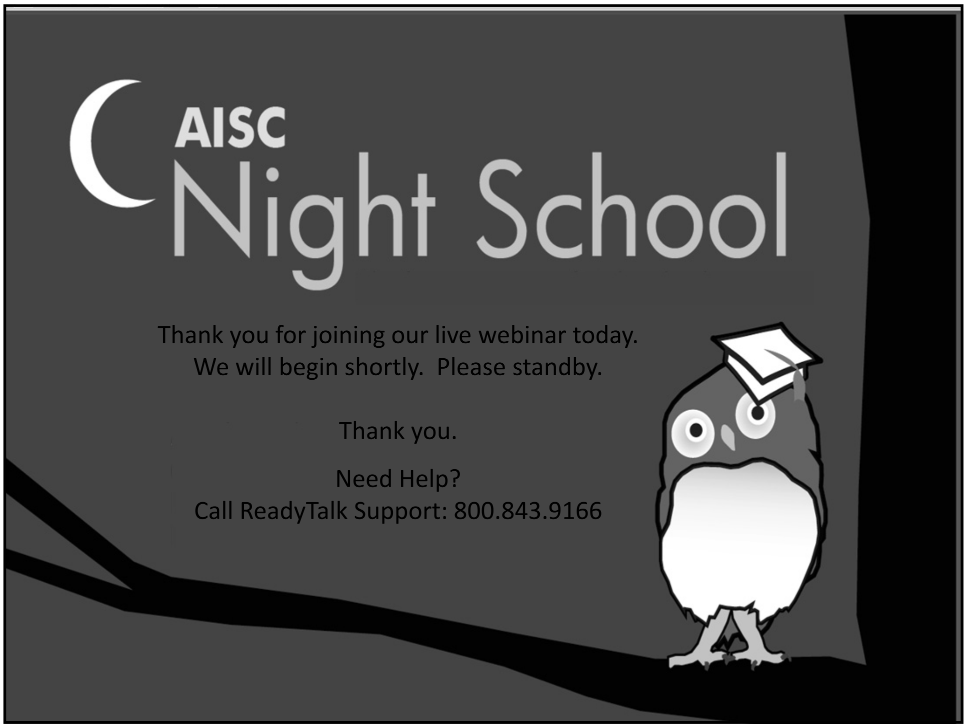

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Course Description

The AISC Direct Analysis Method from Soup to Nuts
March 2, 2015

This session presents a comprehensive overview of the Direct Analysis Method of design, which was first introduced in Appendix 7 of the 2005 AISC 13th Edition Specification and is referred to as the preferred method of design in Chapter C of the 2010 AISC 14th Edition Specification. The lecture emphasizes the key fundamental concepts and the practical application of the method to building frames.

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Learning Objectives

- Understand what the Direct Analysis Method (the DM) is all about.
- Synthesize and organize the various rules for applying the DM and the Effective Length Method (the ELM).
- Articulate the proper application of notional loads equivalent to desired out-of-plumbness effects in building structures.
- Apply the DM to a representative stability critical building frame.
- Evaluate and compare responses calculated from the DM and the ELM versus the “true” stability behavior of the example building frame.

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Stability Design of Steel Structures – Applying Modern Methods of Structural Analysis

Session 5
The AISC Direct Analysis Method from Soup to Nuts

Donald W. White, Ph.D.

structural STEEL

Course Overview

- Eight Sessions
  - Course Intro. and Modern Analysis (1 & 2)
  - Resources for Learning Stability by Analysis (3)
  - Second-Order Elastic Analysis (4)
  - Direct Analysis Method (5)
  - Low- and Medium-Rise Steel Buildings (6)
  - Advanced Application of Stability Design (7)
  - Design by Inelastic Analysis (8)
- Presented by members of the Structural Stability Research Council (SSRC)
  - Don White and Ron Ziemian
  - Great to join AISC in this effort!

The AISC DM from Soup to Nuts

- Topic 5.1 Key Concepts
- Topic 5.2 Details of the AISC Stability Design Methods (with emphasis on the DM)
- Topic 5.3 Modeling of Nominal Out-of-Plumbness (explicitly and using equivalent notional lateral loads)
- Topic 5.4 Example Design of a Single-Story Multi-Bay Frame
- Topic 5.5 Comparison to Test Simulation Results, Synthesis of Concepts

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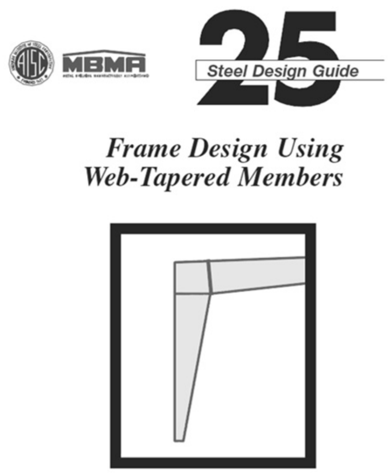
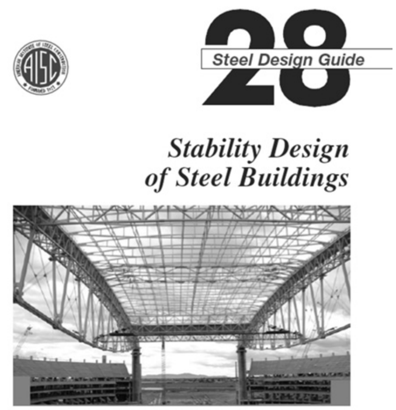
Source Materials





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The AISC DM from Soup to Nuts **Don White**

TOPIC 5.1

KEY CONCEPTS, AISC CHAPTER C
GENERAL REQUIREMENTS FOR FRAME STABILITY DESIGN

Chapter C, AISC 2010

- Lists three sanctioned methods of stability **design**
  - Direct Analysis Method (DM)
  - Effective Length Method (ELM)
  - First-Order Analysis Method (FOM)
- Specifies the DM as the "preferred" approach

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Meaning of "Design" in AISC Spec.

- Design =
  - ***Selection*** of a structural configuration
  - ***Analysis*** of the configuration to determine ***required strengths*** of components
  - ***Proportioning*** of components to have adequate ***available strength***
- With the exception of design by high-end test simulation (permitted by AISC Appendix 1), all the design methods include:
  - Requirements for ***structural analysis***
  - Rules for ***checking of component resistances*** based on the analysis results
- The rules for checking vary as a function of the analysis requirements

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In a few words …

What is the Direct Analysis Method all about?

- Assessment of "*overall*" system structural stability, including the effects of
  - Geometric imperfections &
  - Reduction in stiffness, due to yielding, as the structure's strength limit is approached

  explicitly within a 2nd-order load-deflection structural analysis

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Why is the DM Preferred?

- More straightforward
  - No *K* factors are required (… the members may be considered as "braced" in the resistance calculations)
  - Real world effects are better accounted for
- More general
  - Applies to all types of structures

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Section C1 – General Requirements

- Any method of design for stability that ***considers*** all of the following is permitted:
  1) All component limit states
  2) Flexural, shear & axial deformations (in all components)
  3) Second-order effects (P-Δ & P-δ)
  4) Geometric imperfections (P-Δ_o & P-δ_o) (out-of-alignment & out-of-straightness)
  5) Stiffness reductions due to inelasticity (including residual stress effects → increases in deformations)
  6) Uncertainty in stiffness & strength
- Let's scrutinize each of these requirements

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(1) ALL COMPONENT LIMIT STATES

- Member axial resistance (P_c)
  – Flexural buckling about the x-axis
  – Flexural buckling about the y-axis (sym. members)
  – Torsional buckling (sym. members, in some cases)
  – Torsional-flexural buckling (i.e., combined y-axis flexure & torsion, singly-sym. members)
  – Constrained-axis torsional buckling (addressed by DG25)
- Member flexural resistance (M_c)
  – Lateral-torsional buckling
  – Flange local buckling
  – Tension flange yielding
- Beam-column P-M strength interaction
- Other component resistances (connections, panel zones, stability bracing, etc…)

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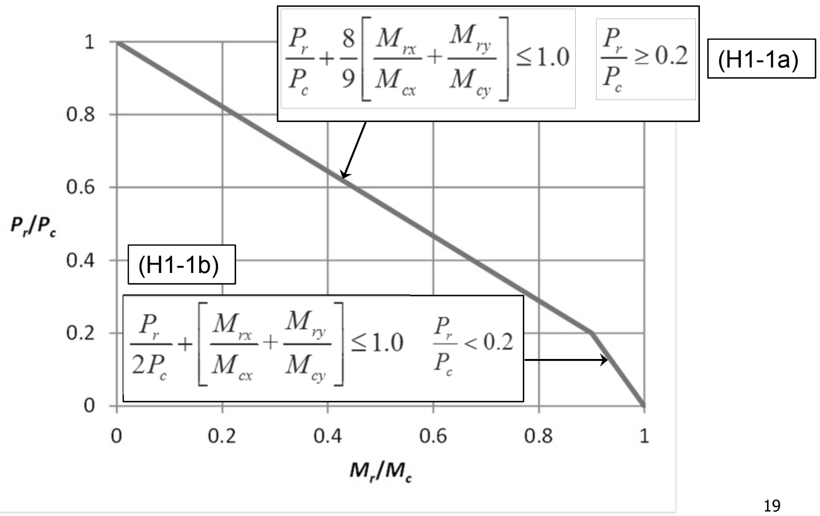
Base AISC Strength Interaction Eqs.

