0.60	0.60	h/249

$\}$ > h/400 → Stiffen

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Example 4: 10-story office building

Drift Check – Moment Frame Optimized for Wind Drift

Drift for Serviceability Limit State Drift Controlled Moment Frame Design			
Level	Deflection 10-yr wind, δ (in.)	Story Drift 10-yr wind, Δ (in.)	Drift Index
ROOF	3.12	0.127	h/1178
10	2.99	0.211	h/710
9	2.78	0.272	h/552
8	2.51	0.310	h/484
7	2.20	0.344	h/436
6	1.86	0.371	h/404
5	1.49	0.375	h/400
4	1.11	0.385	h/400
3	0.737	0.362	h/414
2	0.374	0.374	h/401

all < h/400 → OK

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Example 4: 10-story office building

Seismic Drift Check

- From ASCE 7-16 Table 12.12-1, allowable story drift = $0.020h_{sx}$ = 0.020 x 150” = 3”
- Max. story drift = 0.79” (level 9)
- Inelastic drift $\delta = \frac{C_d \delta_{xe}}{I_e} = \frac{3(0.79")}{1} = 2.37" < 3"$ → OK

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Example 4: 10-story office building

Seismic Stability Check (ASCE 7-16, 12.8.7)

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$$ (ASCE 7-16 Eqn 12.8-16)

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$ (ASCE 7-16 Eqn 12.8-17)

- θ checked at each level, worst case θ = 0.22 < 0.25 at first level → OK

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Example 4: 10-story office building

Strength Design Analysis – Final Check

- Perform a second-order elastic analysis including P-Δ and P-δ effects using **reduced** member properties

- Property modifiers for the analysis
  - Axial stiffness = $0.8EA$
  - Flexural stiffness = $0.8\tau_b EI$

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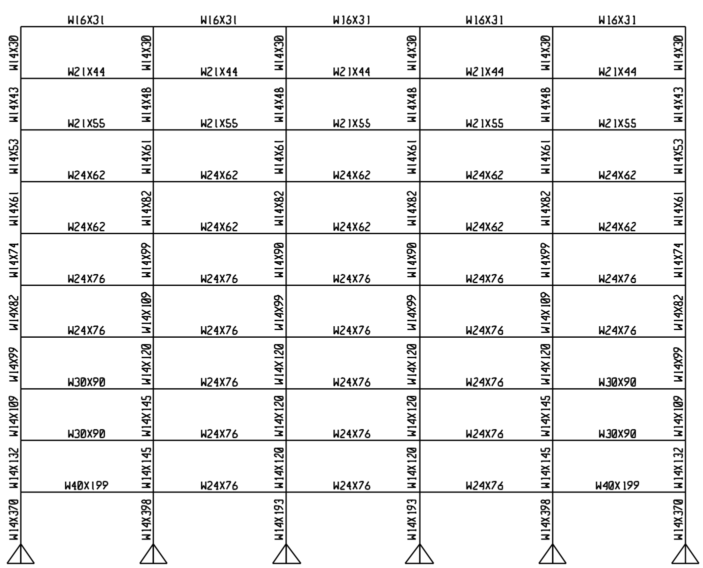
Example 4: 10-story office building

Moment Frame Design – Final Check



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Example 4: 10-story office building

Second-Order to First-Order Drift Ratio

LEVEL	$\Delta_{2nd}/\Delta_{1st}$ (reduced stiffness)
ROOF	1.30
10	1.38
9	1.45
8	1.51
7	1.57
6	1.61
5	1.63
4	1.64
3	1.64
2	1.67

$\Delta_{2nd\ order}/\Delta_{1st\ order} \leq$ 1.7 (using reduced stiffness) → Analysis OK

(notional lateral loads only required with gravity loads)

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Example 4: 10-story office building

Second-Order to First-Order Drift Ratio

LEVEL	$\Delta_{2nd}/\Delta_{1st}$ (nominal stiffness)
ROOF	1.23
10	1.29
9	1.34
8	1.38
7	1.42
6	1.45
5	1.47
4	1.47
3	1.47
2	1.49

