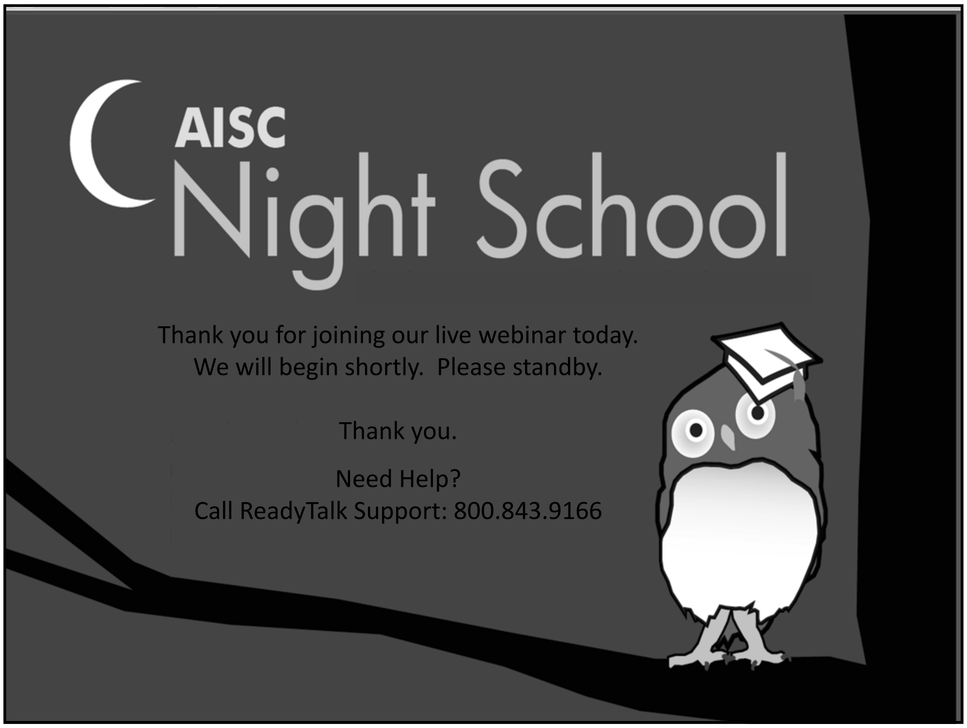

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Course Description

Stability Design of Low- and Medium-Rise Steel Buildings
March 9, 2015

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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Learning Objectives

- Assess the application of the AISC stability design provisions in the context of a somewhat realistic multi-story building example
- Highlight the importance of story sidesway stiffness to the stability design of building structures
- Understand how service wind drift limits, and ASCE 7 seismic drift and seismic P-Δ limits, impact the sidesway second-order effects in buildings
- Apply an a more rational live load reduction procedure that avoids unjustified conservative global P-Δ moments
- Calculate a proper limit on the 1^{st}-order service wind drift ratio such that a target 2^{nd}-order service wind drift ratio is not exceeded
- Perform a fast "back-of-the-envelope" preliminary strength design
- Apply useful rules of thumb to attain target story stiffnesses in sway moment frames
- Avoid conservative assumptions that impact drift estimates

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

Session 6
Stability Design of Low- and Medium-Rise Steel Buildings

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!

Stability Design of Low- and Medium-Rise Steel Buildings

- Topic 6.1 Overview of Example Building
- Topic 6.2 Design of Braced Frames
  - Estimating B_2
  - Live Load Reduction Considerations
- Topic 6.3 Design of Moment Frames
  - 1^{st}-order wind drift limits accounting for 2^{nd}-order effects
  - Characteristics of upper-bound B_2
  - Fast "back-of-the envelope" preliminary strength design
  - Story stiffness design

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Stability Design of Low- and Medium-Rise Steel Buildings — Don White

TOPIC 6.1

OVERVIEW OF EXAMPLE MULTI-STORY STEEL OFFICE BUILDING

MYTHICAL CIRQUE DU SOLEIL (CDS) BUILDING, ATLANTA, GA

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Overarching Design Attributes

- Specifications & Standards
  - ANSI/AISC 360-10 (LRFD)
  - ASCE/SEI 7-10
  - IBC 2012
- Low-rise, rigid, regular shaped, simple diaphragm, enclosed building, Risk Category II, $I_s = I_e = 1.0$
- Building location: 17th St. & Northside Drive, Atlanta (Latitude: 33.8° N, Longitude: -84.4° W)
- Wind: Exposure B, $K_{zt} = 1.0$, Directional procedure
- Seismic: Site Class C, $S_{DS} = 0.151g$, $S_{D1} = 0.103g$, Seismic Design Category B, System not specifically detailed for seismic resistance ($R = \Omega_o = C_d = 3$), All the diaphragms classify as rigid