AISC Section H1.1: Doubly and Singly Symmetric Members in Flexure & Compression



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Definition of Terms (ASD)

$P_r = P_a$ = required axial strength (ASD)

$P_c = P_n/\Omega_c$ or P_n/Ω_t = allowable axial strength

$M_r = M_a$ = required flexural strength (ASD)

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

$\Omega_c = \Omega_b = 1.67$

$\Omega_t = 1.67$ (tension yielding)

Required strength determined based on Chapter C requirements

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Definition of Terms (LRFD)

$P_r = P_u$ = required axial strength (LRFD)

$P_c = \phi_c P_n$ or $\phi_t P_n$ = design axial strength

$M_r = M_u$ = required flexural strength (LRFD)

$M_c = \phi_b M_n$ = design flexural strength

$\phi_c = \phi_b = 0.9$

$\phi_t = 0.9$ (tension yielding)

Required strength determined based on Chapter C requirements

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(2) FLEXURAL, AXIAL & SHEAR DEFORMATIONS

- Included ***where important*** by modeling the corresponding component flexibilities in the analysis, e.g.,
  – Member or panel zone shear deformations
  – Connection deformations
  – Diaphragm deformations
  – Column or brace axial deformations

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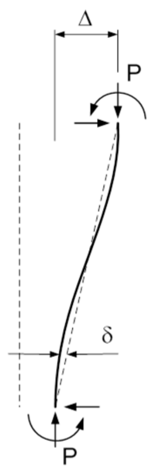
(3) SECOND-ORDER EFFECTS



PΔ = Additional moment (couple) due to the axial force acting through the relative transverse displacement of member (or segment) ends

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Second-Order Effects

PΔ = Additional moment (couple) due to the axial force acting through the relative transverse displacement of member (or segment) ends

Pδ = Additional moment due to the axial force acting through the transverse displacement relative to the member (or segment) chord

Any 2^{nd}-order analysis method is permitted, as long as it is sufficient, e.g.:

- Amplified 1^{st}-order analysis by the AISC *Appendix 8* method
- Other amplified 1^{st}-order analysis methods
- Various P-Δ analysis methods
- Explicit general purpose 2^{nd}-order analysis methods

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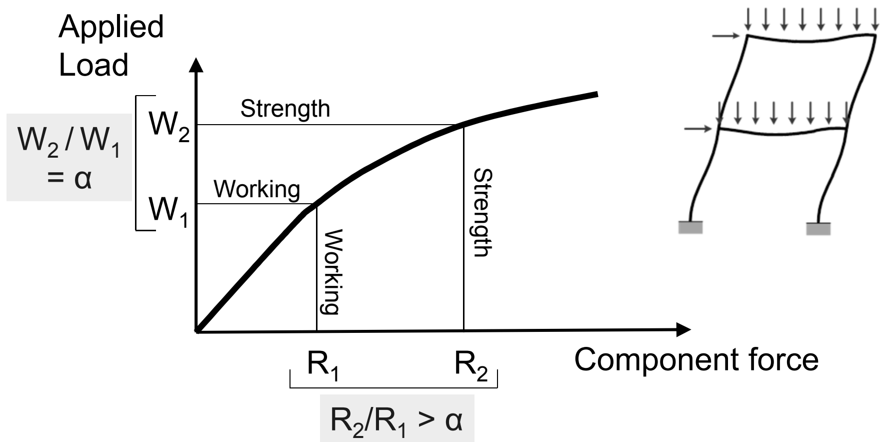
2nd-Order Analysis with ASD Loads



Due to 2nd-order effects, the internal forces are NOT proportional to the applied loads

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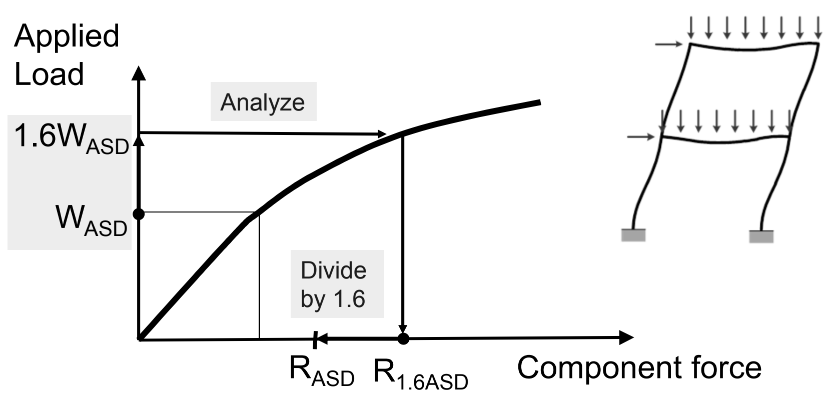
2nd-Order Analysis with ASD Loads



When using an explicit 2nd-order analysis:
multiply the loads by α = 1.6 → run the analysis → divide the results by 1.6

When using amplifiers (e.g., B_1 & B_2):
simply include α = 1.6 in the load term of the amplifier, e.g., αP_{story}, α = 1.6

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Include **ALL** Gravity & Other Loads that Influence the Structure's Stability

- … the loads in any gravity columns, walls, etc. that are stabilized by the lateral load resisting system must be included in the analysis
- This is often handled by the use of a "dummy column"

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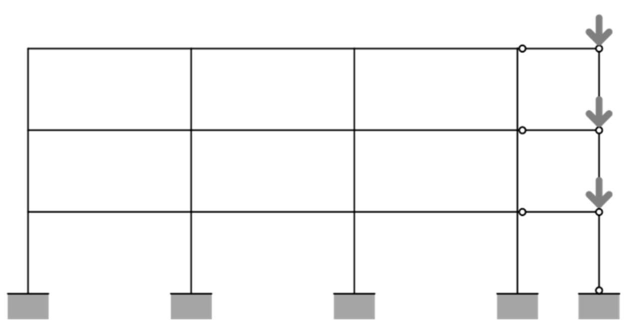
What is a Dummy Column?



A dummy column is a single pin-connected column within a frame model that is tied to each of the story levels by a pin-ended strut and is subjected at each level to the total vertical load transferred to all the columns, walls, etc. that are stabilized by the lateral load resisting frame

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(4) GEOMETRIC IMPERFECTIONS & (5) STIFFNESS REDUCTIONS

- All the AISC Specification stability design approaches are based inherently on the consideration of:
  - **Initial geometric imperfections** (Δ_o & δ_o) within fabrication & erection tolerances, as specified by the *Code of Standard Practice*
  - **Reductions in stiffness** due to the onset of yielding, including residual stress effects, prior to reaching the maximum strength

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Geometric Imperfections & Stiffness Reductions

- The traditional ELM considers these effects on overall system stability via only the member P_c values, via K factors (or alternatively, the corresponding system buckling analysis)
- This works only for moment frames, and only for the beam-columns in these frames
- For design by the ELM, the 2010 Specification also requires the consideration of sway imperfections ($\Delta_o/L = 0.002$) with *gravity-only* load combinations in the structural analysis
- **The DM accounts for these effects explicitly in the structural analysis**

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(6) UNCERTAINTY IN STIFFNESS & STRENGTH

- The ELM considers these uncertainties only via ϕ and Ω factors on the component resistances
- Part of the stiffness reduction in the DM accounts for ***uncertainty in the structure's stiffness*** at strength load levels
  - Note that structures with small 2^{nd}-order effects are insensitive to reductions in stiffness
  - This characteristic is captured inherently by the DM

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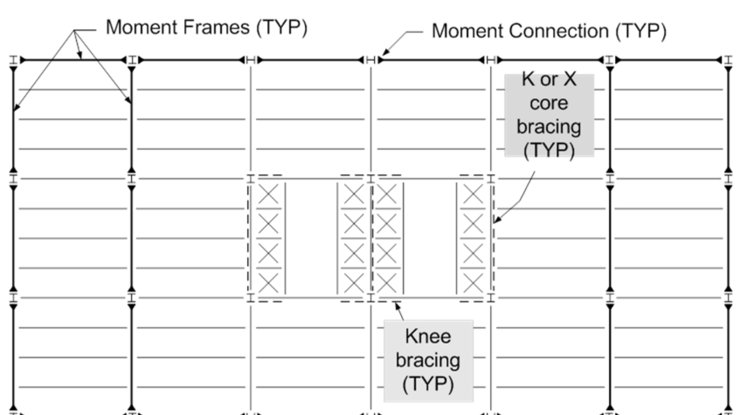
Why is the DM Preferred?