$\Delta_{2nd\ order}/\Delta_{1st\ order} \leq$ 1.5 (using nominal stiffness) → Analysis OK

(notional lateral loads only required with gravity loads; ELM can be used)

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Example 4: 10-story office building

Compare Direct Analysis Design with Effective Length Method Design

- Using DA, the drift-controlled moment frame had $\Delta_{2nd\ order}/\Delta_{1st\ order} < 1.5$ with nominal stiffness properties → ELM can be used
- For ELM, perform second-order analysis using final member sizes and nominal (unreduced) stiffness
- Notional loads are already applied to all gravity-only combinations (same notional loads required for ELM in gravity cases)
- Calculate ELM *K*-factors for moment frame

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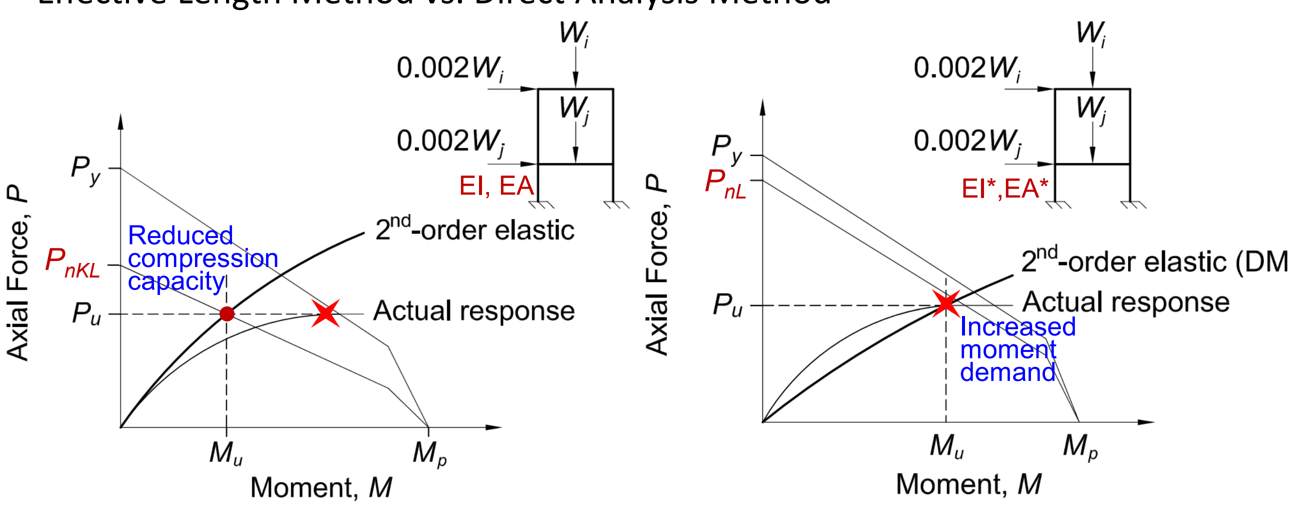
Example 4: 10-story office building

Effective Length Method vs. Direct Analysis Method



Effective Length Method

Direct Analysis Method

Figures from AISC 360-16 Commentary

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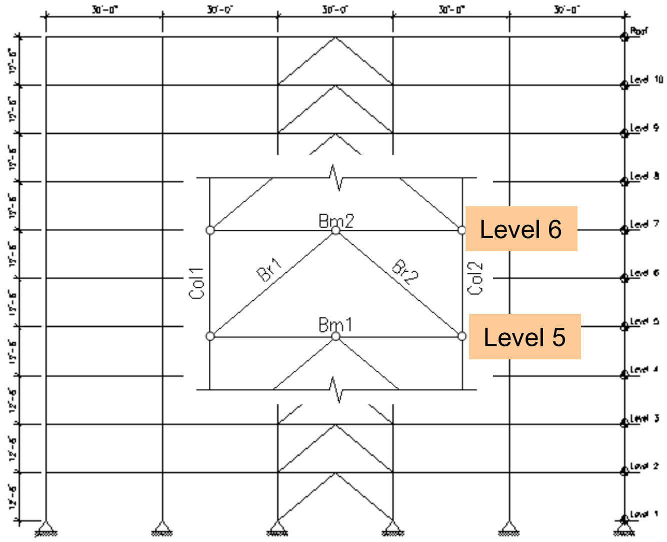
Example 4: 10-story office building