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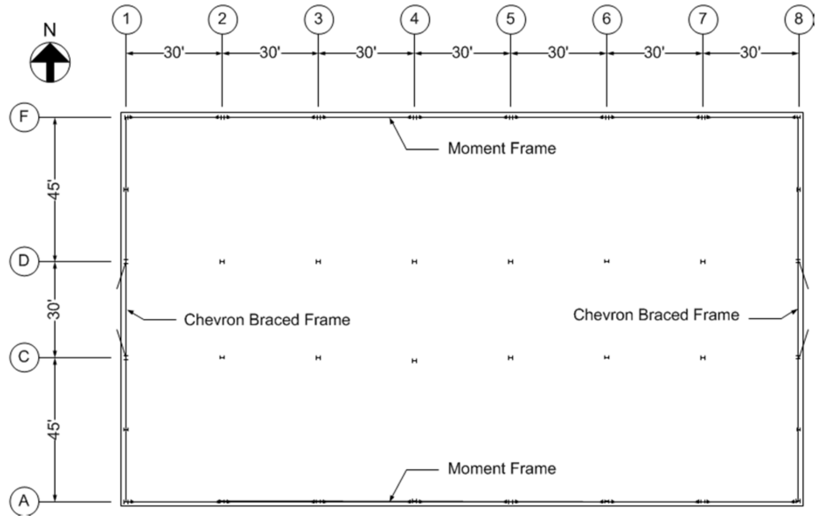
Basic Building Layout

(Credits: Geschwindner (2012))



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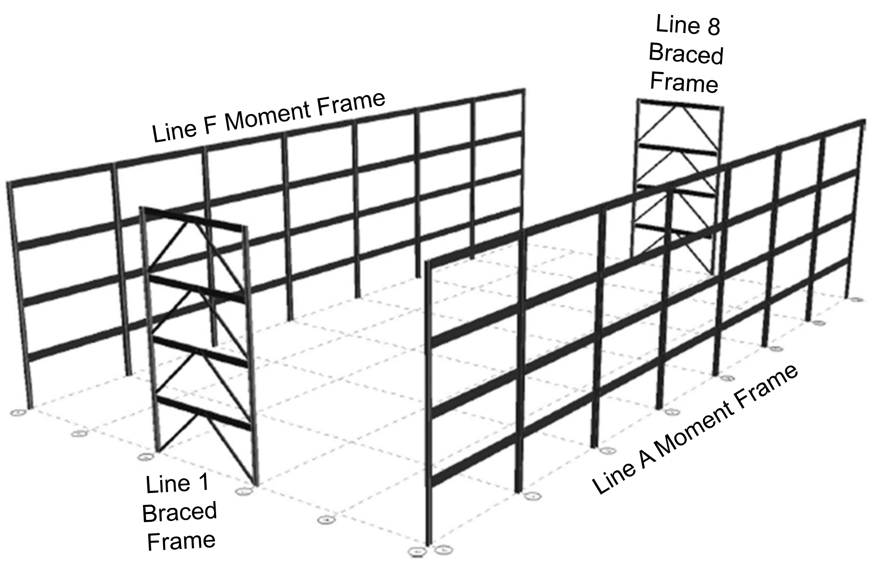
Lateral Load Resisting Systems



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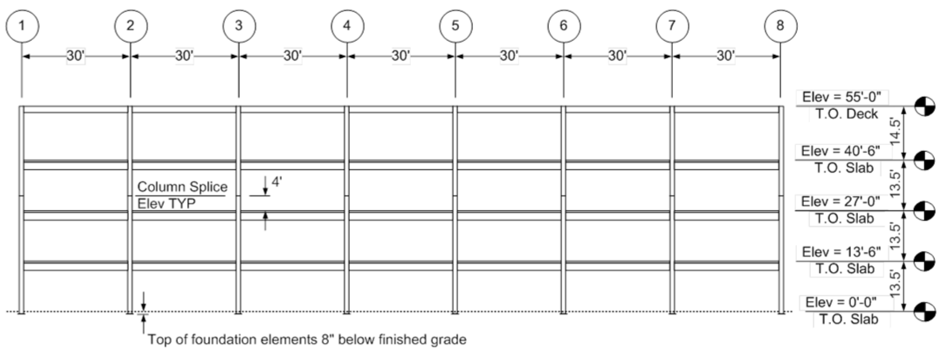
Moment Frames on Col. Lines A & F



