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Course Description

The Direct Analysis Method – Application and Examples

December 8, 2016

The Direct Analysis Method first appeared in the 2005 AISC *Specification for Structural Steel Buildings* as an alternate way to design for stability. It was upgraded to Chapter C in the 2010 Specification as the primary method to design structures for stability. For the many engineers transitioning from the Effective Length Method to the Direct Analysis Method, the best way to learn is by example. Using a series of design examples that progress from quite simple to quite interesting, the attendee will leave with a real appreciation for how to apply this relatively new design method.

Learning Objectives

- 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 a that second order analysis is performed correctly

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Direct Analysis Method
Application and Examples

David Landis, P.E.
Senior Principal/Design Director, Structures Group
Walter P Moore

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DIRECT ANALYSIS METHOD APPLICATIONS AND EXAMPLES

- What is it and why use it?
- How does it compare to the effective length method?
- Application
- Examples

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What is the Direct Analysis Method?

- Rational approach to stability analysis and design
- P-Δ and P-δ effects are accounted for through second-order analysis
- Geometric imperfections accounted for through direct inclusion in analysis model or by applying "notional loads"
- Inelastic effects such as distributed plasticity are accounted for using flexural and axial stiffness reductions
- Design using K = 1.0 (no more K-factors!)

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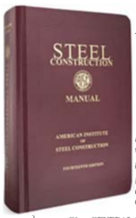
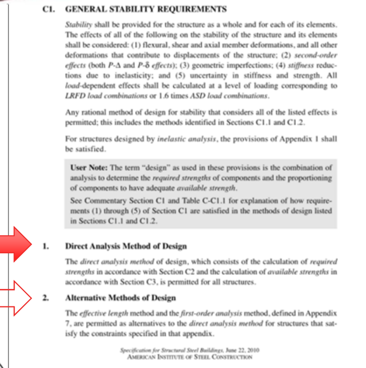
DIRECT ANALYSIS METHOD



AISC 360-10



CHAPTER C

DESIGN FOR STABILITY

…es requirements for the design of structures for *stability*. The *direct* …resented herein; alternative methods are presented in Appendix 7.

…ized as follows:

…l Stability Requirements
…ation of Required Strengths
…ation of Available Strengths

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: (1) flexural, shear and axial member deformations, and all other deformations that contribute to displacements of the structure; (2) *second-order effects* (both *P*-Δ and *P*-δ *effects*); (3) geometric imperfections; (4) *stiffness* reductions due to inelasticity; and (5) uncertainty in stiffness and strength. 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.

For structures designed by *inelastic analysis*, the provisions of Appendix 1 shall be satisfied.

User Note: The term "design" as used in these provisions is the combination of analysis to determine the *required strengths* of components and the proportioning of components to have adequate *available strength*.

See Commentary Section C1 and Table C-C1.1 for explanation of how requirements (1) through (5) of Section C1 are satisfied in the methods of design listed in Sections C1.1 and C1.2.

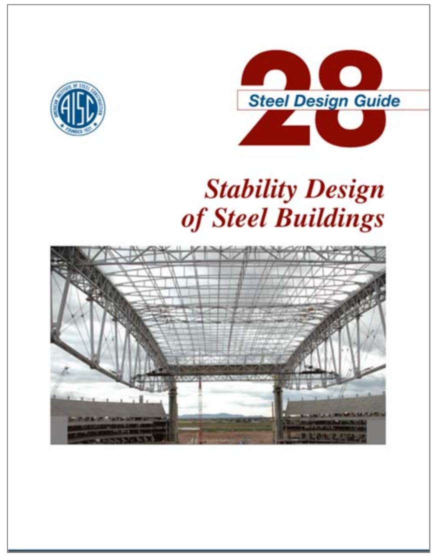
1. Direct Analysis Method of Design

The *direct analysis method* of design, which consists of the calculation of *required strengths* in accordance with Section C2 and the calculation of *available strengths* in accordance with Section C3, is permitted for all structures.

2. Alternative Methods of Design

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

Specification for Structural Steel Buildings, June 22, 2010
AMERICAN INSTITUTE OF STEEL CONSTRUCTION



Why use the Direct Analysis Method?

- Primary method
- Applicable to all types of structural systems
- Captures internal structure forces more accurately
- Correct design of beams and connections providing rotational column restraint
- No need to calculate *K*-factors
- Applicable for all sidesway amplification values ($\Delta_{2nd\ order}/\Delta_{1st\ order}$)
- Effective length method is limited ($\Delta_{2nd\ order}/\Delta_{1st\ order} < 1.5$)

Second-Order Effects – What are they?

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

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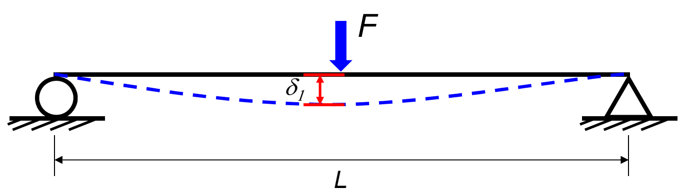
P-δ effect – What is it?

- Equilibrium satisfied on deformed geometry
- Member-level effect
- Member curvature produces additional moment

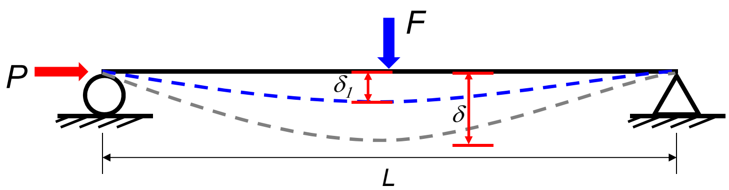


$M = FL/4$

14

P-δ effect – What is it?

- Equilibrium satisfied on deformed geometry
- Member-level effect
- Member curvature produces additional moment



$$M = FL/4 + P\delta$$

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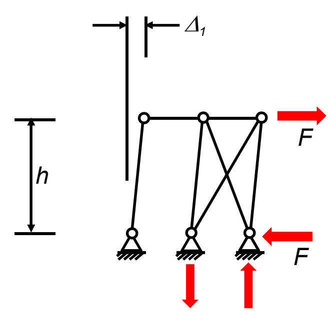
P-Δ effect – What is it?

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

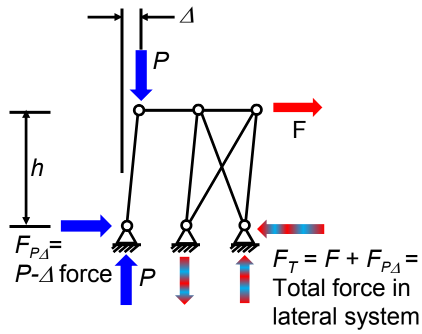


$$M_{OT} = Fh$$

16

P-Δ effect – What is it?

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



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

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Second-Order Effects – What are they?

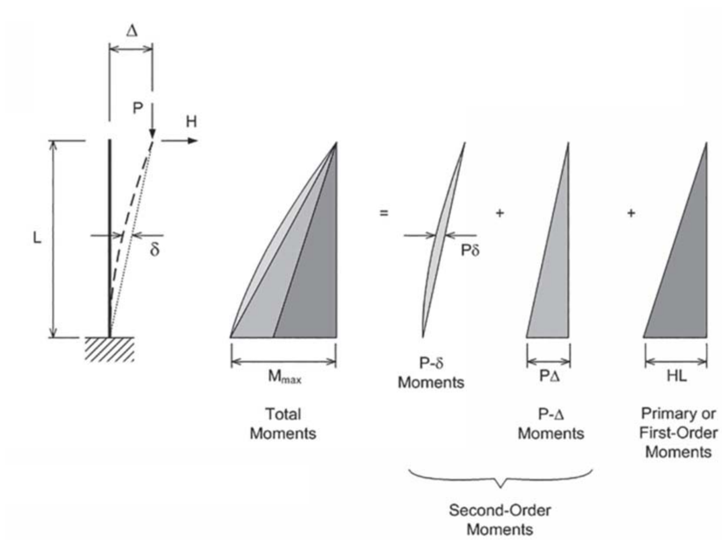

Figure from AISC Design Guide 28

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Second-Order Effects – What are they?

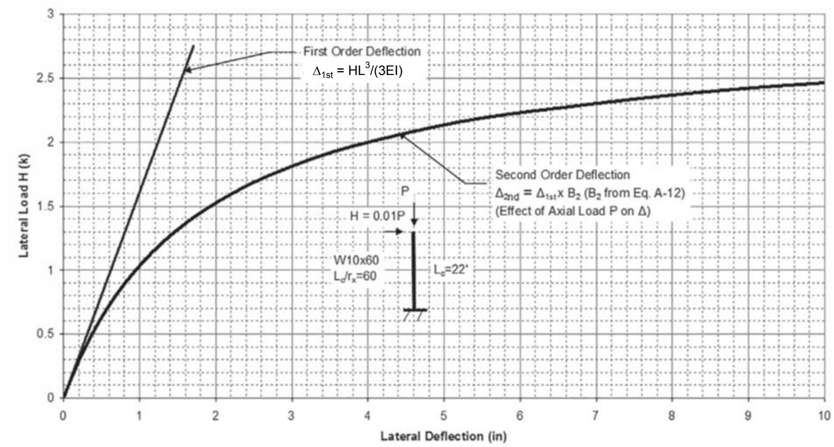

Figure from AISC Design Guide 28

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Direct Analysis Method vs. Effective Length Method

	Effective Length Method (ELM)	Direct Analysis Method (DA)
Type of analysis	Second-order or Amplified First Order	Second-order or Amplified First Order
Member stiffness	Nominal EI & EA	Reduced EI & EA
Notional loads	$0.002Y_i$ minimum	$0.002Y_i$ Minimum if $\Delta_{2nd\ order} / \Delta_{1st\ order} \leq 1.7$ Additive if $\Delta_{2nd\ order} / \Delta_{1st\ order} > 1.7$
Column effective length	Side-sway buckling analysis – determine K	$K = 1$

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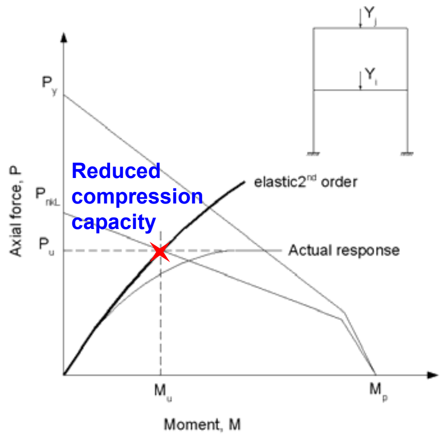
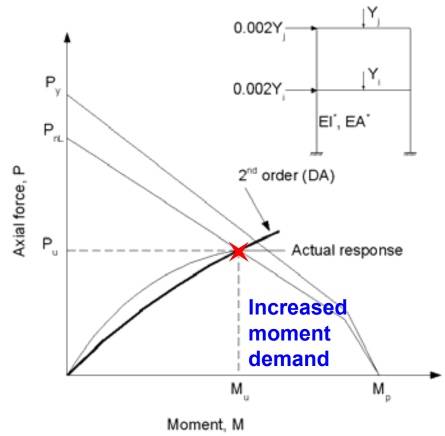
Direct Analysis Method vs. Traditional Effective Length Method