Members for Design Check – Braced Frame



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Example 4: 10-story office building

Braced Frame – DA vs. ELM

Load Combination		Bm1	Bm2	Col1	Col2	Br1	Br2
15	P_r (kips)	-276	**-258**	-62	**-1347**	314	**-362**
	M_r (kip-in)	556	**554**	1	**1**	31	**39**
16	P_r (kips)	**-276**	-258	**-1347**	-62	**-362**	314
	M_r (kip-in)	**556**	554	**1**	1	**39**	31

Design Forces - DA

Load Combination		Bm1	Bm2	Col1	Col2	Br1	Br2
15	P_r (kips)	-271	**-253**	-73	**-1336**	308	**-355**
	M_r (kip-in)	548	**547**	0	**0**	32	**37**
16	P_r (kips)	**-271**	-253	**-1336**	-73	**-355**	308
	M_r (kip-in)	**548**	547	**0**	0	**37**	32

Design Forces - ELM

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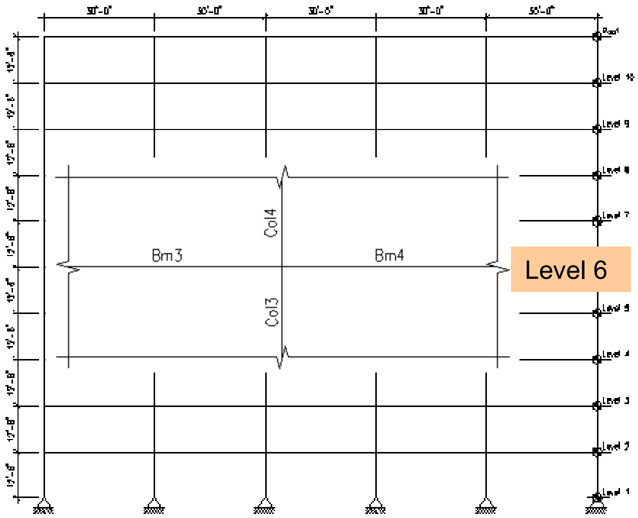
Example 4: 10-story office building

Members for Design Check – Moment Frame



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Example 4: 10-story office building

Moment Frame – DA vs. ELM

Load Combination		Bm3	Bm4	Col3	Col4
13	P_r (kips)	0	0	-359	-300
	M_r (kip-in)	7337	7263	5744	5243
14	P_r (kips)	**0**	**0**	**-355**	**-298**
	Mr (kip-in)	**7662**	**7263**	**5831**	**5323**

Design Forces - DA

Load Combination		Bm3	Bm4	Col3	Col4
13	P_r (kips)	0	0	-359	-300
	M_r (kip-in)	6397	6873	5312	4884
14	P_r (kips)	**0**	**0**	**-355**	**-298**
	Mr (kip-in)	**7251**	**6873**	**5397**	**4964**

Design Forces - ELM

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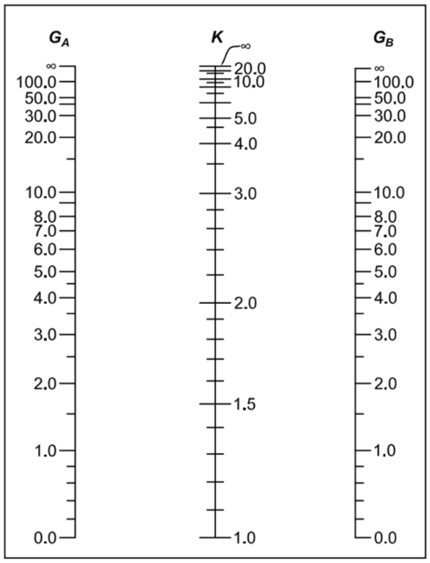
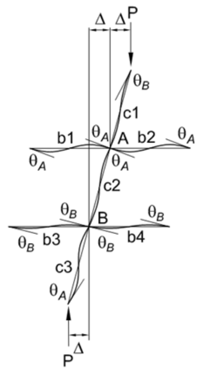
Example 4: 10-story office building