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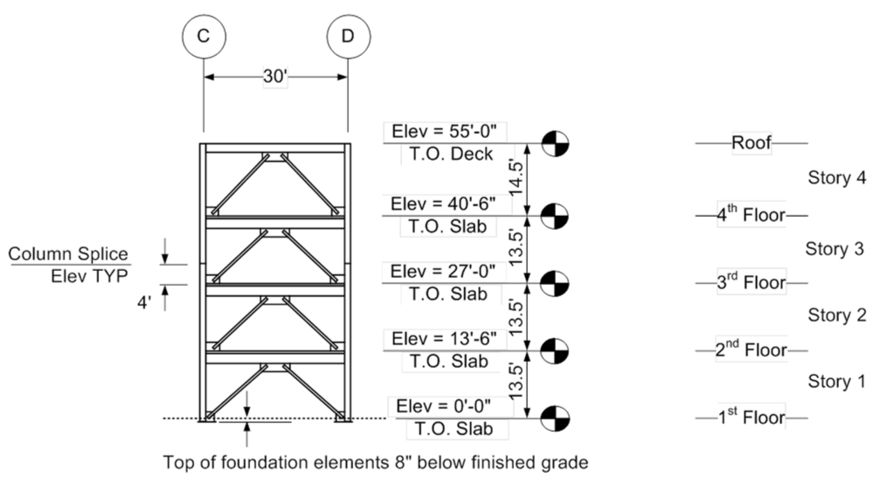
Braced Frames on Col. Lines 1 & 8



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Lateral and Gravity Load Systems

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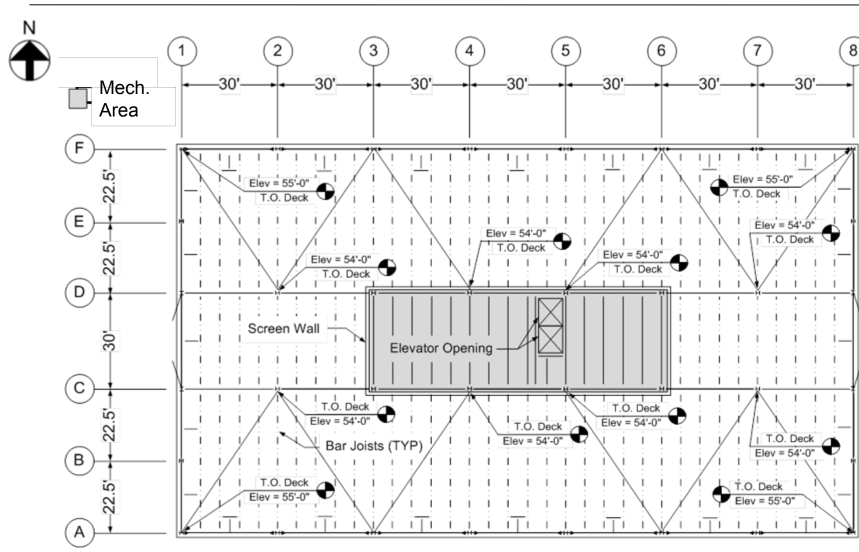
Roof Framing Plan & Live Load



20 psf uniform roof live load (reducible to as low as 12 psf for members with tributary area > 600 ft^2) governs over uniform snow & rain loads, Uniform snow + rain on snow = 8.5 psf

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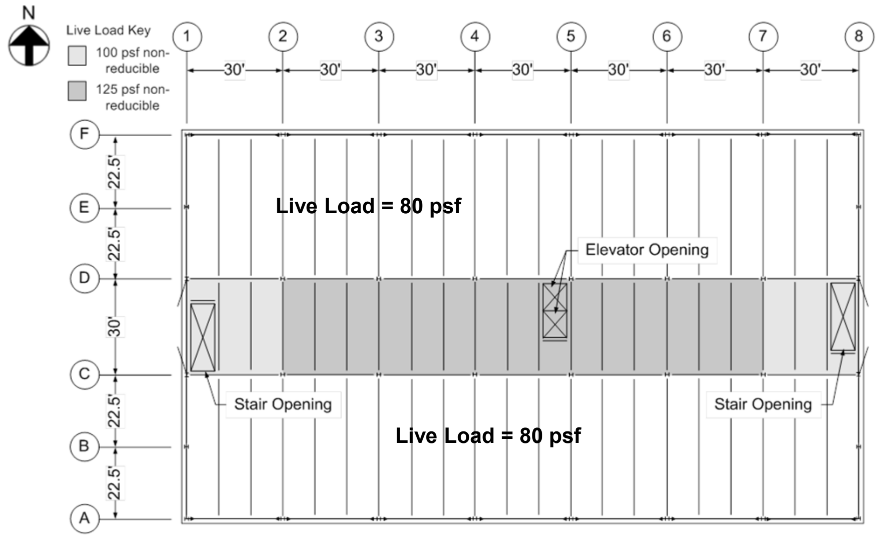
3rd & 4th Floor Framing Plan & Live Load