Effective Length Method



Direct Analysis Method

Figures from AISC 360-10 Commentary

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Direct Analysis Method vs. Traditional Effective Length Method

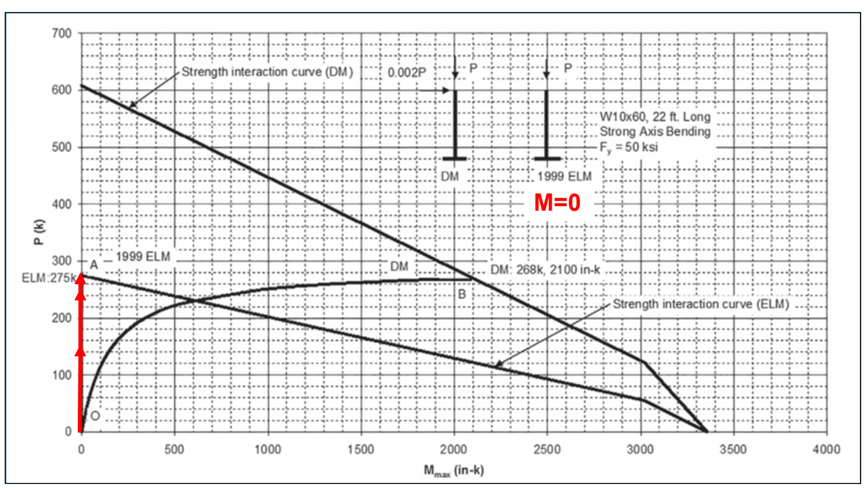

Figure from AISC Design Guide 28

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Direct Analysis Method vs. Traditional Effective Length Method

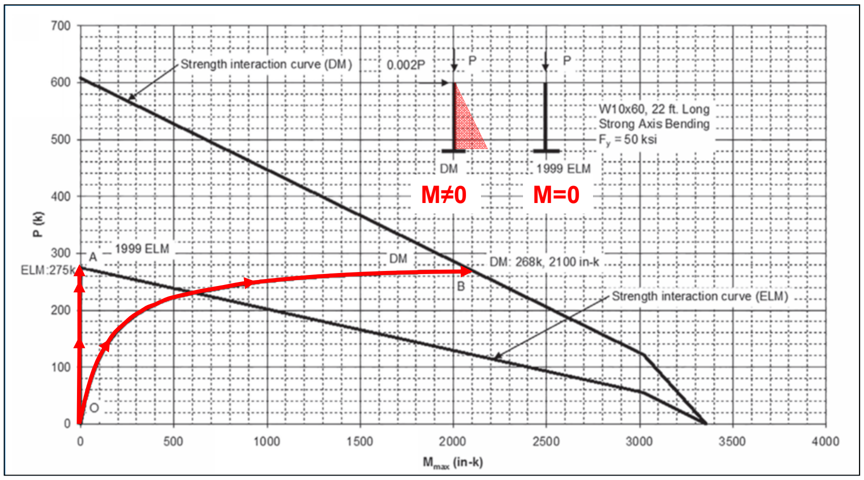

Figure from AISC Design Guide 28

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Direct Analysis Method vs. Traditional Effective Length Method

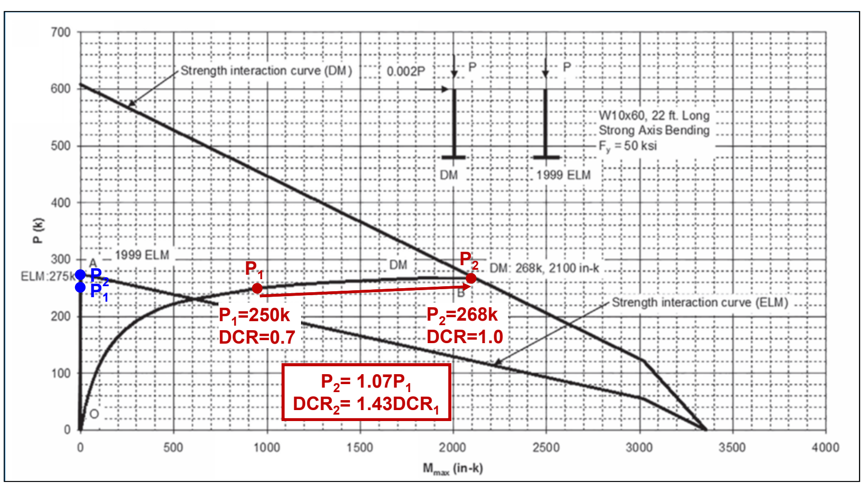

Figure from AISC Design Guide 28

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Direct Analysis Method vs. Traditional Effective Length Method

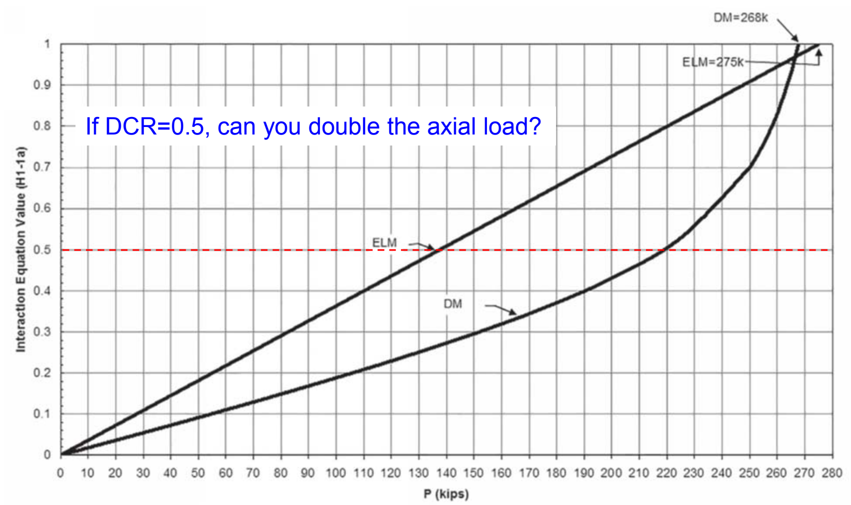

Figure from AISC Design Guide 28

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Direct Analysis Method vs. Effective Length Method

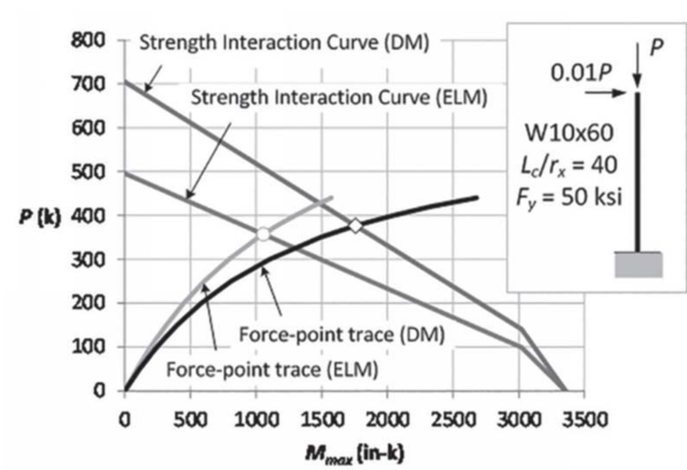

Figure from AISC Design Guide 28

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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
- 2nd-order analysis – include both P-Δ and P-δ
- K=1 for member design
- Serviceability checks use unreduced stiffness

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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
- 2nd-order analysis – include both P-Δ and P-δ
- K=1 for member design
- Serviceability checks use unreduced stiffness

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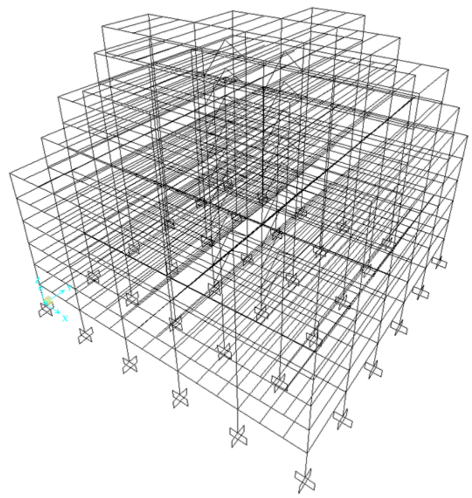
DIRECT ANALYSIS METHOD APPLICATION

→ Accurately model frame behavior



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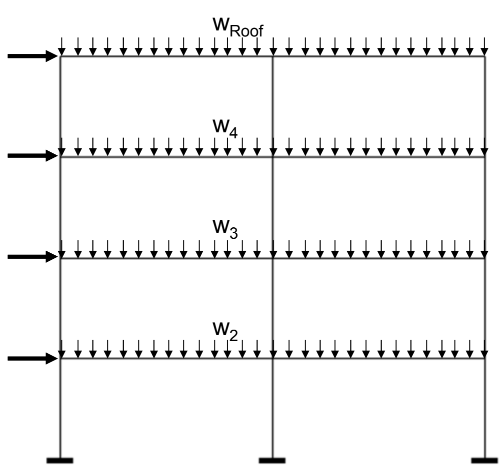
DIRECT ANALYSIS METHOD APPLICATION

→ Accurately model frame behavior



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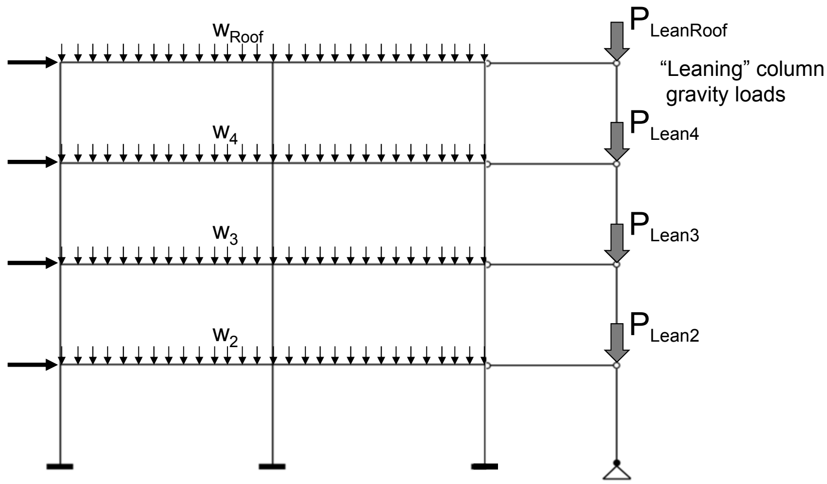
DIRECT ANALYSIS METHOD APPLICATION

→ Accurately model frame behavior



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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
→ 2nd-order analysis – include both *P-Δ* and *P-δ*
→ *K*=1 for member design
→ Serviceability checks use unreduced stiffness

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DIRECT ANALYSIS METHOD APPLICATION

→ Factor Loads (even for ASD!)