ELM *K*-factor Computation – Sidesway Uninhibited Alignment Chart





$$G = \frac{\Sigma(E_{col}I_{col}/L_{col})}{\Sigma(E_gI_g/L_g)} = \frac{\Sigma(EI/L)_{col}}{\Sigma(EI/L)_g}$$

(Eqn C-A-7-3)

$$\frac{G_AG_B(\pi/K)^2 - 36}{6(G_A+G_B)} - \frac{(\pi/K)}{\tan(\pi/K)} = 0$$

(Eqn C-A-7-2)

(AISC 360-16 commentary Fig. C-A-7.2)

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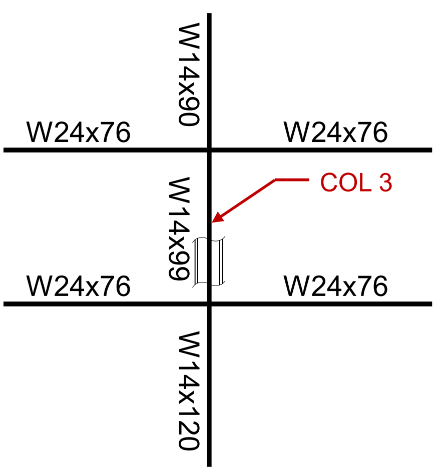
Example 4: 10-story office building

ELM *K*-factor Computation – Sidesway Uninhibited Alignment Chart



$$G = \frac{\sum(EI/L)_c}{\sum(EI/L)_g}$$

(Eqn C-A-7-3)

$$\frac{G_AG_B(\pi/K)^2 - 36}{6(G_A+G_B)} - \frac{(\pi/K)}{\tan(\pi/K)} = 0$$

(Eqn C-A-7-2)

G_{top} = 1.21

G_{bot} = 1.42

K = 1.41

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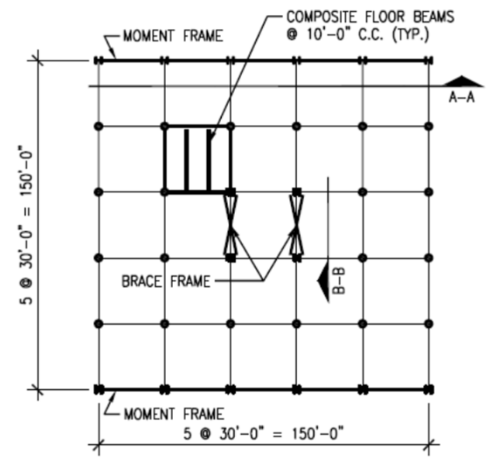
Example 4: 10-story office building

ELM *K*-Factor Adjustment



PLAN

- Only 2 moment frames
- "Leaning" gravity columns stabilized by the moment frames
- Adjust *K*-factor for the effect of leaning columns

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Example 4: 10-story office building

ELM *K*-factor Adjustment – Story Buckling Method

$$K_2 = \sqrt{\frac{\pi^2 EI/L^2}{P_r}\left(\frac{\sum\limits_{all\ \ col} P_r}{\sum\limits_{non-leaning\ \ cols} \frac{\pi^2 EI}{(K_{n2}L)^2}}\right)} \geq \sqrt{\frac{5}{8}}K_{n2} \qquad \text{(Eqn C-A-7-8)}$$

- P_r = 355 kips; ΣP_r = 17,916 kips; I = 1,110 in^4; K_{n2} = 1.41
- For columns supporting level 6, $\Sigma(I/K_{n2})$ = 8782.2 in^4
- K_2 = 2.52

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Example 4: 10-story office building

Interaction Equation Comparison

COL 3 (ELM)

M_r = 5,397 kip-in; P_r = 355 kips

Try W14x99

ϕM_n = 7,752 kip-in (Table 3-2)