- Finding K can be complicated!



Add a few setbacks & irregularities → Forget it !!

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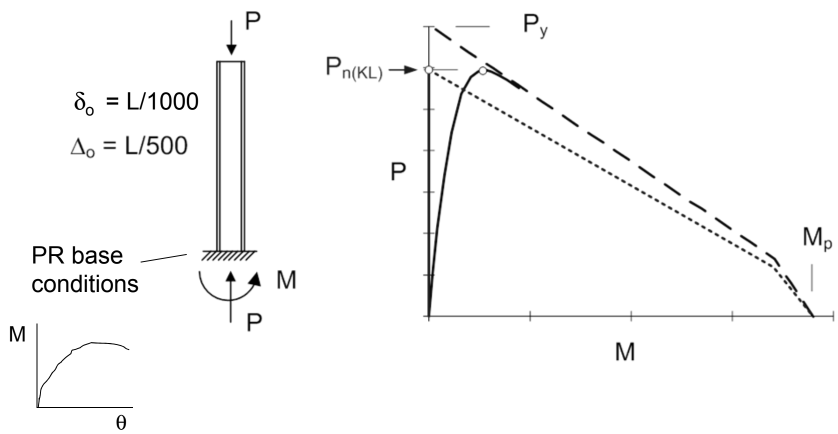
Why is the DM Preferred?

- The *physical behavior of even the most basic frames* is practically *NEVER a bifurcation problem*
- It is *always a load-deflection problem*



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Basic Concept of the DM

Estimate the required forces using a 2nd-order load-deflection analysis … accounting for:

- *nominal stiffness reductions at strength levels &*
- *geometric imperfection effects*

Check the member resistances accounting for limit states not captured by the analysis

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TOPIC 5.2

DETAILS OF THE AISC STABILITY DESIGN METHODS
(WITH EMPHASIS ON THE DM)

AISC (2010) CHAPTER C

Primary Distinction of the DM

- Handling of the assessment of "overall" system structural stability, including the effects of:
  - Geometric imperfections &
  - Reduction in stiffness, due to yielding, as the structure's strength limit is approached

  explicitly within a 2nd-order load-deflection structural analysis
- As a result:
  - No need for the use of K > 1 to evaluate the sidesway resistance of moment frames (i.e., the members may be considered as "braced" in the resistance calculations)
- However… the above global 2nd-order analysis still doesn't cover all the bases with respect to the component strength limit states
  - Component resistance eqs. are applied w/ P_r , M_{rx}, M_{ry} to account for limit states not captured by the analysis

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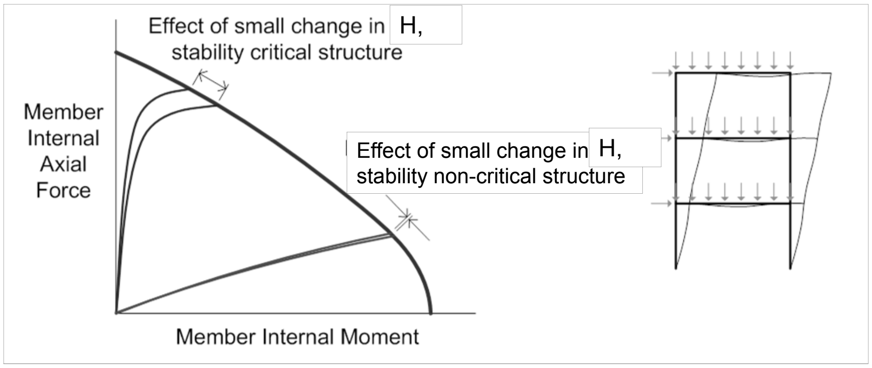
Stability Critical/Non-Critical Structures

- For stability critical structures, a small change in H can produce a large change in the vertical load carrying capacity
- AISC definition of stability critical: $\Delta_{2nd}/\Delta_{1st} > 1.7$ ($>$ 1.5 based on unreduced stiffness)



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Stability Non-Critical Structures

Analysis Requirements

DM	ELM
Model $\Delta_o = \frac{L}{500}$ for gravity-only load combinations (explicitly or using notional loads) AND Use 0.8E and τ_bI, where τ_b = 1 for $\frac{\alpha P_r}{P_y} \le 0.5$ $\tau_b = 4\frac{\alpha P_r}{P_y}\left(1-\frac{\alpha P_r}{P_y}\right)$ for $\frac{\alpha P_r}{P_y} > 0.5$ **Eq. (C2-2b)** For other materials, use 0.8E or use smaller elastic stiffness if required by the governing code	Model $\Delta_o = \frac{L}{500}$ for gravity-only load combinations (explicitly or using notional loads) AND Use nominal elastic stiffness
or Model $\Delta_o = 0.003L$ for gravity-only load combinations AND Use 0.8E (use smaller elastic stiffness for other materials if required by the governing code)	

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Stability Non-Critical Structures

Resistance Calculation Requirements

DM	ELM
Calculate P_c using $K = 1$ *	Calculate P_c based on "system" buckling
or, if $\frac{\alpha P_r}{P_{eL}} \leq 0.1$ or if $\delta_o = 0.001L$ is modeled, P_c may be calculated for flexural buckling limit states using $K = 0$ (DG25 Extension)	Except if $B_2 \leq 1.1$, P_c may be calculated using $K = 1$

* An appropriate braced column *K* value could be used, but we don't usually bother since the additional benefits of using $K < 1$ are often small

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Note on Stiffness Reduction

- *Do not reduce E in Specification Resistance Eqs.*
- *Do not reduce E in service load, response spectrum analyses, building period calculations, etc.*

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Stability Critical Structures

- ELM Not Allowed
- For the DM… Analysis Requirements

Model $\Delta_o = \frac{L}{500}$ for ALL load combinations
(explicitly or using notional loads)

AND

Use 0.8E and $\tau_b I$, where $\tau_b = 1$ for $\frac{\alpha P_r}{P_y} \leq 0.5$ **EQ. (C2-2A)**

$$\tau_b = 4\frac{\alpha P_r}{P_y}\left(1 - \frac{\alpha P_r}{P_y}\right) \text{ for } \frac{\alpha P_r}{P_y} > 0.5$$ **EQ. (C2-2B)**

For other materials, use 0.8E or use smaller elastic stiffness if required by the governing code

or Model $\Delta_o = 0.003L$ for ALL load combinations

AND

Use 0.8E (use smaller elastic stiffness for other materials if required by the governing code)

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Stability Critical Structures

- ELM Not Allowed
- For the DM… Resistance Calculation Requirements

Calculate P_c using $K = 1$*
or, if $\frac{\alpha P_r}{P_{eL}} \leq 0.1$ or if $\delta_o = 0.001L$ is modeled, Calculate P_c for flexural buckling limit states using $K = 0$

* An appropriate braced column K value could be used, but we don't usually bother since the additional benefits of using $K < 1$ are often small

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Why do we use $P_{c(L)}$ (K = 1) in the DM?