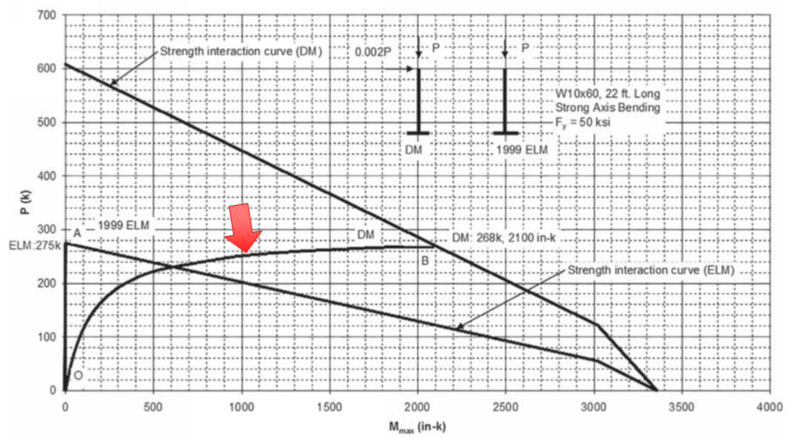

Fig. 3-1. Direct analysis method versus 1999 AISC Specification effective length method, cantilever column with axial load application to failure.

Figure from AISC Design Guide 28

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DIRECT ANALYSIS METHOD APPLICATION

→ Factor Loads (even for ASD!)

- LRFD load combinations
- 1.6 * ASD load combinations (divide resulting forces by 1.6)

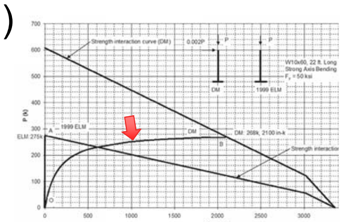

Figure from AISC Design Guide 28

- 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
- 2nd-order analysis – include both $P\text{-}\Delta$ and $P\text{-}\delta$
- K=1 for member design
- Serviceability checks use unreduced stiffness

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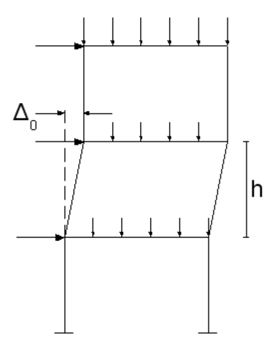
Buildings are not built perfect!

- 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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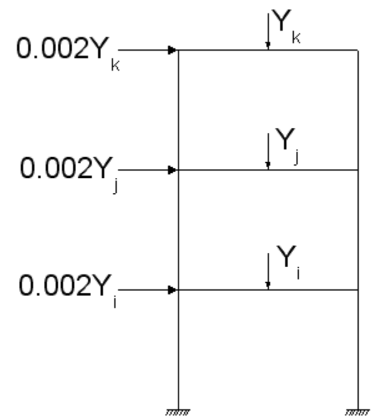
What is the Purpose of Notional Loads?

- Account for geometric imperfections, non-ideal conditions and inelasticity in members
- 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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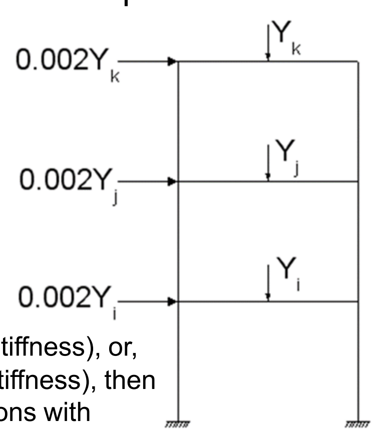
DIRECT ANALYSIS METHOD APPLICATION

- Consider initial geometric imperfections
  - Apply "notional loads" or "notional displacements"
  - Notional Loads:
    - $N_i = 0.002\alpha Y_i$
    - α = 1.0 (LRFD), 1.6 (ASD)
    - Y_i = gravity load applied at level *i*
    - N_i added to other loads



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

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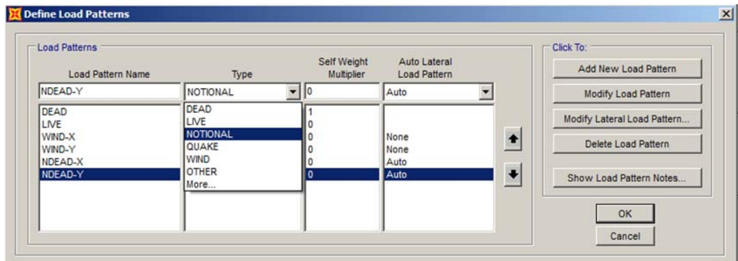
DIRECT ANALYSIS METHOD APPLICATION

- Notional Loads:
  Define Notional Loads and "auto" generate notional loads



(SAP2000 shown)

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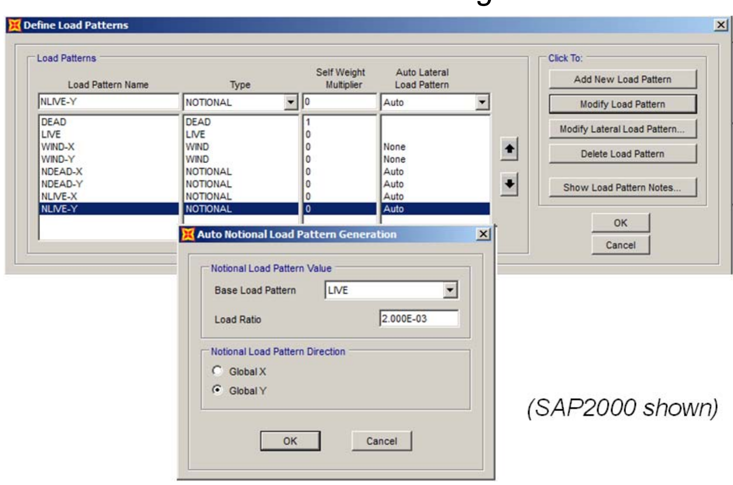
DIRECT ANALYSIS METHOD APPLICATION

- Notional Loads:
  Define Notional Loads and "auto" generate notional loads



(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
- 2nd-order analysis – include both *P-Δ* and *P-δ*
- *K*=1 for member design
- Serviceability checks use unreduced stiffness

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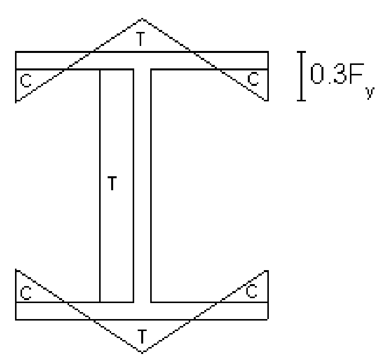
Residual Stresses affect behavior of compression members

- 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

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DIRECT ANALYSIS METHOD APPLICATION

→ Reduce all stiffness that contributes to stability

- Flexural and axial stiffness reductions
- $EA^* = 0.8EA$
- $EI^* = 0.8\,\tau_b EI, \quad \tau_b \leq 1.0$
- τ_b: $\tau_b = 1.0$ when $\alpha P_r/P_y \leq 0.5$

  $\tau_b = 4(\alpha P_r/P_y)[1-(\alpha P_r/P_y)]$ when $\alpha P_r/P_y > 0.5$

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

(τ_b simplification: $\tau_b = 1.0$ can be used if $0.001\alpha Y_i$ added to N_i)

($N_i = 0.003\alpha Y_i$ instead of $0.002\alpha Y_i$)

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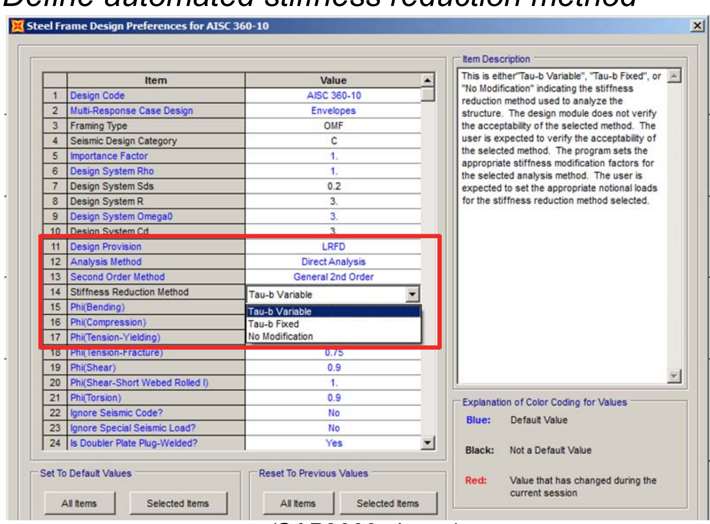
DIRECT ANALYSIS METHOD APPLICATION

- Stiffness Reductions:

  Define automated stiffness reduction method



(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
- → 2nd-order analysis – include both P-Δ and P-δ
- → K=1 for member design
- → Serviceability checks use unreduced stiffness

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DIRECT ANALYSIS METHOD APPLICATION

- → 2nd-order analysis – include both P-Δ and P-δ

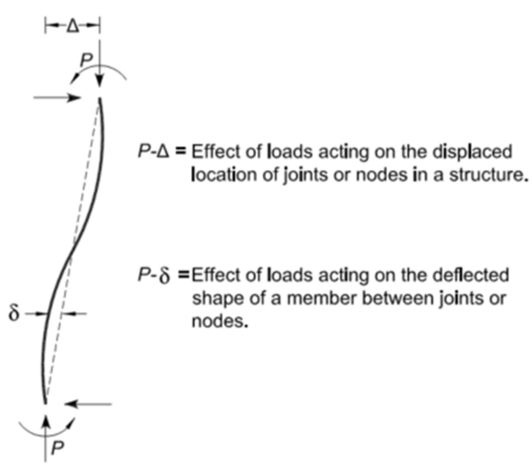

Figure from AISC 360-10 Commentary

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DIRECT ANALYSIS METHOD APPLICATION