$(KL/r)_x$ = 2.52 x 150 / 6.17 = 61.26

$(KL/r)_y$ = 1 x 150 / 3.71 = 40.43

ϕP_n = 995 kips (Eqns E3-1, E3-2)

Interaction equation H1-1a:

355/995 + (8/9)(5397/7752) = 0.98

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Example 4: 10-story office building

Interaction Equation Comparison

COL 3 (ELM)

M_r = 5,397 kip-in; P_r = 355 kips

Try W14x99

ϕM_n = 7,752 kip-in (Table 3-2)

$(KL/r)_x$ = 2.52 x 150 / 6.17 = 61.26

$(KL/r)_y$ = 1 x 150 / 3.71 = 40.43

ϕP_n = 995 kips (Eqns E3-1, E3-2)

Interaction equation H1-1a:

355/995 + (8/9)(5397/7752) = 0.98

COL 3 (DA)

M_r = 5,831 kip-in; P_r = 355 kips

Try W14x99

ϕM_n = 7,752 kip-in (Table 3-2)

$(KL/r)_x = (L/r)_x$ = 150 / 6.17 = 24.31

$(KL/r)_y = (L/r)_y$ = 150 / 3.71 = 40.43

ϕP_n = 1162 kips (Eqns E3-1, E3-2)

Interaction equation H1-1a:

355/1162 + (8/9)(5831/7752) = 0.97

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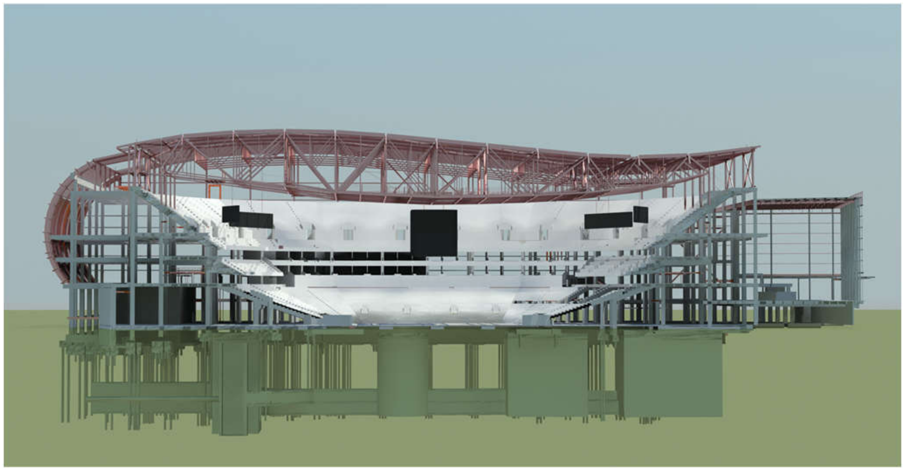
Example 5: Long-Span Roof Truss Bracing System
KFC Yum! Center



Rendering courtesy of Populous

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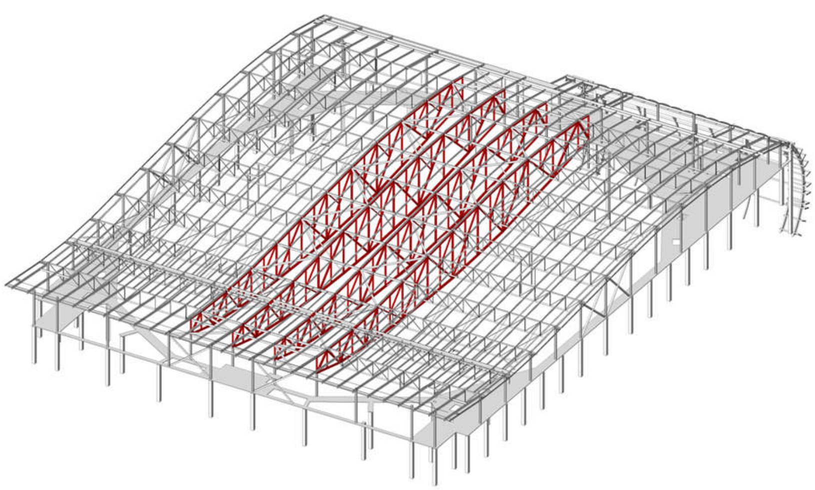
Example 5: Long-Span Roof Truss Bracing System
KFC Yum! Center