- To capture potential ***non-sway buckling*** of ***light members*** subjected to ***large axial loads***
- Alternative procedure from AISC DG 25 (extension to AISC Specification provisions):
  - The "flexural buckling" axial resistance (P_{nx} or P_{ny}) may be taken equal to $\boldsymbol{P_{n(L=0)} = QP_y}$ when
    - $\alpha P_r \leq 0.1 P_{eL}$ or
    - An appropriate $\delta_o = 0.001L$ is included in the analysis
  - Torsional, torsional-flexural and/or lateral-torsional buckling limit states still must be checked using the actual unbraced lengths (when calculating P_n & M_n)

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TOPIC 5.3

MODELING OF NOMINAL OUT-OF-PLUMBNESS
(EXPLICITLY AND USING NOTIONAL LATERAL LOADS)

AISC (2010) CHAPTER C

Nominal Out-of-Plumbness

- Use a base $\Delta_o = L/500$ … maximum tolerance from the AISC Code of Standard Practice
- Recommendations:
  - *Explicitly cant the frame geometry* if facilitated by the analysis software
    … Easier to understand and automate for general cases
  - Otherwise, apply equivalent notional loads
    $N_i = 0.002\,Y_i$ **Eq. (C2-1)**
    where Y_i = vertical load applied AT a given level

(Note: AISC (2010) specifies αY_i in Eq (C-2-1), to address the adjustment from allowable load levels to the strength load levels; Y_i is taken here as the vertical load appropriate for consideration of the stability effects from out-of-plumbness.)

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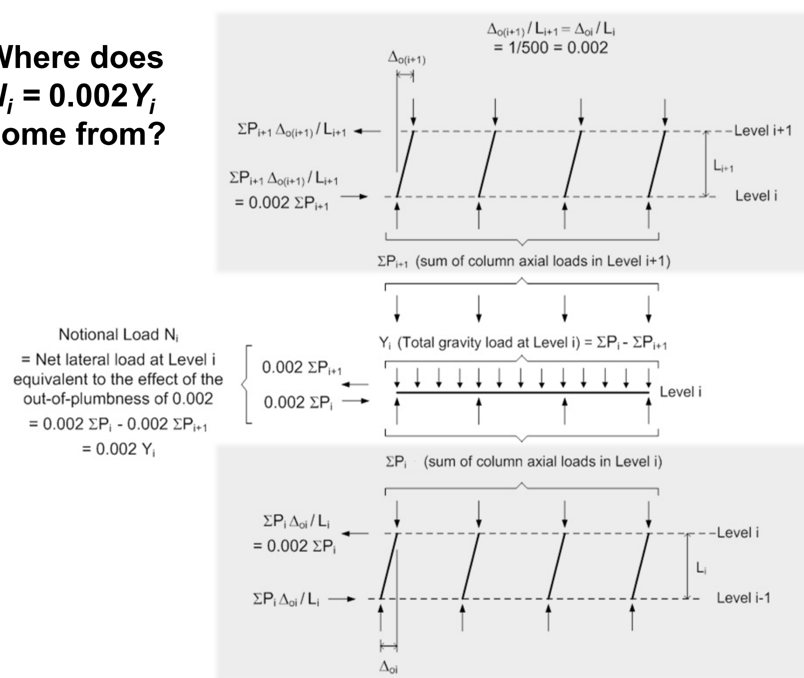
Where does $N_i = 0.002Y_i$ come from?



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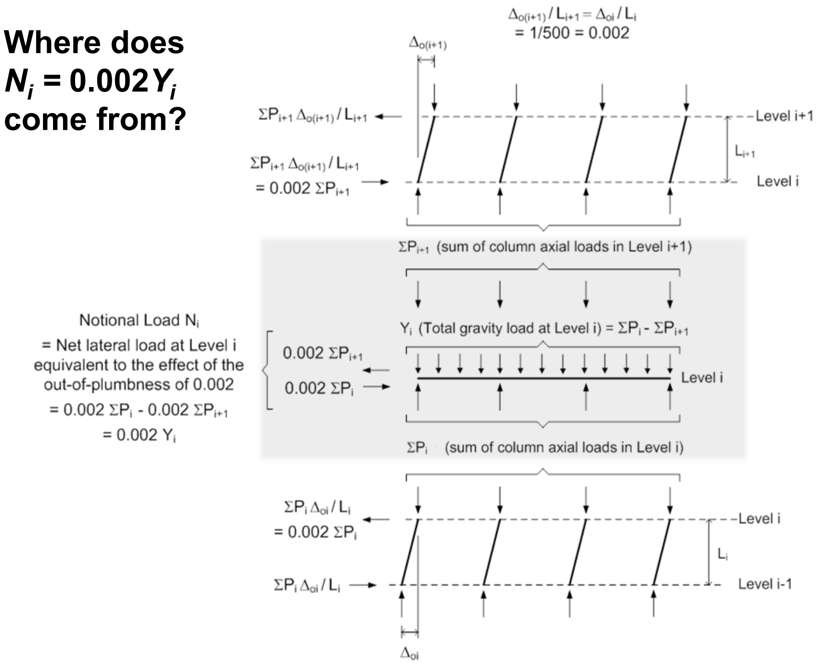
Where does $N_i = 0.002Y_i$ come from?



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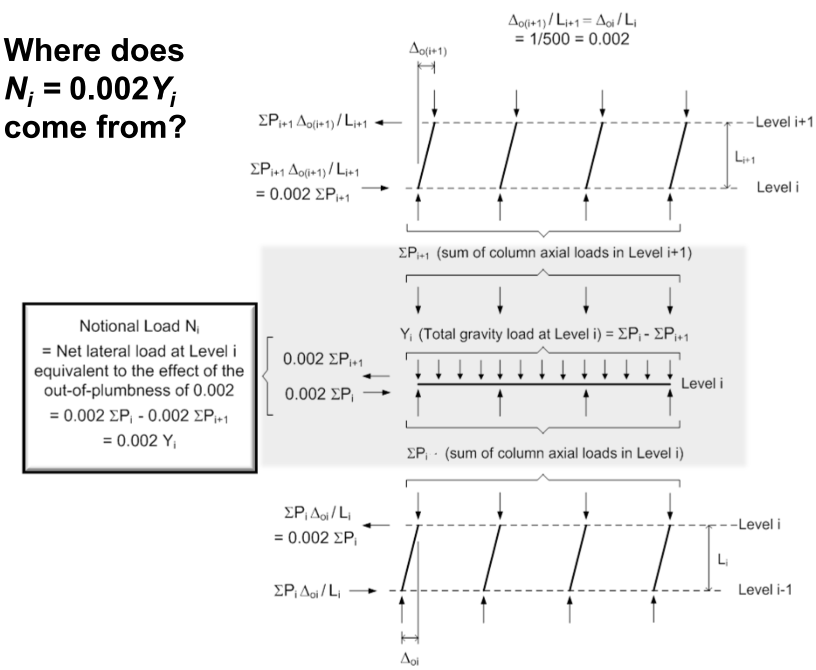
Where does $N_i = 0.002Y_i$ come from?



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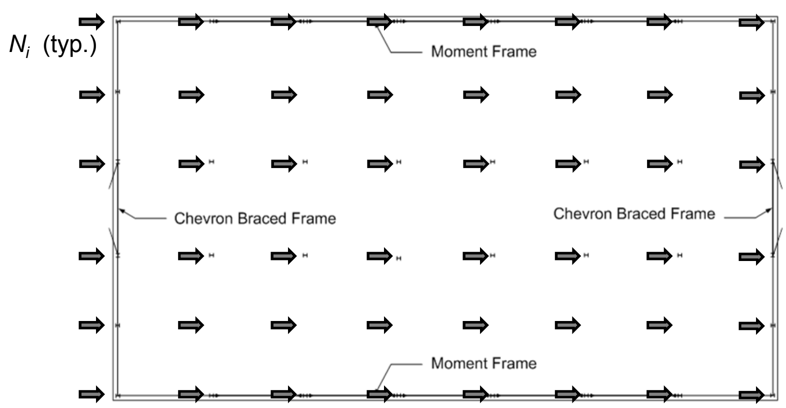
For flexible diaphragms

or any other cases where the position of the vertical load in-plan may have an important stability effect:

- Y_i = *vertical load transferred to each column, etc. at a given level*
- $N_i = 0.002Y_i$ = *notional load applied AT the corresponding plan location*