→ 2nd-order analysis – include both *P-Δ* and *P-δ*

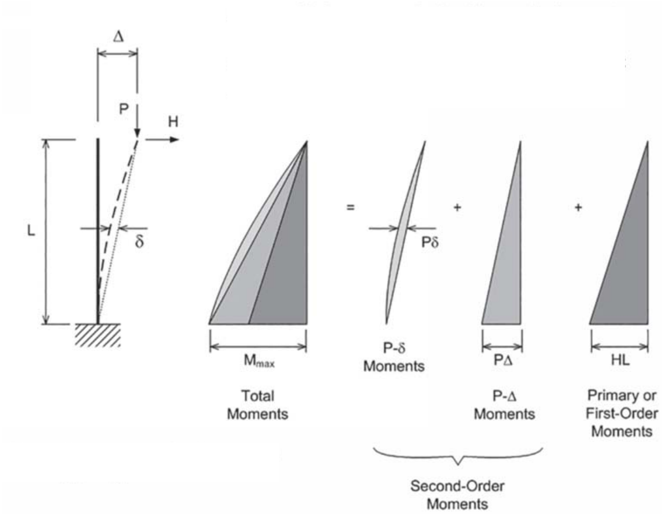

Figure from AISC Design Guide 28

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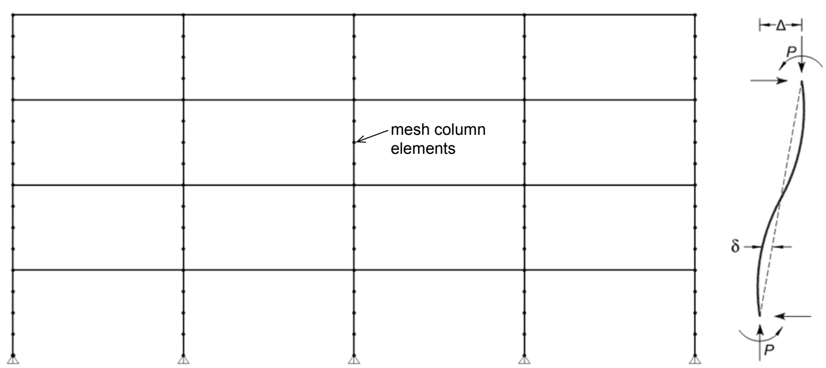
DIRECT ANALYSIS METHOD APPLICATION

→ 2nd-order analysis – include both *P-Δ* and *P-δ*

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



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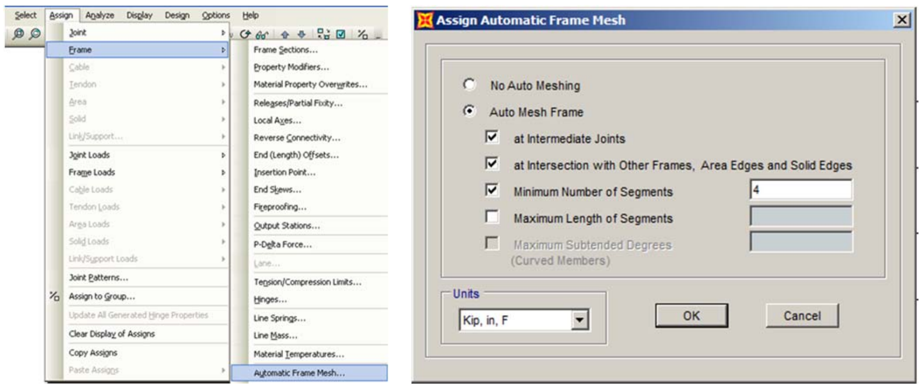
DIRECT ANALYSIS METHOD APPLICATION

→ 2[nd]-order analysis – include both *P-Δ* and *P-δ*

Internally ***mesh frame elements*** *to adequately capture P-δ effects*



(SAP2000 shown)

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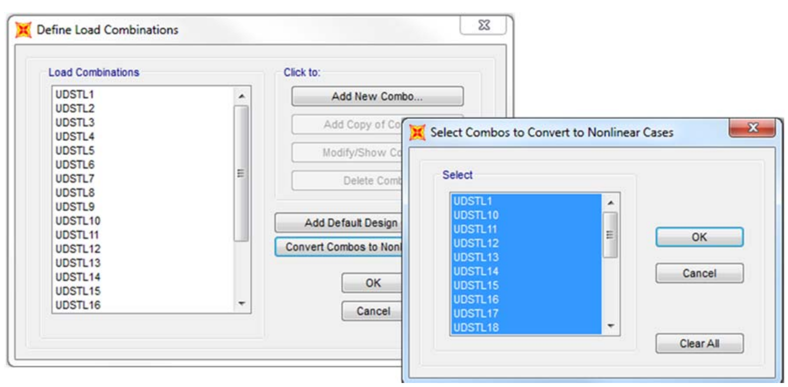
DIRECT ANALYSIS METHOD APPLICATION

→ 2[nd]-order analysis – include both *P-Δ* and *P-δ*

Generate ***nonlinear load cases*** *for 2[nd]-order analysis*



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DIRECT ANALYSIS METHOD APPLICATION

→ 2nd-order analysis – include both *P-Δ* and *P-δ*

- Reduction factors to *EI* and *EA* are assigned only after design check is run (*SAP2000*)
- Iterate as necessary
- Check $\Delta_{2nd\ order}/\Delta_{1st\ order}$ ratio
  - If $\Delta_{2nd\ order}/\Delta_{1st\ order} \leq 1.7$ (reduced stiff.) or 1.5 (nominal stiff.), 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 stiff.) or 1.5 (nominal stiff.), then include N_i in ***all*** load combinations
  - Simplification: include N_i in all load combinations, then no need to check $\Delta_{2nd}/\Delta_{1st}$ ratio

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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

→ 2nd-order analysis – include both *P-Δ* and *P-δ*

→ *K*=1 for member design

→ Serviceability checks use unreduced stiffness

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DIRECT ANALYSIS METHOD APPLICATION

→ Member design

- $K = 1 \longrightarrow KL = L$
- Effective length = actual length
- No more *K*-factors!

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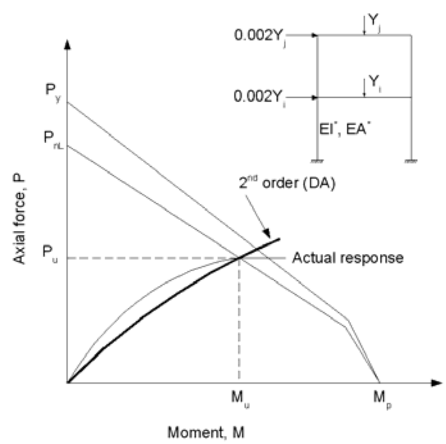
Rationale Behind *K* = 1.0

→ The DA method accounts for both *P*-Δ and *P*-δ effects

→ Geometric imperfections considered explicitly

→ Loss of stiffness under high compression loads considered during analysis

→ Net effect – amplify 2nd order forces to come close to actual response



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DIRECT ANALYSIS METHOD APPLICATION

→ Member design

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

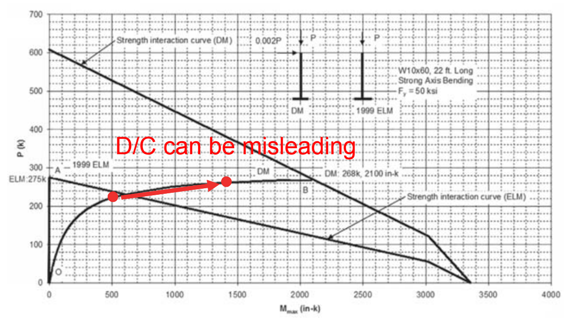

Figure from AISC Design Guide 28

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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

→ 2nd-order analysis – include both $P\text{-}\Delta$ and $P\text{-}\delta$

→ K=1 for member design

→ Serviceability checks use unreduced stiffness

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DIRECT ANALYSIS METHOD APPLICATION

→ Reduced stiffness is only used in strength analysis

→ Serviceability checks use unreduced stiffness

- Check drift limits for wind and seismic using nominal (unreduced) stiffness properties
- Determine building periods using nominal (unreduced) stiffness
- Check vibration using nominal (unreduced) stiffness

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DIRECT ANALYSIS METHOD SUMMARY

→ Accurately model frame behavior

→ Factor loads (even for ASD)

→ Consider initial imperfections (apply notional loads)

→ Reduce all stiffness that contributes to stability

→ 2nd-order analysis – include both P-Δ and P-δ

- (mesh compression elements to capture P-δ)

→ K=1 for member design

→ Serviceability checks use unreduced stiffness

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QUESTION 1

True or False?

τ_b calculations can be simplified by increasing notional lateral loads from $.002\alpha Y_i$ to $.003\alpha Y_i$

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DIRECT ANALYSIS METHOD EXAMPLES

→ Examples using the Direct Analysis method

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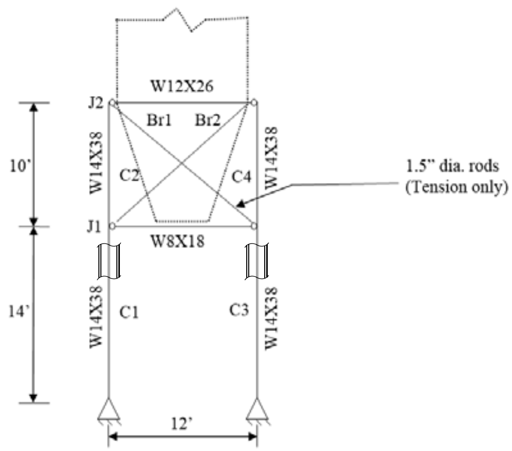
EXAMPLE 1: GRAIN STORAGE BIN

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

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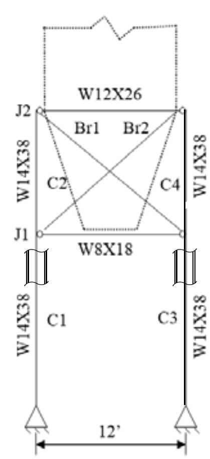
EXAMPLE 1: GRAIN STORAGE BIN
Loads, material properties, definitions, and design requirements

- 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 = 7.0 kips with centroid 9 ft above top of frame
    - Horizontal load, W_H = 3.5 kips at top of each column (ΣW_H = 7.0 kips)
    - Vertical load, W_V = 7.0 x 9/12 = +/-5.25 kips at top of each column
- A992 steel for wide flange shapes, A36 steel rods
- Use Δ_o/H = 0.002 initial out-of-plumbness
- No interstory drift requirement under nominal wind and gravity loads



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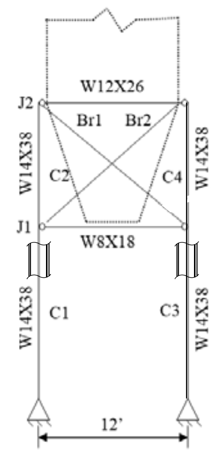
EXAMPLE 1: GRAIN STORAGE BIN
Connection types

- 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.