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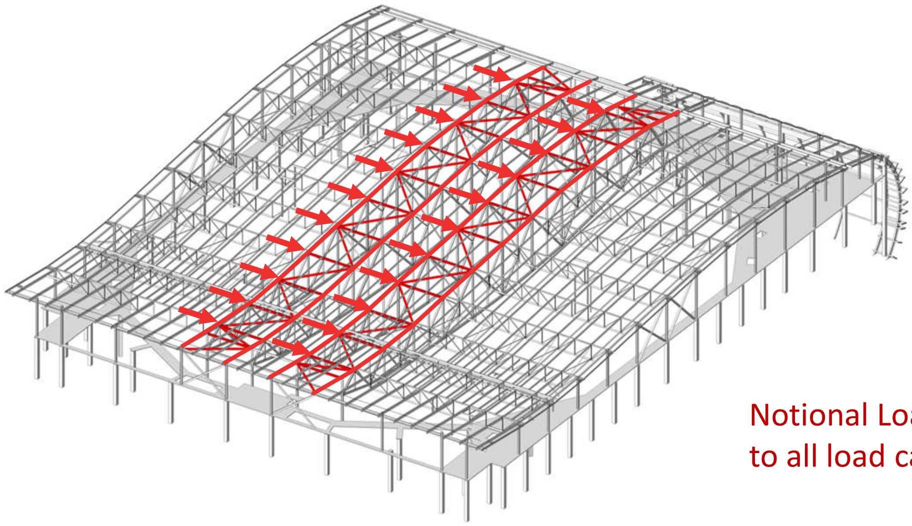
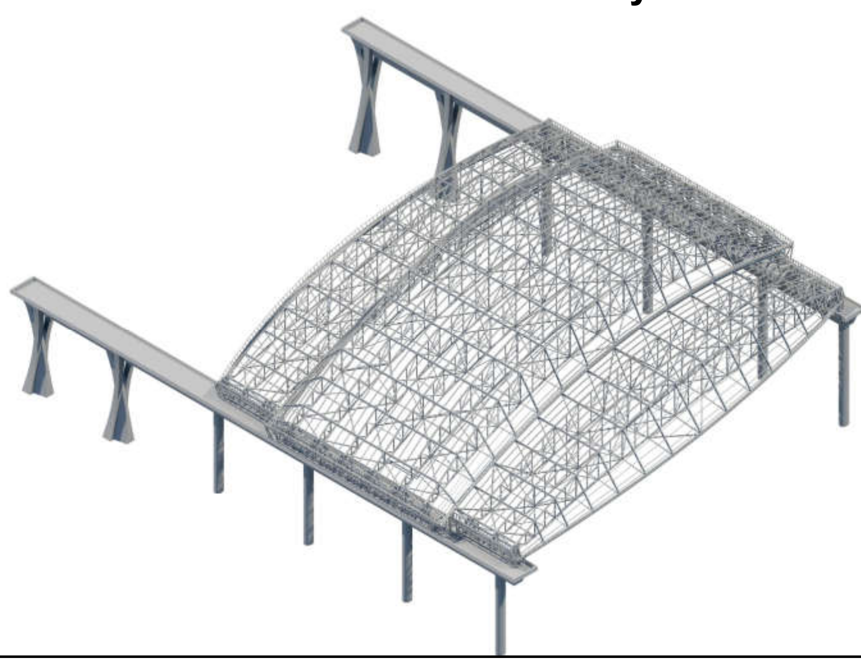
Example 5: Long-Span Roof Truss Bracing System
KFC Yum! Center