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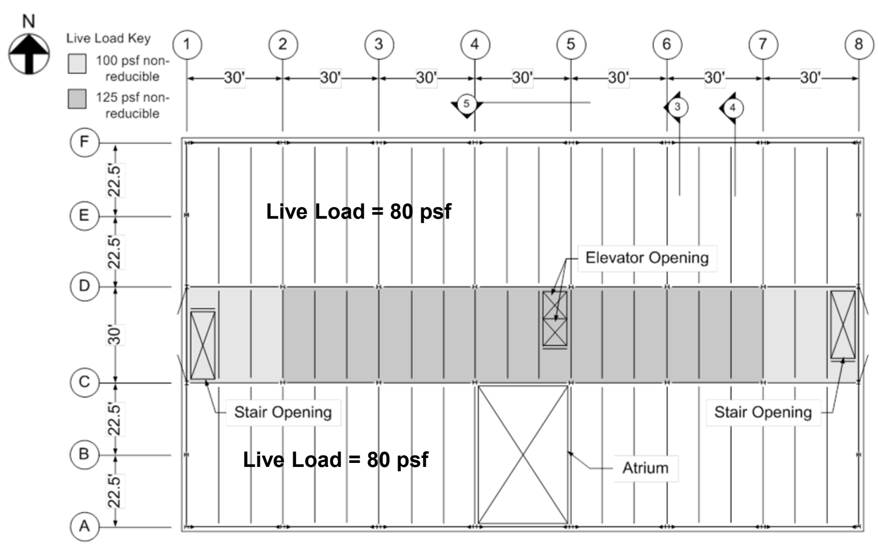
2nd Floor Framing Plan & Live Load



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Roof and Floor Plan Particulars

- 6 inch slab overhangs on the outer column lines and at slab openings
- Building envelope 9.5 in outside the edge of the slab
- 2 ft high parapet at roof (continuation of exterior façade panels above top of roof deck)
- 6 ft high screen walls around mechanical area at roof level
- Total height of façade panels supported at the roof level = 9.25 ft
- Total height of façade panels supported at each floor level = 13.5 ft (7.25 ft supported @ foundation)
- Openings for stairs & specifics of stair framing neglected; load calculations are based on an idealized solid diaphragm with no openings at all the levels

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Façade

- Architectural Precast Concrete Panels
- Spandrel supported, Dead weight: 30 psf on wall area

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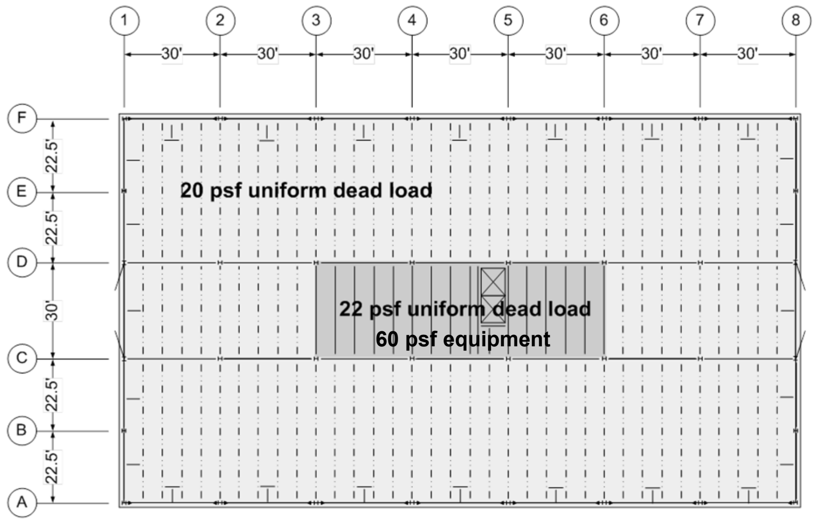
Roof Dead Load



Column allowance: 2 psf on roof, except 2.5 psf in roof mechanical area

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Floor Dead Load

- Slab and metal deck 60 psf
- Infill beams, girders & spandrels allowance 8 psf
- Misc. (Ceiling, MEP, …) 12 psf
- Total 80 psf