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EXAMPLE 1: GRAIN STORAGE BIN
Load combinations

Assume the following 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) + 1.6W
Comb4 = 1.2(D + Grain) – 1.6W
Comb5 = 0.9D + 1.6W
Comb6 = 0.9D – 1.6W

(the grain load is handled as a dead load by engineering judgment)

NDead, NGrain: Notional lateral loads = 0.002D and 0.002Grain

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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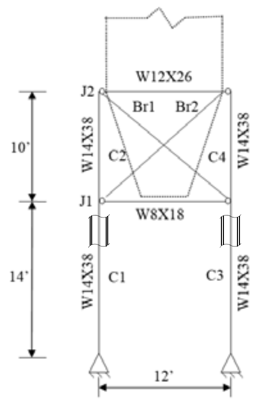
EXAMPLE 1: GRAIN STORAGE BIN
Drift limits

Verify if the ratio of second-order to first-order story drift ≤ 1.5 at each level of the frame for all load combinations



Joint	Combination	Drift: 1st order	Drift: 2nd order	Ratio
J1	Comb1	0.095	0.114	1.20
J1	Comb3	1.744	2.034	1.17
J2	Comb1	0.035	0.036	1.05
J2	Comb3	0.236	0.258	1.10

Since $\Delta_{2nd}/\Delta_{1st} \leq 1.5$ (w/ unreduced properties), Notional loads can be applied in gravity-load combinations only; not required in combination with lateral loads.

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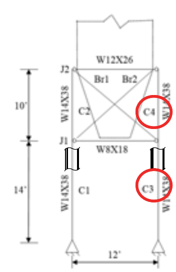
EXAMPLE 1: GRAIN STORAGE BIN
Property modifiers for strength analysis only (AISC spec section C2.3)



Axial stiffness = $0.8EA$

Flexural stiffness = $0.8\tau_b EI$

For example for Columns C3 and C4 in Comb3:

P_r/P_y = 113^k / (50 ksi x 11.2 in^2) = 0.20 < 0.5 ∴ τ_b=1.0

By inspection, τ_b for columns C1 and C2 = 1 also

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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

K = 1 for all members in strength calculations (Chapter C, Section C3)

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

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

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

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

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

Demand/Strength < 1, OK

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Strength C1 (Comb4)

Calculations for Column C1:

→ $K = 1$; $KL_x = L_x = 14$ 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

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Strength C1 (Comb4)

Calculations for Column C1, continued:

→ P_u = 112.9 kips and M_u = 94.7 kip-ft

→ $P_u/\phi P_n$ = 112.9/213 = 0.53 > 0.2; use interaction eqn H1-1a:

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

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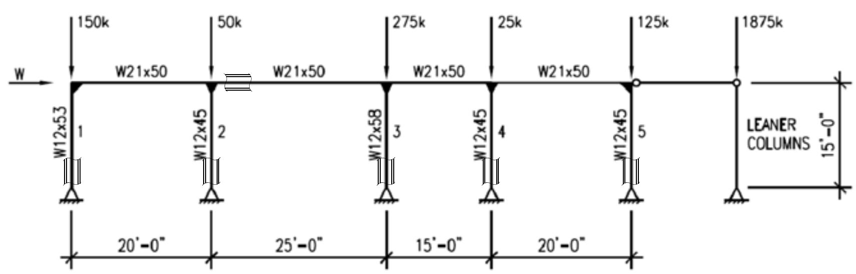
EXAMPLE 2: UNSYMMETRICAL MOMENT FRAME BUILDING

Check each column for conformance to 2010 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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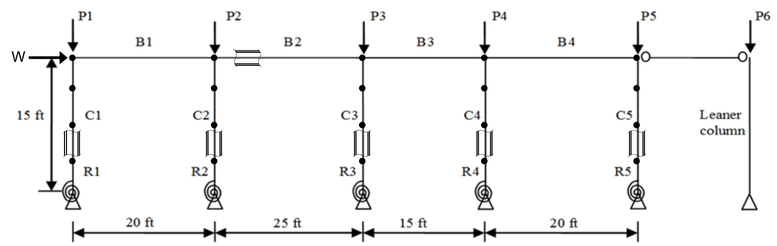
EXAMPLE 2: UNSYMMETRICAL MOMENT FRAME BUILDING
Material properties, definitions, and design requirements

- Column loads are factored gravity loads
- All columns are subjected to strong axis bending in the plane shown
- Wind load W = 12 kips (ASCE 7-05, unfactored)



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EXAMPLE 2: UNSYMMETRICAL MOMENT FRAME BUILDING
Material properties, definitions, and design requirements

- Assume all column bases have a rotational spring stiffness
  $\beta = 6EI/10L$ (derived for "pin base" at foundation using G=10)
- Interstory Drift (Δ/H) limit under wind load = 1/500
- A992 steel

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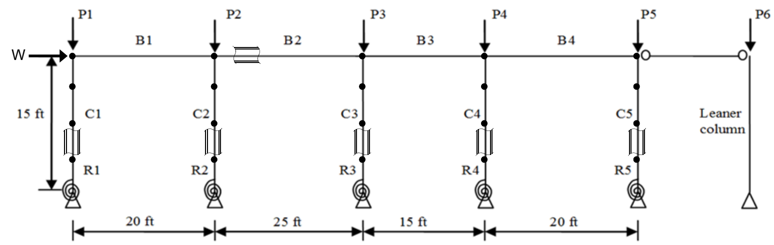
EXAMPLE 2: UNSYMMETRICAL MOMENT FRAME BUILDING
Loads



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 ($\beta = 6EI/10L$) 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

- Perform a second-order elastic analysis including P-Δ and P-δ effects, using reduced member stiffness
- 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.)

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EXAMPLE 2: UNSYMMETRICAL MOMENT FRAME BUILDING
Load combinations

- ASCE 7 load combinations:

Comb2a = 1.2*D* + 1.6*L* + 1.2*NDead* + 1.6*NLive*
Comb2b = 1.2*D* + 1.6*L* – 1.2*NDead* – 1.6*NLive*
Comb4a = 1.2*D* + 1.0*L* + 1.2*NDead* + 1.0*NLive* + 1.6*W*
Comb4b = 1.2*D* + 1.0*L* – 1.2*NDead* – 1.0*NLive* – 1.6*W*

NDead = 0.002*D* notional lateral load
NLive = 0.002*L* notional lateral load

- The check $\Delta_{2nd}/\Delta_{1st}$ vs. 1.7 is determined using the reduced stiffness
- From the second-order analysis results,

$$\Delta_{2nd}/\Delta_{1st} > 1.7$$

- Therefore, the notional lateral loads are *applied additively to* ***all*** *load combinations.* (Chapter C, Section 2.2a)

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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)	-149	-54	-272	-30	-127
	$M_{r,bot}$ (k-in)	87	72	104	79	66
	$M_{r,top}$ (k-in)	-269	-234	-355	-299	-165
COMB4a	P_r (kips)	-121	-50	-228	-27	-113
	$M_{r,bot}$ (k-in)	366	321	431	328	300
	$M_{r,top}$ (k-in)	-1057	-1088	-1374	-1166	-857
COMB4b	P_r (kips)	-136	-42	-237	-24	-100
	$M_{r,bot}$ (k-in)	-370	-332	-438	-330	-314
	$M_{r,top}$ (k-in)	1031	1154	1319	1132	948

- Check for τ_b:
- Check column with the highest axial force: Column C3
  - P_r = 272 kips and A = 17 in^2
  - P_y = 50 ksi x 17 in^2 = 850 kips
  - P_r/P_y = 272/850 = 0.32 < 0.5 ; Therefore, confirmed that τ_b = 1.0

(a) When $\alpha P_r/P_y \le 0.5$

$$\tau_b = 1.0$$

(b) When $\alpha P_r/P_y > 0.5$

$$\tau_b = 4(\alpha P_r/P_y)[1-(\alpha P_r/P_y)]$$

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EXAMPLE 2: UNSYMMETRICAL MOMENT FRAME BUILDING
Strength checks

- K = 1 for all members in strength calculations
- Strength calculations are done using nominal member properties
- Representative calculations for Column C3 (W12x58):
- Governing combination is Comb4a where P_r = 228 kips (compression) and M_r = -1,374 k-in (M_{top} = -1,374k-in, and M_{bot} = 431 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

- For W12x58 column, L_b = 15 ft
- M_r at top = -1,374 k-in
- M_r at bottom = 431 k-in
- $C_b = 12.5\,M_{max}/[2.5M_{max} + 3M_a + 4M_b + 3M_c] = 2.11$ (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)(1,374/3,888) = 0.75 < 1 OK

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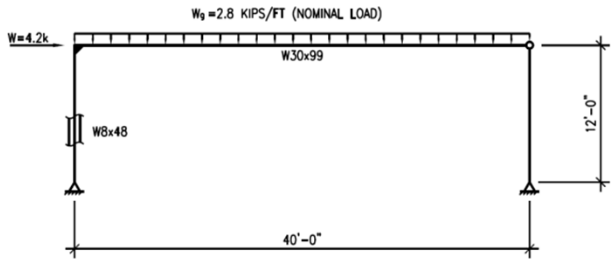
EXAMPLE 3: MARKET SHED BUILDING – SIMPLE MOMENT FRAME

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
- Wind = 20 psf nominal wind load (ASCE 7-05)
- Gravity load = 20 psf Dead + 60 psf Live = 80 psf total
- Use Δ_o/H = 0.002 out-of-plumbness
- Limit lateral deflection Δ = 1” under nominal wind load and total gravity loads (D+L) using a second-order analysis

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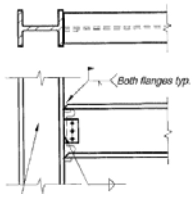
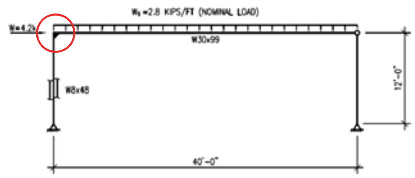
EXAMPLE 3: MARKET SHED BUILDING – SIMPLE MOMENT FRAME
Connection types

- 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 assumed to be a bolted simple shear connection

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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 = 4.2 kips
- 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
Load combinations

ASD load combinations (Chapter C, C2.1.4):

Member design forces are obtained by analyzing the structure for 1.6 times ASD load combinations and then dividing the 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* + W)
Comb5b = 1.6(D + *SelfWt* - W)
Comb6a = 1.6(D + *SelfWt* + 0.75L_r + 0.75W)
Comb6b = 1.6(D + *SelfWt* + 0.75L_r - 0.75W)

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

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EXAMPLE 3: MARKET SHED BUILDING – SIMPLE MOMENT FRAME
Analysis

- Direct Analysis is performed using the *reduced* properties at 1.6 times the ASD load combination level using second-order analysis that considers both P-Δ and P-δ. (Column elements are meshed to capture the P-δ effects.)
- Check lateral drift ratio for application of notional lateral loads (using nominal stiffness)
  - $\Delta_{2nd\ order}/\Delta_{1st\ order} < 1.5$ (using nominal stiffness)
  - Therefore, permissible to apply notional lateral loads only in gravity-only load combinations

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EXAMPLE 3: MARKET SHED BUILDING – SIMPLE MOMENT FRAME
Property modifiers for analysis only

→ Section properties are reduced for strength analysis:

- Axial stiffness = 0.8*EA*
- Flexural stiffness = $0.8\tau_b EI$.
- *Assume* τ_b=1.0. (This assumption is checked later.)