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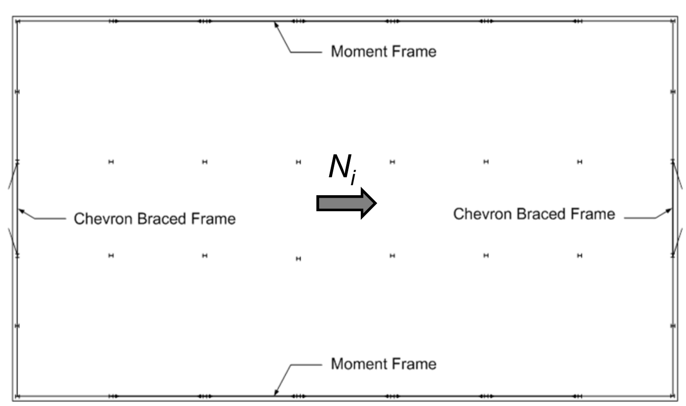
For rigid diaphragms

and in any other cases where the position of the vertical load in-plan is not of significance:

- Y_i = *total vertical load applied at a given level*
- $N_i = 0.002Y_i$ = *notional load applied AT the center of the loading*



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Application of Δ_o/L or N_i

- Direction
  - For greatest destabilizing effect
  - … Not all that difficult, but requires some thought … this is discussed further in the following slides
- Pattern
  - For structures up to about 7 to 10 stories, supporting gravity loads primarily through nominally-vertical columns, walls or frames, …

    Use a uniform Δ_o/L (or the corresponding N_i) in the same direction at all levels

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Application of Δ_o/L or N_i

- For gravity-only load combinations
  - Apply the Δ_o/L or N_i independently in the two orthogonal orientations approximating the principal stiffness orientations of the structure
    - "Independently" means one orientation at a time
    - Off-axis notional loads (i.e., diagonal to the approximate principal stiffness orientations) need not be considered
  - No torsional notional load, or corresponding initial twist of the structure, is required

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Application of Δ_o/L or N_i

- For gravity-only load combinations
  - Apply the Δ_o/L or N_i in the + & – directions along the above axes (same direction at all levels)… total of four Δ_o/L or N_i cases
  - For simple cases, where:
    1) The structure exhibits zero sidesway, and
    2) Symmetry of the design is enforced by *grouping* of the design selections,

    … apply the Δ_o/L or N_i just in the + directions … total of two Δ_o/L or N_i cases
  - For simple cases where the structure's sidesway deformations are in the same direction in all of its levels

    … apply the Δ_o/L or N_i in the + or – direction that increases the net sidesway deformation in each level… total of two Δ_o/L or N_i cases
    - This is the direction that increases the ***overall*** destabilizing effects
    - Generally, one need not apply the Δ_o/L or N_i in a direction opposite from the structure's sidesway deflections (e.g., to minimize the reduction in the internal forces in certain components due to the sidesway effects)

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Application of Δ_o/L or N_i

- For lateral load combinations
  - Apply the Δ_o/L or N_i in the direction of the resultant of all the lateral loads
  - No torsional notional load, or corresponding initial twist of the structure, is required … the ASCE 7 eccentric wind & accidental seismic eccentricity loadings are sufficient to address torsional concerns

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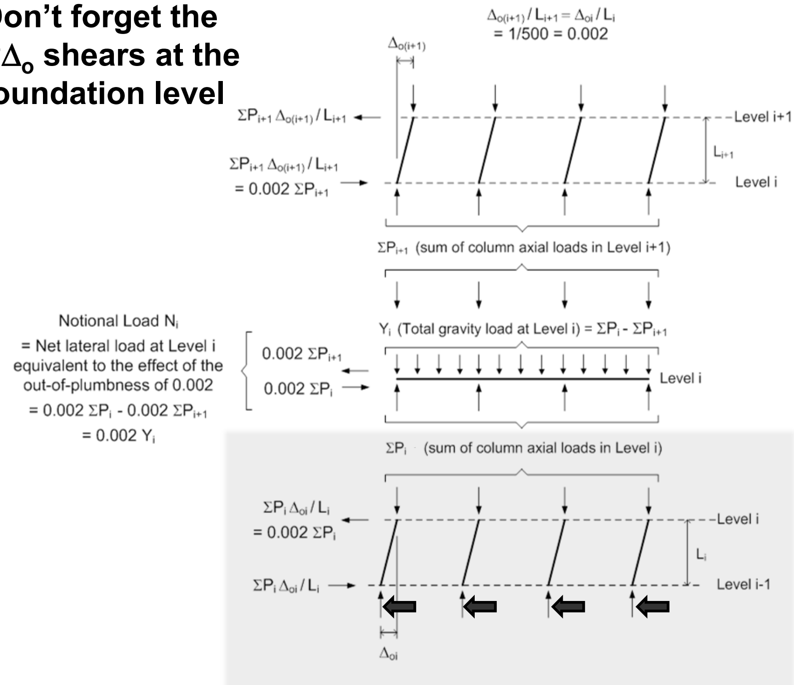
Don't forget the $P\Delta_o$ shears at the foundation level



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Notional Load Effects at the Foundation Level

- Notional loads produce small, fictitious base shears
- This is because they are derived from the story $P\text{-}\Delta_o$ shears associated with the out-of-plumb geometry (AISC doesn't specify the $P\text{-}\Delta_o$ shears at the base)
- The ***total*** base shear due to out-of-plumbness is always zero
- The correct horizontal reactions at the foundation may be estimated by correctly applying the $P\text{-}\Delta_o$ horizontal forces at the base of the structure… These forces are:
  - Equal and opposite to the sum of all the notional loads, &
  - Distributed among the vertical load-carrying components in the same proportion as the gravity load supported by those components

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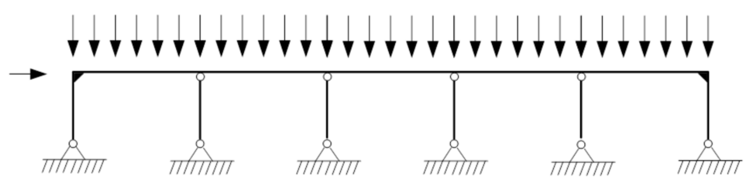
TOPIC 5.4

EXAMPLE DESIGN OF A SINGLE-STORY
MULTI-BAY MOMENT FRAME

Example Multi-Bay Moment Frame



Exterior columns:	W12x65	F_y = 50 ksi
Interior columns:	W8x31	E = 29,000 ksi
Beams:	W24x62	
Beam span lengths:	40 ft	
Story height:	20 ft	

Unfactored Loads:	D = 1.0 kip/ft	LRFD Load Combinations:
	S = 1.2 kip/ft	LC1: 1.2D + 1.6S
	W = 16 kips	LC2: 1.2D + 0.5S + 1.0W

Columns braced out-of-plane, top & bottom
Girders braced @ 5 ft cc by purlins (& flange diagonals where req'd)

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Member Properties

- W12x65
  - $A = 19.1\ in^2$
  - $I_x = 533\ in^4$, $r_x = 5.28$ in, $Z_x = 96.8\ in^3$
  - $r_y = 3.02$ in, $L_p = 11.9$ ft, $L_r = 35.1$ ft, BF = 5.41
  - $P_y = 955$ kips
  - $\phi_b M_{n\ max} = 356$ ft-kips = 4272 in-kips (noncompact flange)
  - $\phi_b M_r = 231$ ft-kips = 2772 in-kips
- W24x62
  - $A = 18.3\ in^2$
  - $I_x = 1550\ in^4$, $r_x = 9.23$ in, $Z_x = 153\ in^3$
  - $r_y = 1.37$ in, $L_p = 4.87$ ft, $L_r = 14.4$ ft
  - $\phi_b M_p = 574$ ft-kips = 6885 in-kips
  - $\phi_b M_r = 344$ ft-kips = 4128 in-kips

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Big Picture

- Sidesway stability effects can be very substantial in this type of frame
  - Wide footprint
    … therefore, relatively large total gravity load
  - Small lateral load requirements
    … therefore, relatively small lateral stiffness
- Large gravity load & Small lateral stiffness
  → Large second-order sidesway effects