- Column allowance 2.5 psf

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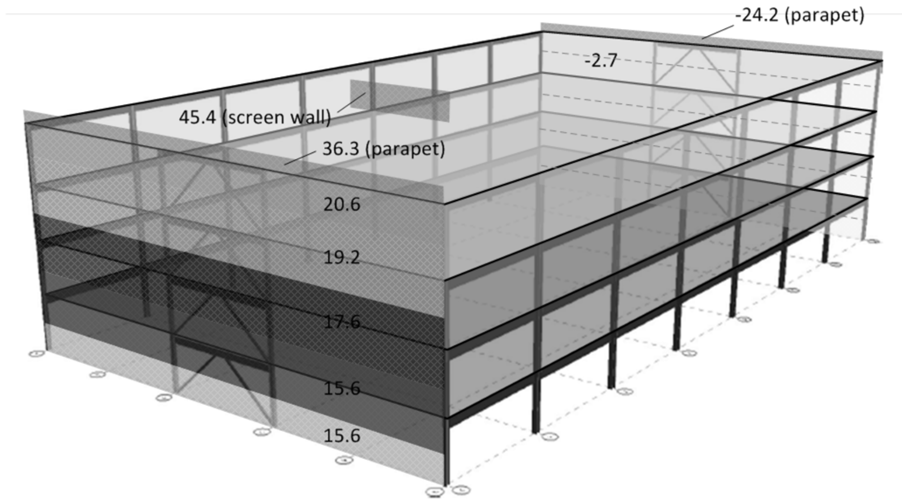
E Wind Case 1a Pressures (psf)

(with inward internal pressure, sidewall pressures not shown)



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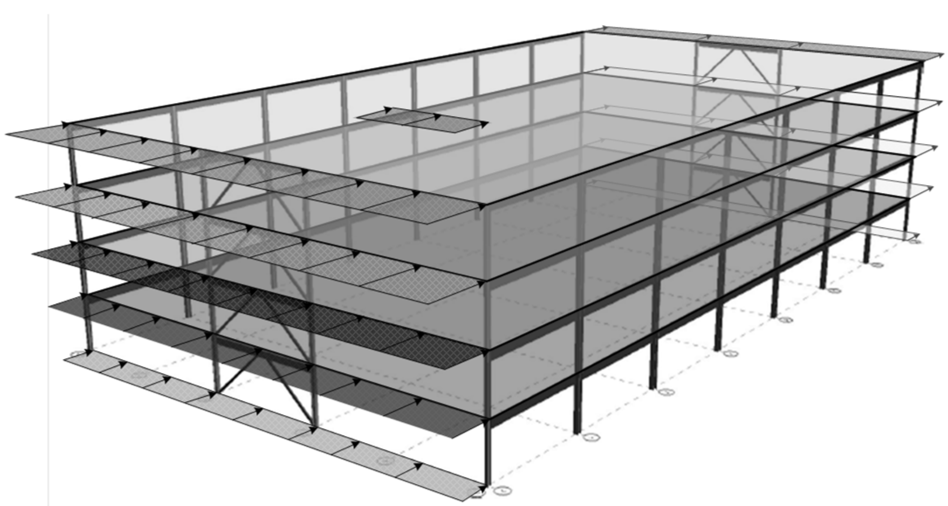
E Wind Case 1a

(Resultant line loads applied to the roof deck, the floor slabs & the foundation)



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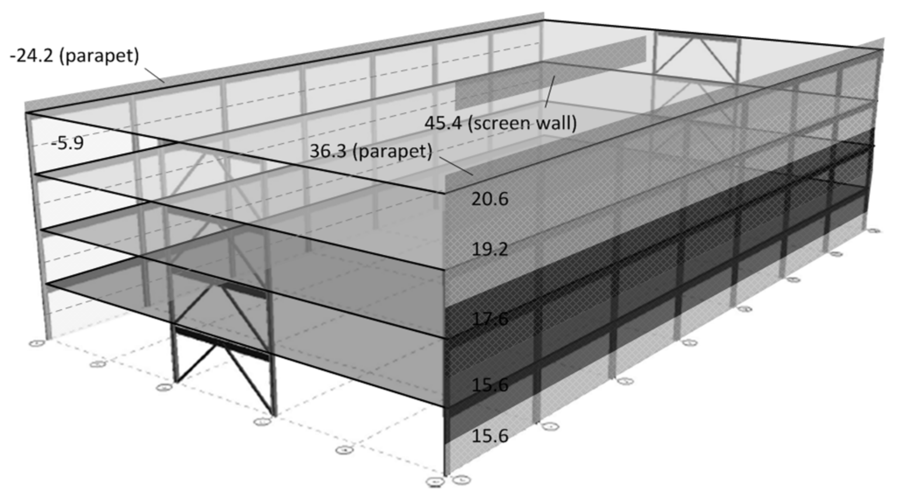
N Wind Case 1b Pressures (psf)