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EXAMPLE 3: MARKET SHED BUILDING – SIMPLE MOMENT FRAME
Serviceability drift limits

→ 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)

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EXAMPLE 3: MARKET SHED BUILDING – SIMPLE MOMENT FRAME
Second-order analysis results (with revised member sizes)

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

αP_r = 1.6x58.6 = 93.8 kips < 0.5 x A_g x 50 ksi = 713 kips, thus, 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 strength calculations
- Strength calculations are performed using nominal section properties
- Strength calculations are not presented here
- The new sizes easily work because drift controls the design of the frame

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QUESTION 2

In the Direct Analysis Method, when are reduced stiffness properties used?

a. Strength analysis
b. Member capacity calculations
c. Serviceability checks
d. All of the above
e. Both a and b

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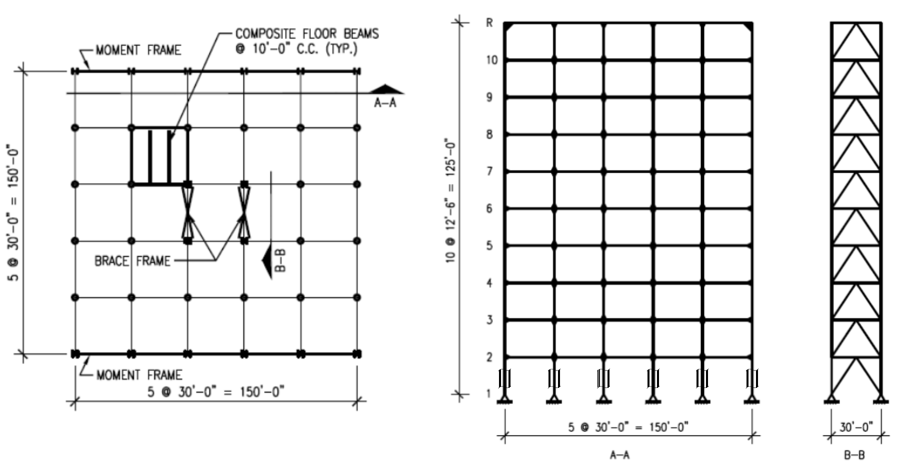
EXAMPLE 4: 10-STORY OFFICE BUILDING



PLAN MOMENT FRAME BRACED FRAME

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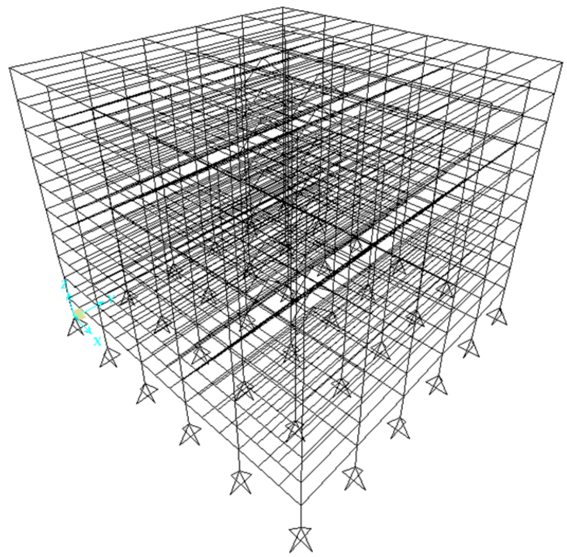
EXAMPLE 4: 10-STORY OFFICE BUILDING
3-D MODEL



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EXAMPLE 4: 10-STORY OFFICE BUILDING

Gravity Loads

Floor
- Composite steel deck (3” + 3½” slab, LWC) = 50 psf
- Superimposed dead load + floor framing = 15 psf
- Wall load = 25 psf (over floor area at all levels)
- Live Load = 100 psf (reducible)

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

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Live Load Reduction

→ Applied according to Section 1607.10, IBC 2012

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

→ 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 x [0.25 + 15 / (2 x 15 x 30)$^{0.5}$] = 75 psf

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Live Load Reduction – Interior Columns

LEVEL	Interior Column, K_{LL} = 4, Tributary area of reducible load			With 100 psf design LL			With 75 psf LL			Correction in Load
	SF	ΣSF	LLR	*P* Live kips	Σ*P* Live kips	Σ*P* Live × LLR kips	*P* Live kips	Σ*P* Live kips	Σ*P* Up Live kips	*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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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

Wind Load Calculation

- ASCE 7-05 wind loads
  - Basic wind speed, V = 90 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
- Auto generation option utilized in SAP

Seismic Load Calculation

- ASCE 7-05 seismic loads
- S_s = 0.317g; S_1 = 0.106g
- Site Class D
- Occupancy Category II
- Importance Factor, I = 1.0
- S_{DS} = 0.327 g; S_{D1} = 0.168 g
- SDC = C
- Steel Systems Not Specifically Detailed for Seismic Resistance - R = 3; C_d = 3
- Equivalent Lateral Force Procedure

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Seismic Design - 2

- 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 = 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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Notional Loads

- Y_i (Dead) = 65 psf + 25 psf + 10 psf (partitions) + 10 psf (vertical framing) = 110 psf
- 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
- *NLive* = 0.002 x 100 x 150 x 150 = 4.5 kips
- *NLiveR* = 0.002 x 30 x 150 x 150 = 1.4 kips

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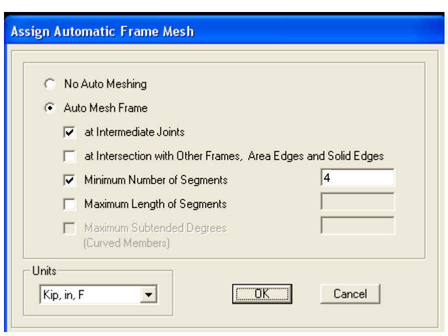
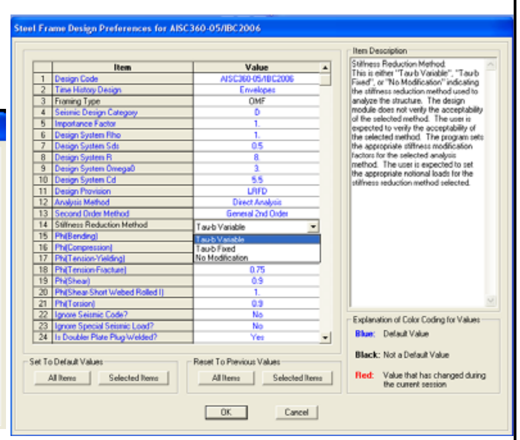
Design Process



Internal Column Meshing



Stiffness Reduction & τ_b

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Nonlinear Load Combinations

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

Notional lateral loads combined with gravity loads (Combo1–Combo8)

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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Strength Design Analysis

- Perform a second-order elastic analysis including $P\text{-}\Delta$ and $P\text{-}\delta$ effects using **reduced** member properties
- Property modifiers for the analysis
  - Axial stiffness = $0.8EA$
  - Flexural stiffness = $0.8\tau_b EI$.
  - Assume $\tau_b = 1.0$. (This assumption is checked later.)

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Serviceability Analysis

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

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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

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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

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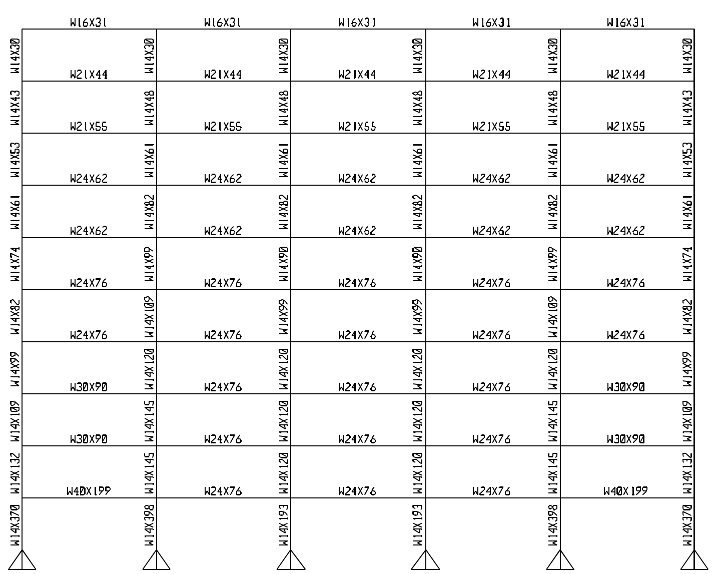
Moment Frame Design – Drift Controlled



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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

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Seismic Drift Check

- From ASCE 7-05 Table 12.12-1, allowable story drift = $0.020h_{sx}$ = 0.020 x 150 in. = 3 in.
- Max. story drift = 0.79” (level 9)
- Inelastic drift = 3 x 0.79” = 2.37 in. < 3 in → OK

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Strength Design Analysis – Final Check

- Perform a second-order elastic analysis including $P\text{-}\Delta$ and $P\text{-}\delta$ effects using **reduced** member properties
- Property modifiers for the analysis
  - Axial stiffness = $0.8EA$
  - Flexural stiffness = $0.8\tau_b EI$.
  - Assume $\tau_b = 1.0$. (This assumption is checked later.)