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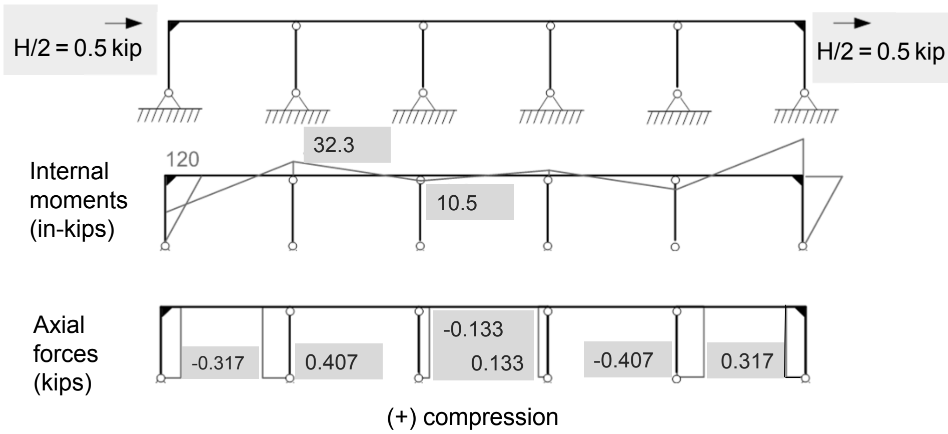
1^{st}-Order Unit Lateral Load Analysis



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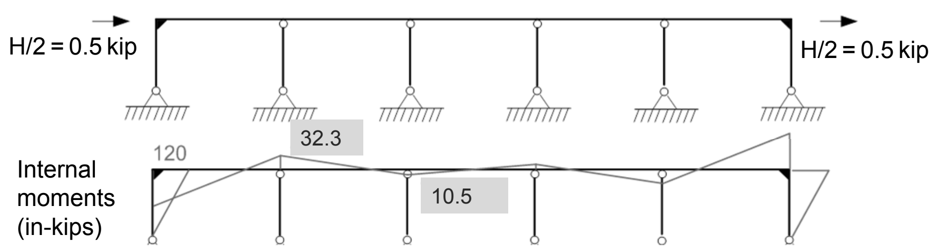
1^{st}-Order Unit Lateral Load Analysis



First-order sidesway deflection due to H: Δ_H = 0.2968 in ← Using reduced stiffness (0.8E)

First-order story sidesway stiffness in terms of the column chord rotations Δ_H/L : $P_{Lstory} = H / (\Delta_H/L)$

P_{Lstory} = **808.6 kips/rad**

Individual contributions of exterior lateral-load resisting columns to the first-order story sidesway stiffness: $P_L = P_{Lstory} / 2$

$P_L = \beta_L (0.8EI_c) / L^2$ → solve for β_L → β_L = **1.884**

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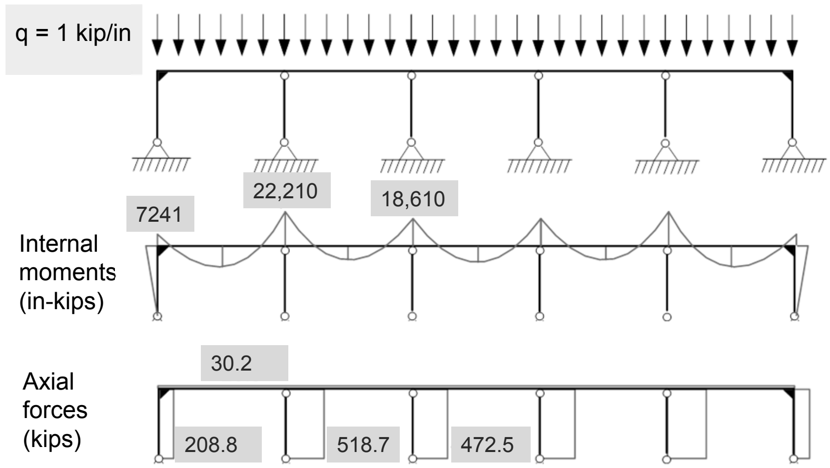
1st-Order Unit Gravity Load Analysis



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GETTING DOWN TO BUSINESS

- Consider Load Combination 1 (1.2D + 1.6S)
- Design by the DM → Analysis requirements:
  - Nominal elastic stiffness reduction
  - Nominal out-of-plumbness
- Use the Amplified Story Drift Method to calculate M_u & P_u
- Compare the results with the AISC “B1-B2” (“NT-LT”) Method
- Compare to General Purpose analysis results

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Determine story sidesway amplifier B_2

q_u = 3.12 kips/ft = 0.260 kips/in

P_{story} = q_u x 5 x 40 ft = **624 kips**

Leeward column $P_u \cong 208.8\, q_u$ = **54.3 kips** → $P_{mf} = 2\,P_u$ = **109 kips**

$R_M = 1 - 0.15\, P_{mf} / P_{story}$ = **0.974** **Eq. (A-8-8)**

Check column axial load level… P_u / P_y = **0.057** ≤ 0.5 → τ_b = **1.0**

Story buckling load : $P_{estory} = R_M\, P_{Lstory}$ P_{estory} = **787.6 kips** **Eq. (A-8-7)**

$$B_2 = \frac{1}{1-\dfrac{P_{story}}{P_{estory}}} = \frac{1}{1-\dfrac{624}{787.6}} = \mathbf{4.814}$$ **Eq. (A-8-6)**

Note: this value of B_2 should raise some red flags, but lets continue

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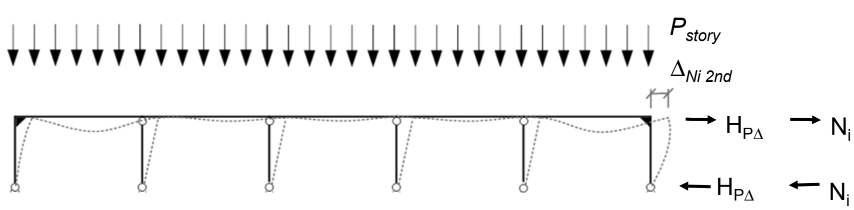
Determine Column Internal Forces

Notional Load: $Y_i = P_{story}$ = 624 kips → $N_i = 0.002\, Y_i$ = **1.248 kips** **Eq. (C2-1)**

1st-order lateral deflection (due to Notional Load, including effect of reduced stiffness): $\Delta_{Ni\,1st} = 0.2968\,(N_i / 1.0)$ = **0.3703 in**

2nd-order lateral deflection (including effect of reduced stiffness):

$\Delta_{Ni\,2nd} = B_2\, \Delta_{Ni\,1st}$ = **1.782 in**



Story PΔ shear force: $H_{P\Delta} = P_{story}\, \Delta_{Ni\,2nd} / L$ = **4.633 kips**

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Determine Column Internal Forces

Leeward column 2nd-order axial force: $P_u = 54.3 + (N_i + H_{P\Delta}) \times 0.317 =$ **56.2 kips**

Determine column non-sway amplifier B_1:

$P_{e1} = \pi^2 (0.8E)\, I_x / L^2 = \pi^2 \times 23{,}200 \times 533 / 240^2 =$ **2118 kips** — **Eq. (A-8-5)**

→ $C_m =$ **0.6** → $B_1 = \dfrac{C_m}{1 - \dfrac{P_u}{P_{e1}}} \geq 1.0$ → $B_1 =$ **1.0** — **Eqs. (A-8-4) (A-8-3)**

Maximum 2nd-order moment at top of leeward column:

$M_u = B_1 \,[\, 7241 \times 0.260 + 120 \times (N_i + H_{P\Delta})\,] =$ **2588 in-kips**

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Column Forces by AISC NT-LT Method

(Appendix 8, Section 8.2)

$B_1 =$ **1.0**

$B_2 =$ **4.814**

$M_{nt} =$ **7241 x 0.260 = 1880 in-kips**

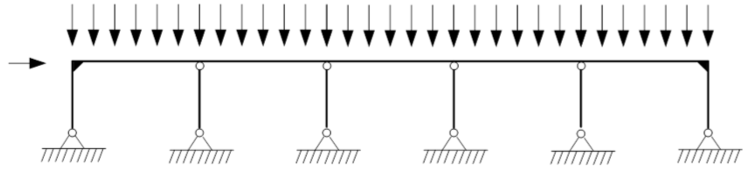
$M_{lt} =$ **120 x 1.248 = 150 in-kips**

$M_u = B_1 M_{nt} + B_2 M_{lt}$ **= 2602 in-kips**

$P_{nt} =$ **208.8 x 0.260 = 54.3 kips**

$P_{lt} =$ **0.317 x 1.248 = 0.4 kips**

$P_u = P_{nt} + B_2 P_{lt}$ **= 56.2 kips**



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Comparison to General Purpose Second-Order Analysis Results

Analysis Method	P_u (kip)	M_u (in-kip)	$\Delta_{2nd} / \Delta_{1st}$
Amplified Story Drift	56.2 (1.005)	2588 (1.015)	B_2 = 4.814
AISC NT-LT	56.2 (1.005)	2602 (1.020)	B_2 = 4.814
General Purpose	55.9	2550	4.515*