(with inward internal pressure, sidewall pressures not shown)



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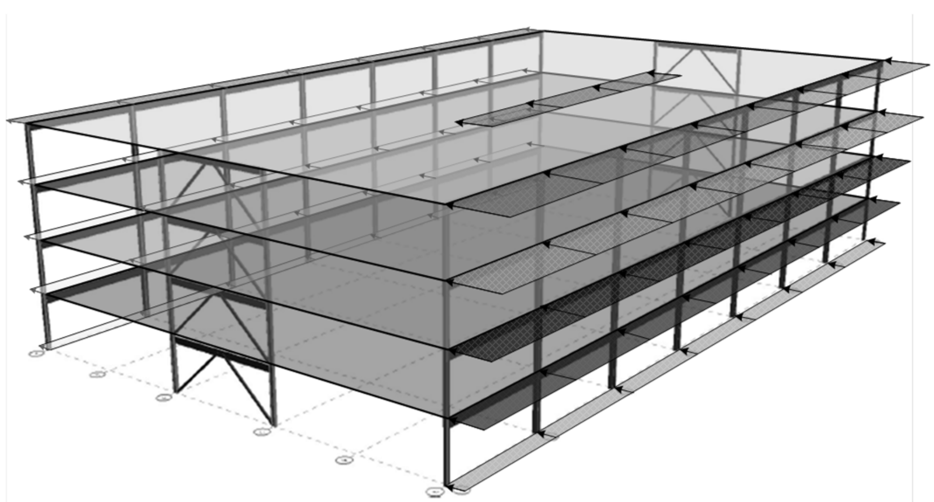
N Wind Case 1b

(Resultant line loads applied to the roof deck, the floor slabs & the foundation)



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Key Seismic Attributes

- Effective Seismic Weight (ASCE 7 Sect. 12.7.2)
  = Dead + 0.25 Live in storage areas +
  + Operating wt. of permanent equipment (60 psf)
  + 10 psf partition allowance where Live $\leq$ 80 psf
- W_{roof} = 959 kip
- W_{floors} = 2574 kip (neglecting all floor openings)
- $W = W_{roof} + 3W_{floors}$ = 8680 kip
- Building Periods & Seismic Coefficients:
  - N–S: $T = 0.682$ s ($< T_S$) → $C_s = 0.050$
  - E–W: $T = T_a = 1.175$ s → $C_s = 0.029$

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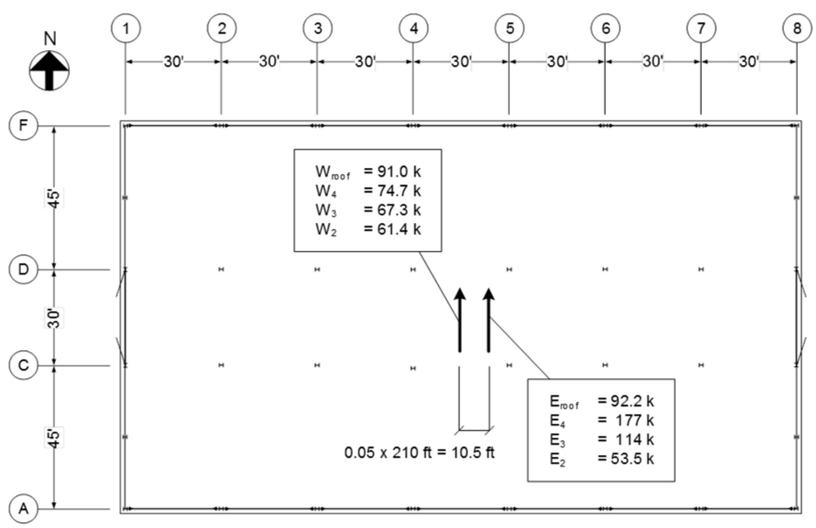
Wind & Seismic N–S Total Loads at Each Level