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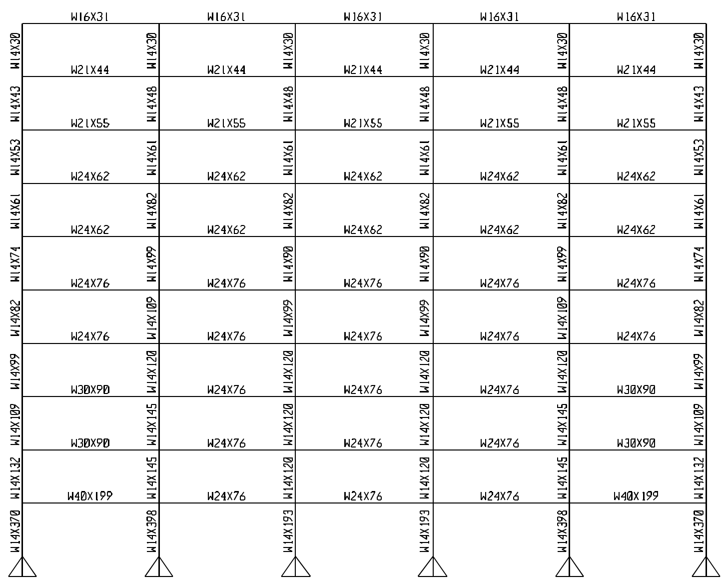
Moment Frame Design – Final Check



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Second-Order to First-Order Drift Ratio

LEVEL	$\Delta_{2nd}/\Delta_{1st}$
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

$\Delta_{2nd\ order}/\Delta_{1st\ order} \leq 1.5$ (nominal properties) ⟶ Analysis OK
(notional lateral loads only required with gravity loads)

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Compare Design with Effective Length Method

- → Using DA, the drift-controlled moment frame had $\Delta_{2nd\ order}/\Delta_{1st\ order} < 1.5$ → ELM can be used
- → For ELM, analyze using final member sizes, with nominal (unreduced) stiffness
- → Notional loads are already applied to all gravity-only combinations (still required for ELM)
- → Will need to calculate K-factors for moment frame

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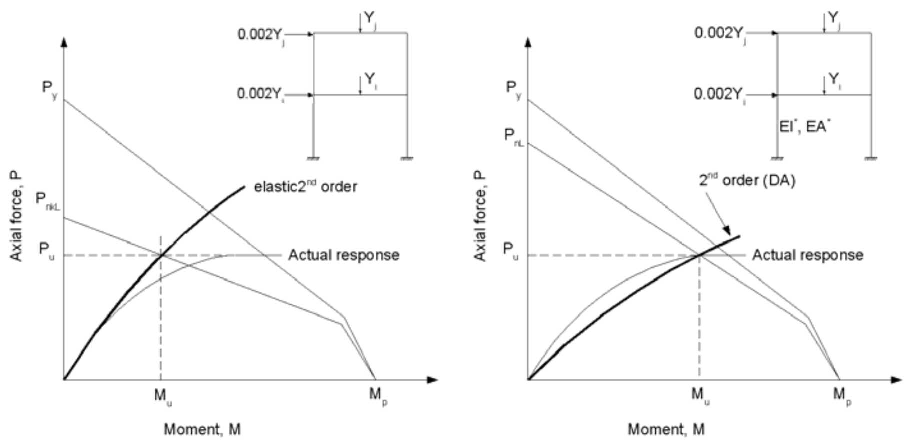
Effective Length Method vs. Direct Analysis Method



Effective Length Method **Direct Analysis Method**

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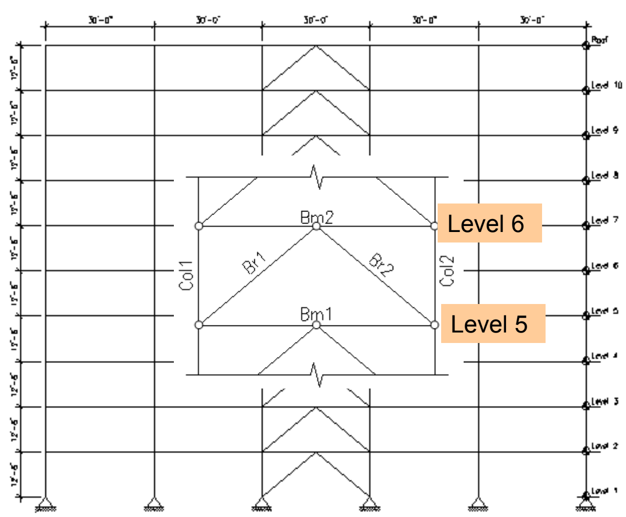
Members for Design Check – Braced Frame



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Braced Frame – DA vs. ELM

Design Forces - DA

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

Design Forces - ELM

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

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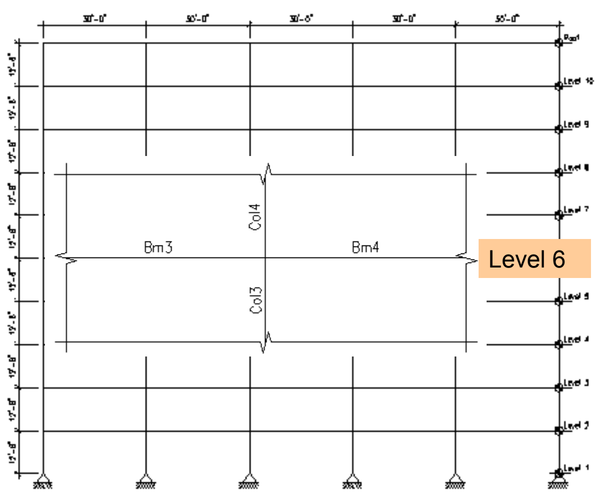
Members for Design Check – Moment Frame



Moment Frame Elevation

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Moment Frame – DA vs. ELM

Design Forces - DA

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 - ELM

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**

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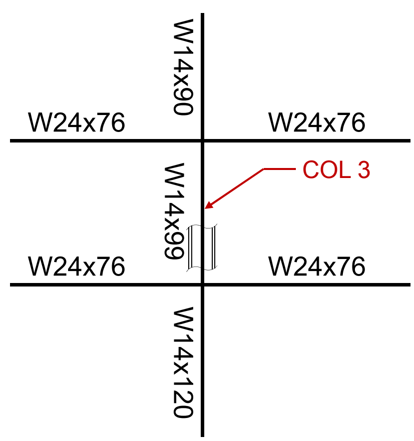
ELM *K*-factor Computation - Nomograph

$$G = \frac{\sum E_c I_c / L_c}{\sum E_b I_b / L_b}$$

$G_{top} = 1.2$

$G_{bot} = 1.4$

$K \approx 1.4$



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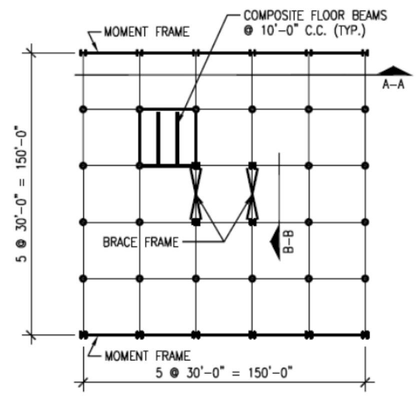
ELM *K*-Factor Adjustment



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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ELM *K*-factor – Story Buckling Method

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

→ P_r = 355 kips; ΣP_r = 17,916 kips; I = 1,110 in^4; K_{n2} = 1.4

→ For columns supporting level 6, $\Sigma(I/K_{n2})$ = 8782.2 in^4

→ **K_2 = 2.52**

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Interaction Equation Comparison

COL 3 (ELM)	COL 3 (DA)
M_r = 5,397 kip-in; P_r = 355 kips	M_r = 5,831 kip-in; P_r = 355 kips
Try W14x99	Try W14x99
ϕM_n = 7,752 kip-in (Table 3-2)	ϕM_n = 7,752 kip-in (Table 3-2)
$(KL/r)_x$ = 2.52 x 150 / 6.17 = 61.26	$(KL/r)_x = (L/r)_x$ = 150 / 6.17 = 24.31
$(KL/r)_y$ = 1 x 150 / 3.71 = 40.43	$(KL/r)_y = (L/r)_y$ = 150 / 3.71 = 40.43
ϕP_n = 995 kips (Eqns E3-1, E3-2)	ϕP_n = 1162 kips (Eqns E3-1, E3-2)
Interaction equation H1-1a: 355/995 + (8/9)(5397/7752) = 0.98	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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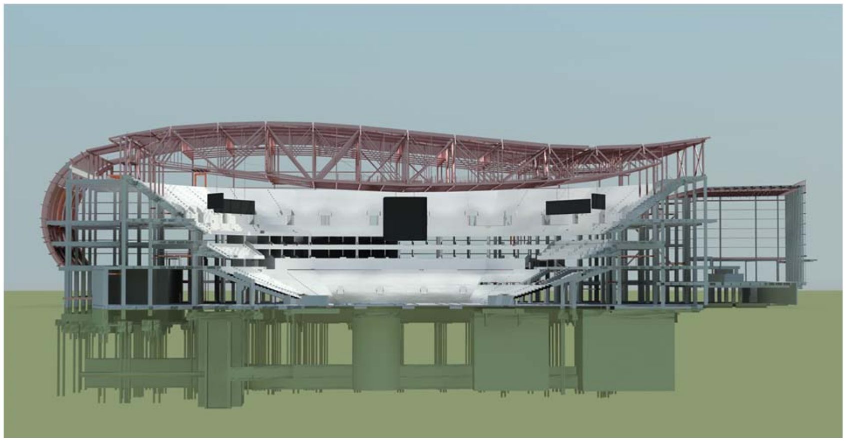
EXAMPLE 5: LONG-SPAN ROOF TRUSS BRACING SYSTEM
KFC Yum! Center



Rendering courtesy of Populous

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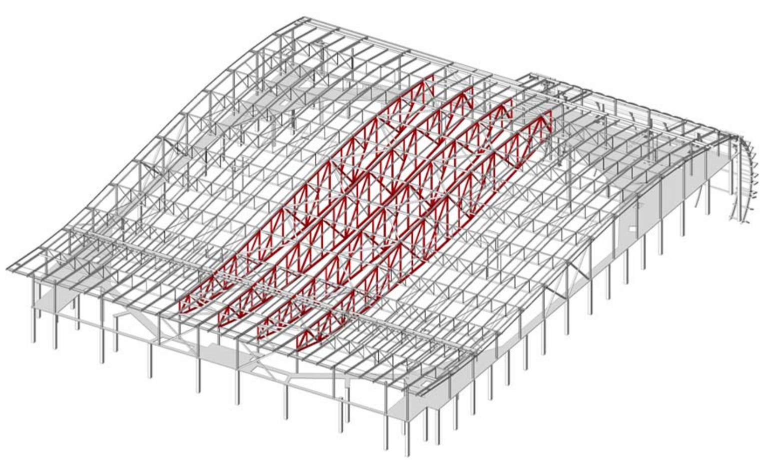
EXAMPLE 5: LONG-SPAN ROOF TRUSS BRACING SYSTEM
KFC Yum! Center