* Measured at the mid-width of the frame

The general purpose analysis is simpler to apply (given an initial design) and avoids sacrificing any accuracy

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Leeward Column Resistance

W12x65: L/r_y = 240 / 3.02 = 79.5 → $F_{ey} = \pi^2 E / (L/r_y)^2$ = **45.3 ksi** **Eq. (E3-4)**

↑ WA flexural buckling governs ↑ No stiffness reduction here

$f_u = P_u / A_g$ = 56.2 / 19.1 = 2.94 ksi → f_u / F_{ey} = **0.064** ≤ 0.1 →

use $\phi_c P_n = 0.9 P_y$ = 0.9 x 955 = 860 kips (extension to Specification)

Note: the col. flange and web are non-slender under axial compression → Q = 1

L_b = 20 ft | C_b = **1.75** | $\phi_b M_p'$ = **356 ft-kips = 4272 in-kips** | L_r = **35.1 ft**

(noncompact flange)

$\phi_b M_n = \phi_b M_p'$ = **4272 in-kips by inspection**

Eq. (C-F1-1) AISC Manual Table 3-2

$$\frac{P_u}{\phi_c P_n} = \mathbf{0.065} < 0.2 \rightarrow UC = \frac{P_u}{2\phi_c P_n} + \frac{M_u}{\phi_b M_n} = \frac{0.065}{2} + \frac{2550}{4272} = 0.63 \text{ (OK)}$$

Eq. (H1-1b)

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Important Note!

- The beam-column unity check is NOT a linear function of the applied loads
- Therefore, for example, if UC = 0.64, this does not mean that 1/0.64 – 1 = 56 % more load can be applied to the structure
  - This has never been the case; however, in the ELM, the variation in the UC with increasing load is approximately linear

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ROOF GIRDER CHECK UNDER LC1

Use fundamental statics given the girder end forces & applied loads to determine the maximum girder internal moment

Compare to the flexural resistance obtained from Ch. F of the Specification

Calculation of Girder Forces

Note: the maximum girder moments are in the outside bay on the “windward” side of the building

Axial force in column on windward side = shear at end of windward girder:
$P_{u\ col} = V_{u\ girder} = 54.3 – (1.248 + 4.633) \times 0.317 = 52.4$ kips

Girder end moment on windward side = moment at top of windward column:
$M_{u\ girder\ end} = B_1$ [7241 x 0.260 – 120 x (1.248 + 4.633)] = 1177 in-kips

Distance from end of girder to maximum positive moment location:
$L_{max} = P_{u\ col} / q_u = 52.4 / 0.260 = 202$ inches = 16.8 ft

Maximum girder positive moment, neglecting any Pδ amplification due to the girder axial force:
$M_u^+ = -M_{u\ girder\ end} + V_{u\ girder} L_{max} - q_u L_{max}^2 / 2 = 4103$ in-kips = 342 ft-kips

Maximum girder negative moment, over the 1st interior column:
$M_u^- = -M_{u\ girder\ end} + V_{u\ girder}\, 480 - q_u\, 480^2 / 2 = -5977$ in-kips = -498 ft-kips

We could calculate the girder axial force similarly; however, this force affects the UC by only about 0.01 for this frame if we include it, so let’s not bother

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“Windward” Girder Strength Check

Recall $L_b = 5$ ft, $L_p = 4.87$ ft, $L_r = 14.4$ ft
$\phi_b M_p = 574$ ft-kips, $\phi_b M_r = 344$ ft-kips = 4128 in-kips for a W24x62

The W24x62 is clearly ok in positive bending, since $\phi_b M_r > M_u^+$

Given the moment gradient in negative bending, $C_b >> 1$ & therefore,
$\phi_b M_n = \phi_b M_p = 574$ ft-kips over the 1st interior column by inspection (assuming flange braces at 5 ft cc to the interior flange in this region)

$UC = M_u^- / \phi_b M_p = 498 / 574 = 0.87$ (OK)

Note: the UC of the girder over the 1st interior column on the windward side is larger than the UC for the leeward column.

This unity check is equal to 1.0 at 1.08 x LC1.

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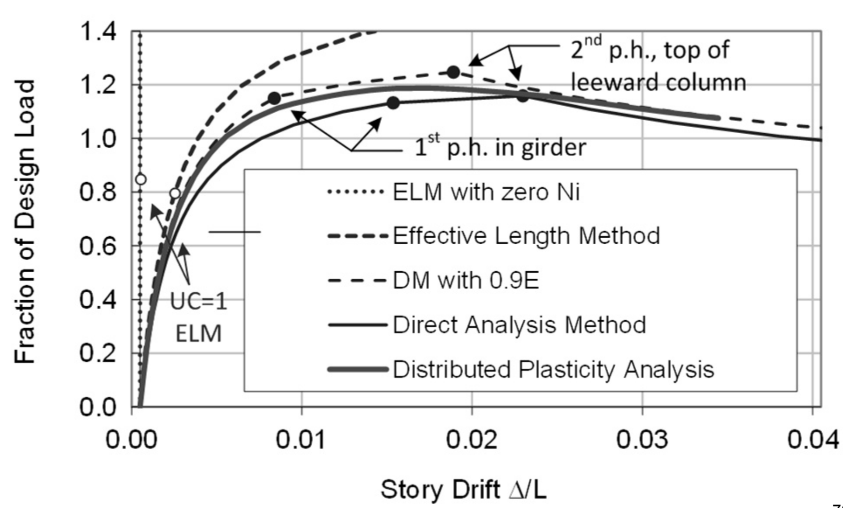
TOPIC 5.5

COMPARISON OF DM AND ELM SOLUTIONS
TO TEST SIMULATION RESULTS, SYNTHESIS OF CONCEPTS

DM, ELM & Simulation Results
Load Combination 1 (1.2D + 1.6S)



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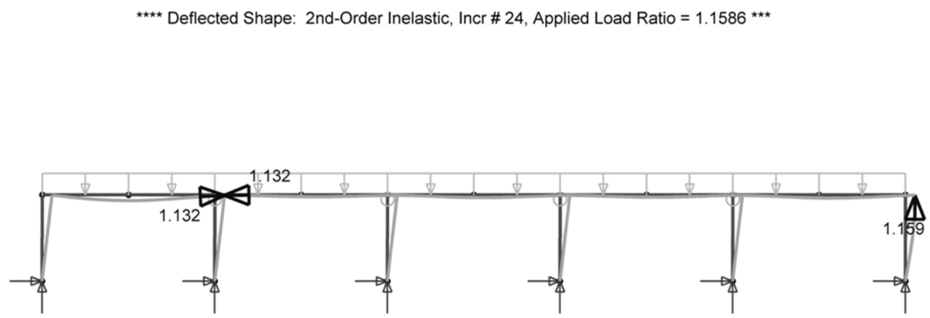
DM, Displaced Configuration at Limit Load, LC 1(1.2D + 1.6S)



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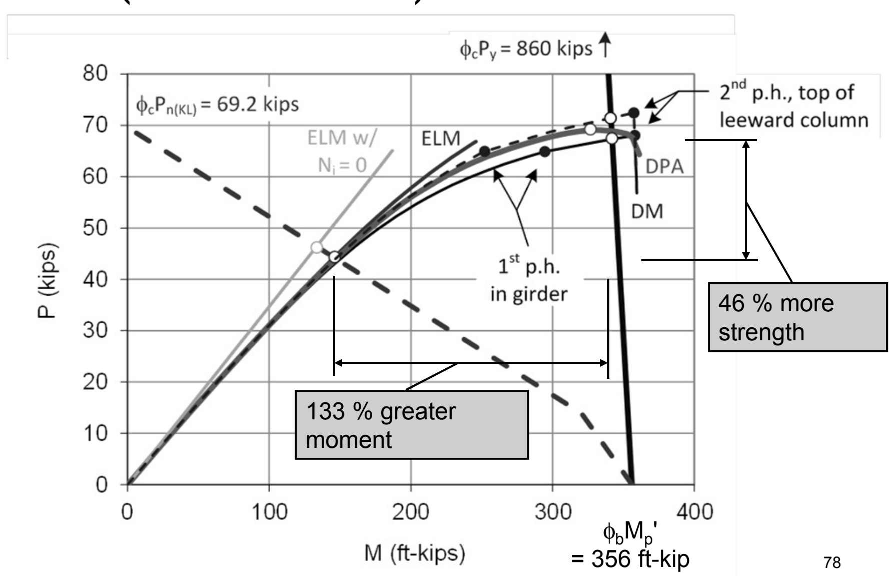
DM, ELM & Simulation Results LC 1 (1.2D + 1.6S)