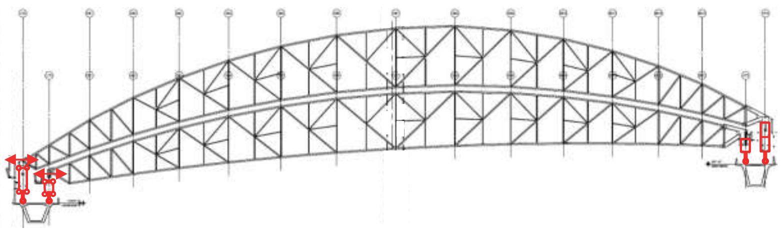


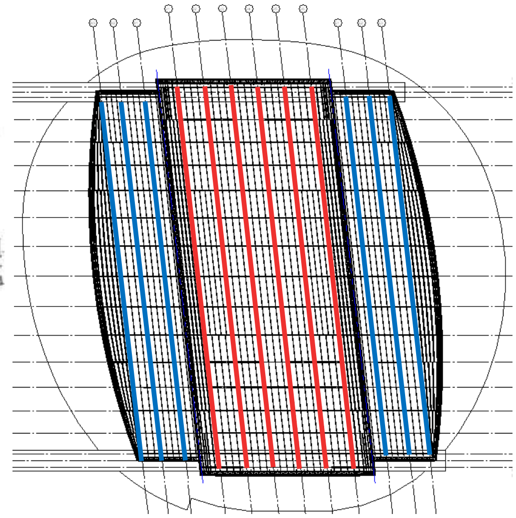
Notional Loads added to all load cases

Example 6: Retractable Roof Panel Stability – Marlins Park



Example 6: Retractable Roof Panel Stability – Marlins Park


Truss Profiles


Roof Panel Top Framing

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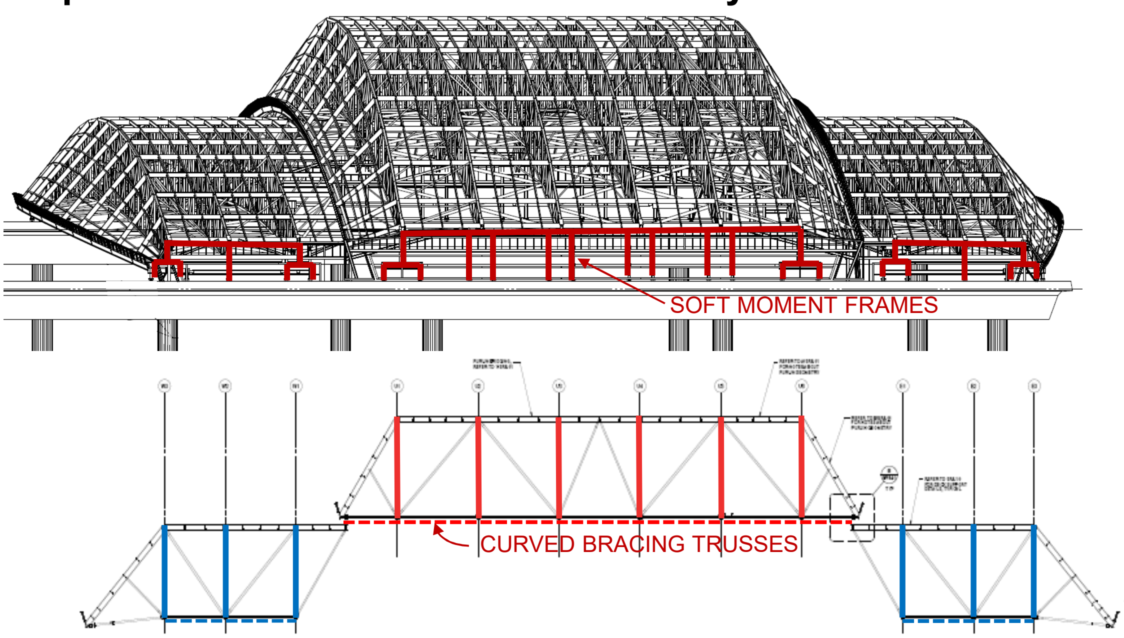
Example 6: Retractable Roof Panel Stability – Marlins Park