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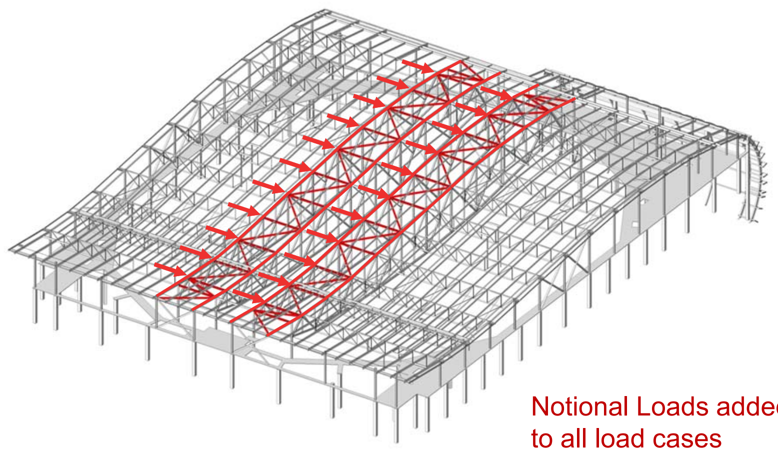
EXAMPLE 5: LONG-SPAN ROOF TRUSS BRACING SYSTEM
KFC Yum! Center



Notional Loads added
to all load cases

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EXAMPLE 6: RETRACTABLE ROOF PANEL STABILITY
MARLINS PARK

Rendering courtesy of Populous

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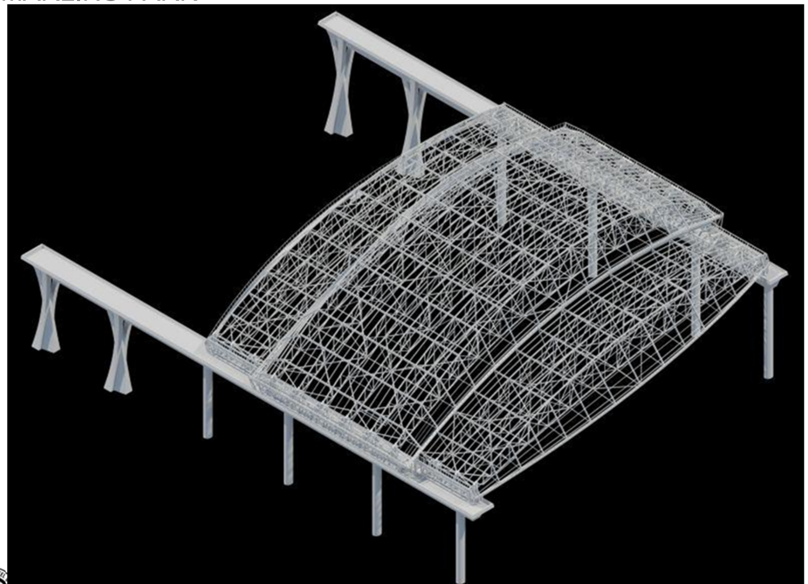
EXAMPLE 6: RETRACTABLE ROOF PANEL STABILITY
MARLINS PARK



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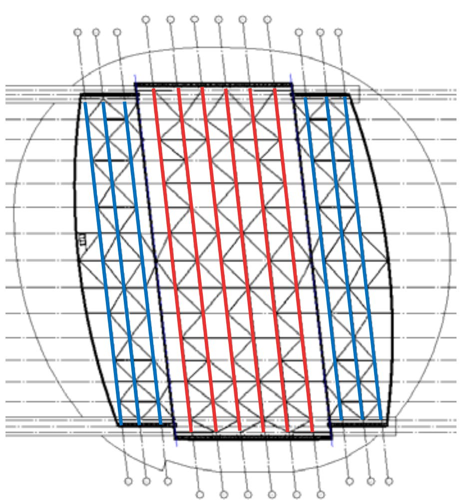
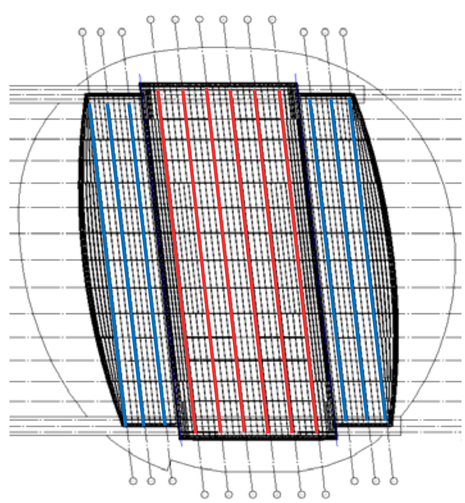
EXAMPLE 6: RETRACTABLE ROOF PANEL STABILITY
MARLINS PARK



Roof Panel Bottom Framing



Roof Panel Top Framing

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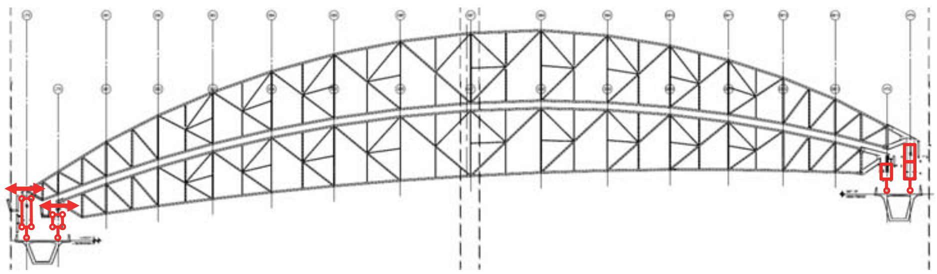
EXAMPLE 6: RETRACTABLE ROOF PANEL STABILITY
MARLINS PARK



Primary Truss Elevations

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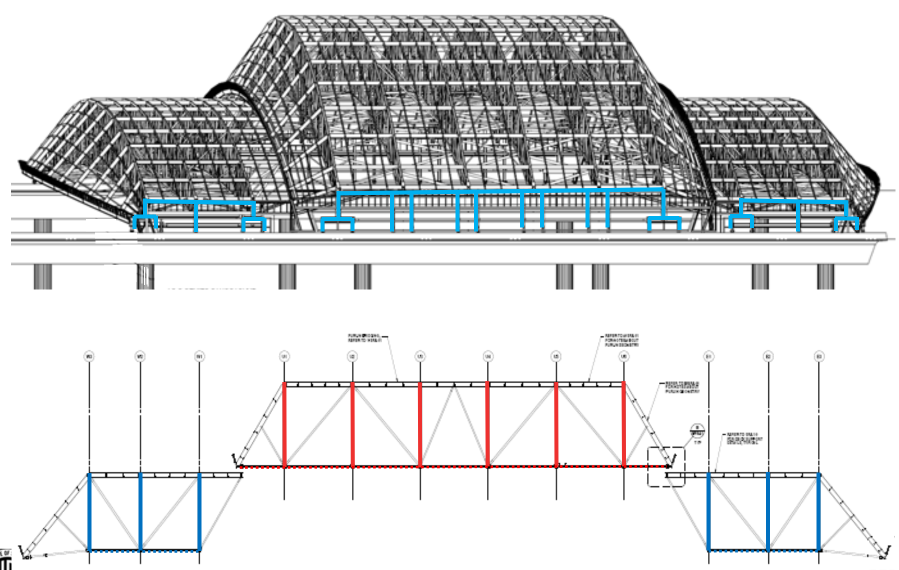
EXAMPLE 6: RETRACTABLE ROOF PANEL STABILITY
MARLINS PARK



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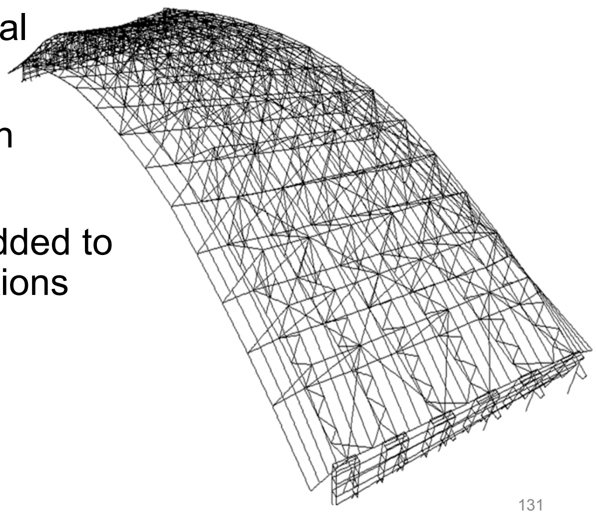
EXAMPLE 6: RETRACTABLE ROOF PANEL STABILITY MARLINS PARK

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



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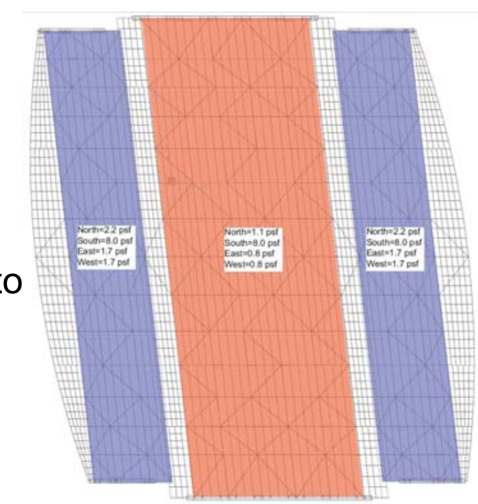
EXAMPLE 6: RETRACTABLE ROOF PANEL STABILITY MARLINS PARK

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



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EXAMPLE 6: RETRACTABLE ROOF PANEL STABILITY

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DIRECT ANALYSIS METHOD SUMMARY

- → Accurately model frame behavior
- → Factor loads (even for ASD)
- → Consider initial imperfections (apply notional loads)
- → Reduce all stiffness that contributes to stability
- → 2^{nd}-order analysis – include both *P-Δ* and *P-δ*
  - (mesh compression elements to capture *P-δ*)
- → *K*=1 for member design
- → Serviceability checks use unreduced stiffness

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QUESTIONS?

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PDH Certificates

Within 2 business days…

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

PDH Certificates

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There's always a solution in steel.

Thank You

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Survey at conclusion of webinar.