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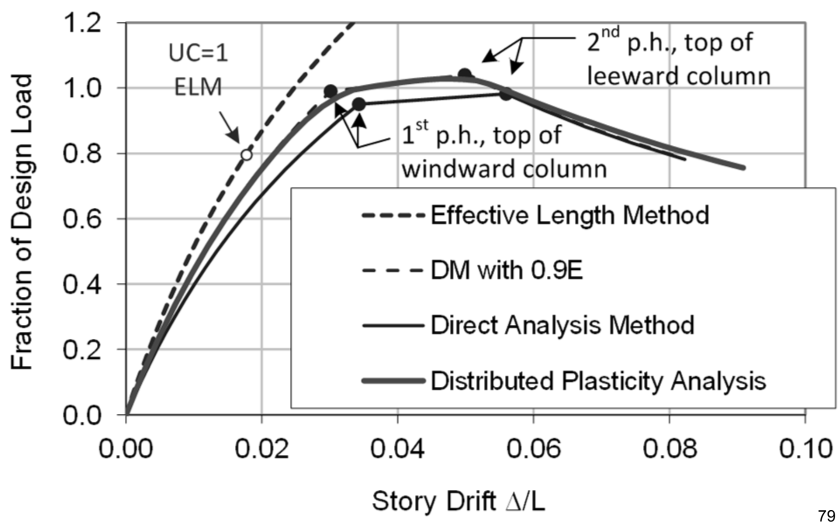
DM, ELM & Simulation Results LC 2 (1.2D + 0.5S + 1.0W)



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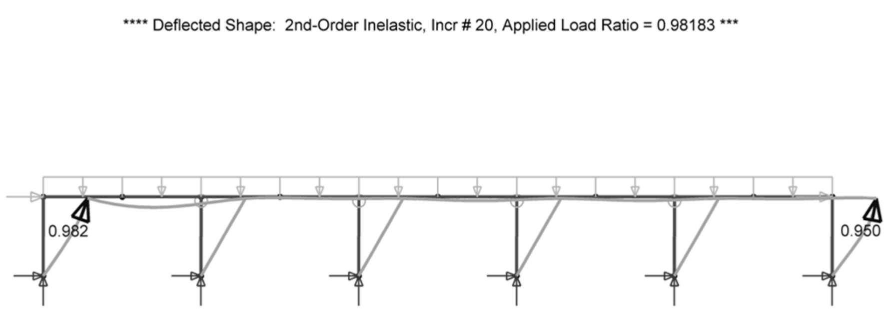
DM Displaced Configuration at Limit Load, LC 2 (1.2D + 0.5S + 1.0W)



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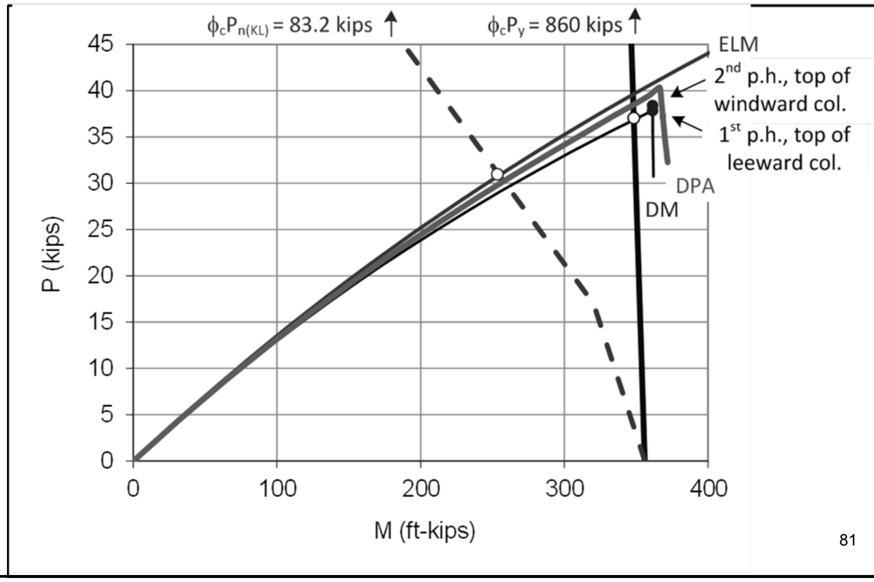
DM, ELM & Simulation Results LC2 (1.2D + 0.5S + 1.0W)



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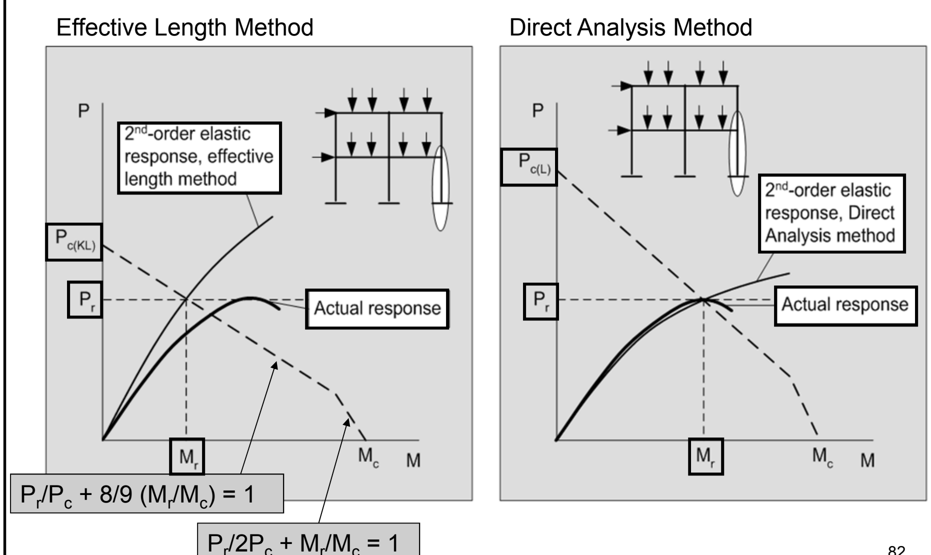
Generalized Required Forces & Beam-Column Interaction Checks



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Summary: DM vs ELM

- Both methods are legitimate
- The DM is more accurate … as long as the 2^{nd}-order analysis calculations are accurate
  - Requirements are needed to ensure P-Δ and P-δ effects are captured accurately
- The ELM requires additional restrictions to guard against its limited ability to represent the "actual" internal forces:
  - Inclusion of a minimum out-of-plumbness effect for gravity-only load combinations
  - Use of the ELM only for structures with Δ_{2nd} / Δ_{1st} < 1.5 (based on nominal elastic stiffness)

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THAT'S IT!

Thanks For Attending!!

References

- Griffis, L. and White, D.W. (2013). **Stability Design of Steel Buildings**, AISC Design Guide 28, American Institute of Steel Construction, 175 pp.
- Kaehler, R., White, D.W., and Kim, Y.D. (2011). **Design of Frames Using Web-Tapered Members**, AISC/MBMA Design Guide 25, 204 pp.
- White, D.W., Surovek, A.E., and Kim, S.C. (2007). "Direct Analysis and Design Using Amplified First-Order Analysis, Part 1 Combined Braced and Gravity Framing Systems," *Engineering Journal*, AISC, 44(4), 305-322.
- White, D.W., Surovek, A.E., and Chang, C.-J. (2007). "Direct Analysis and Design Using Amplified First-Order Analysis, Part 2 Moment Frames and General Framing Systems," *Engineering Journal*, AISC, 44(4), 323-340.

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Up Next…

- Session 6: March 9 –
  Stability Design of Low- and Medium-Rise Steel Buildings
  by D.W. White, PhD
- The application of some of the most useful techniques and procedures for stability design of steel buildings will be presented using a single relatively comprehensive building design example, all within the context of the Direct Analysis Method. The approaches emphasized will include: application of live load reduction equations to account for global reduction in second-order effects; preliminary estimation of sidesway amplification based on wind drift, seismic drift and seismic P-Δ limits; preliminary strength design; preliminary lateral stiffness design; and basic stiffness optimization based on virtual work

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Thank You

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