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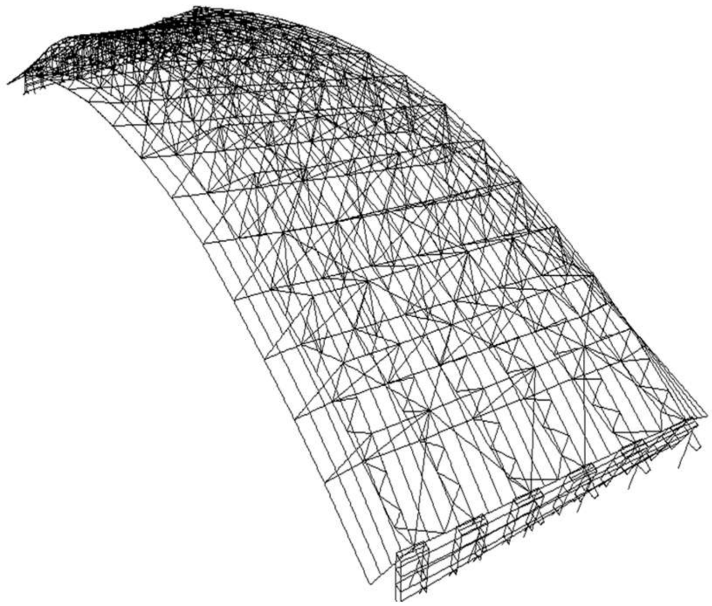
Example 6: Retractable Roof Panel Stability – Marlins Park

- Generate potential buckling shapes
- Mimic effects with notional loads
- Notional loads added to all load combinations



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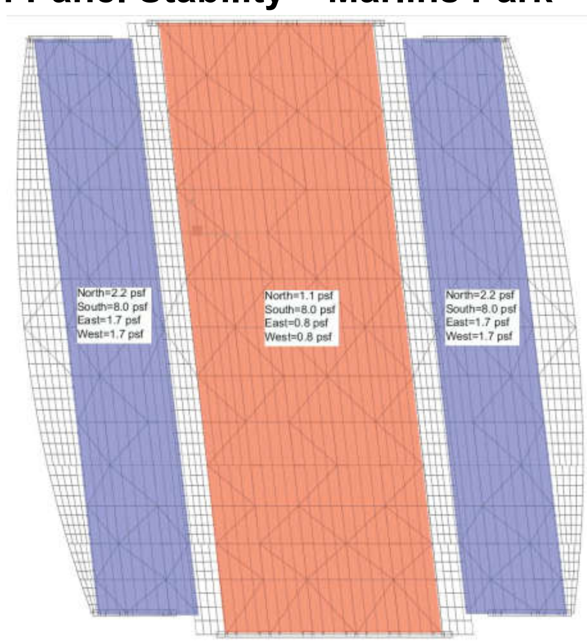
Example 6: Retractable Roof Panel Stability – Marlins Park

- Generate potential buckling shapes
- Mimic effects with notional loads
- Notional loads added to all load combinations
- Reduce system stiffness
- Perform second-order analyses



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Example 6: Retractable Roof Panel Stability – Marlins Park

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Direct Analysis Method Application Summary

- Accurately model frame behavior
- Factor loads (even for ASD)
- Consider initial imperfections (apply notional loads)
- Reduce all stiffness that contributes to stability
- Second-order analysis – include both P-Δ and P-δ
- K=1 for member design
- Nominal (unreduced) stiffness for building periods and serviceability checks

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AISC | Questions?

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Steel.

CEU / PDH Certificates

For those participating at their own connection…

- Reporting attendance is not necessary.
- Certificates will be issued based on AISC’s attendance record.
- You will be receiving certificates via email from registration@aisc.org.

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CEU / PDH Certificates

For those participating at one connection with a group…

- Main registrant will report attendance via an online form. (The link will be provided in an email from registration@aisc.org.)
    - Username: Same as AISC website username.
    - Password: Same as AISC website password.
- Once attendance has been reported, each group member will be receiving certificates via email from registration@aisc.org.

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AISC | Thank you

Smarter. Stronger. Steel.