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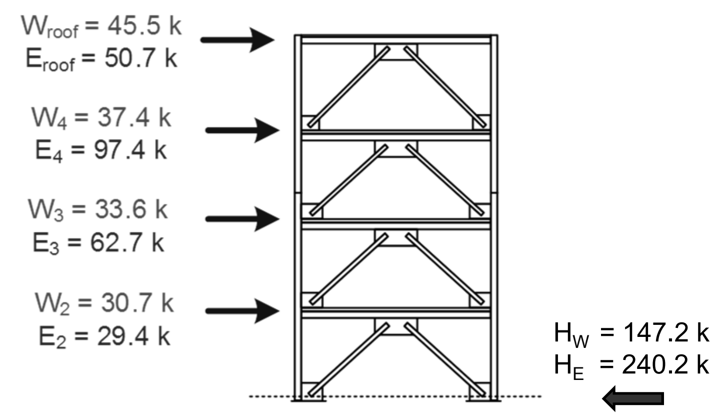
Wind & Seismic N-S Loads on Individual Braced Frames



Vertical seismic load effect = $0.2S_{DS}$ = 0.03, included with dead load in LRFD earthquake load combinations

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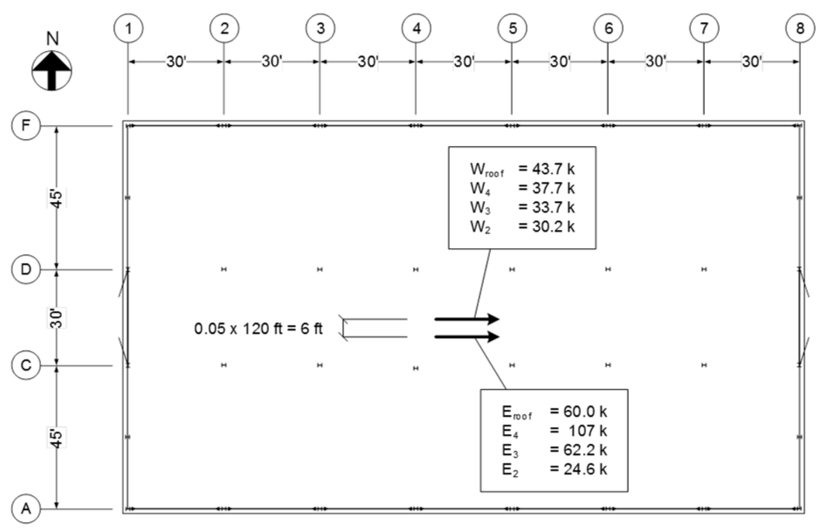
Wind & Seismic E-W Total Loads at Each Level



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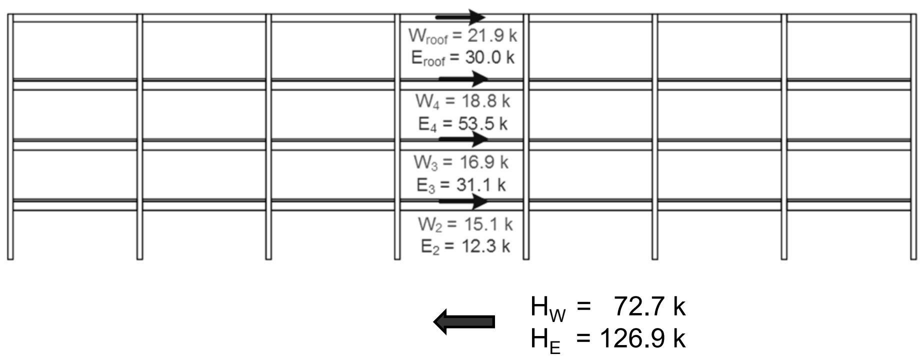
Wind & Seismic E-W Loads on Individual Moment Frames



Vertical seismic load effect = $0.2S_{DS}$ = 0.03, included with dead load in LRFD earthquake load combinations (ASCE 7 Sect. 12.4.2)

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Target Loading & Member Design

- Assume that the ASCE 7 Load Combination 5, 1.23D+1.0E+0.5L+0.2S governs
  (This earthquake load combination dominates for a large number of the frame members)
- Focus on bottom story components
- Design braced frames using the ELM
  - The DM doesn't provide any advantage for braced frame design, although it can be argued that it gives a better estimate of the true internal forces at the ultimate strength condition
- Design moment frames using the DM
- Note: in many cases, the member selections in braced frames are largely governed by strength, whereas the member selections in sway moment frames are largely governed by lateral stiffness

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TOPIC 6.2

DESIGN OF CDS BUILDING BRACED FRAMES

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Outline: Braced Frame Calcs.

- Vertical load takedowns
- Upper-bound estimation of B_2

$$B_2 = \frac{1}{1 - \frac{P_{story}}{P_{estory}}} \qquad P_{estory} = R_M \frac{HL}{\Delta_H} \qquad R_M = 1$$

Eqs. (A-8-6) (A-8-7) (A-8-8)

- Live Load reduction considerations
- Design bottom story column & bracing diagonal for

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

Eq. (A-8-2)

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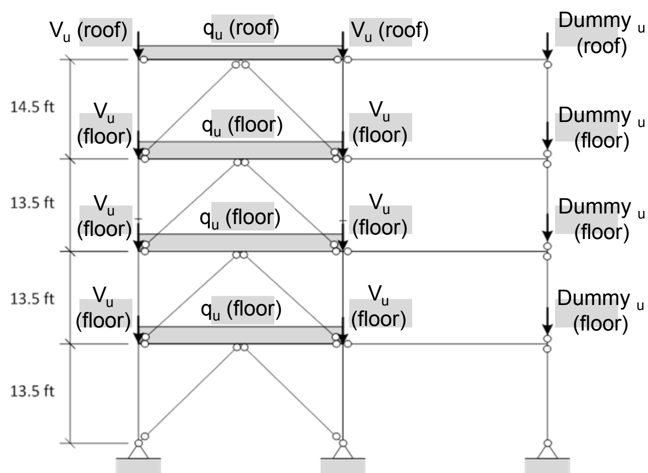
Vertical Load Calcs., Braced Frames

Load Combination 5, 1.23D + 1.0E + 0.5L + 0.2S



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N-S Spandrel Distributed Loads

Roof: q_D = 22 psf x 3.5 ft — DL allowance
+ 30 psf x 9.25 ft — façade
= 355 plf
q_S = 8.5 psf x 3.5 ft = 30 plf — snow + rain on snow

$\mathbf{q_u}$ = 1.23 x 355 plf + 0.2 x 30 plf = **443 plf**

Floors: q_D = 82.5 psf x 5.5 ft — DL allowance
+ 30 psf x 13.5 ft — façade
= 859 plf
q_{L1} = 100 psf x 5.5 ft = 550 plf — spandrels @ stairs
q_{L2} = 80 psf x 5.5 ft = 440 plf — spandrels @ offices

$\mathbf{q_u = q_{u1}}$ = 1.23 x 859 plf + 0.5 x 550 plf = **1.331 klf** @ stairs
$\mathbf{q_{u2}}$ = 1.23 x 859 plf + 0.5 x 440 plf = **1.277 klf** @ offices

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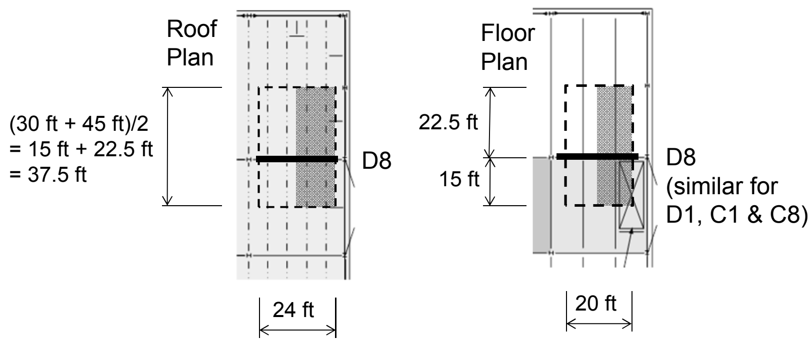
Col. C8 & D8 loads from E-W Girders

Roof: V_{u1} = [1.23 x 22 psf + 0.2 x 8.5 psf] x (24 ft/2) x 37.5 ft = **12.9 kip**

Floors: V_{u1} = { [1.23 x 82.5 psf x 37.5 ft] + 0.5 x 80 psf x 22.5 ft + 0.5 x 100 psf x 15 ft } x 20 ft/2 = **54.6 kip**



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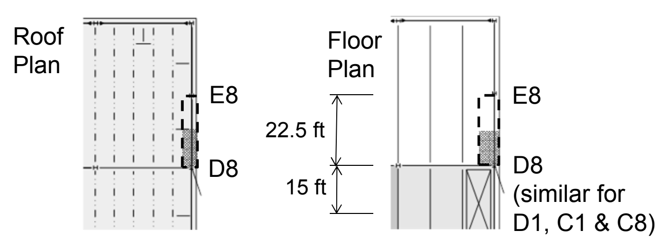
Col. D8 Loads from Spandrel D8-E8

Roof: V_{u2} = 443 plf x 22.5 ft/2 = **5.0 kip**

Floors: V_{u2} = 1.277 klf x 22.5 ft/2 = **14.4 kip**



Summary, Col. D8 Concentrated Loads

Roof: $V_u = V_{u1} + V_{u2}$ = 12.9 k + 5.0 k = **17.9 kip**

Floors: $V_u = V_{u1} + V_{u2}$ = 54.6 k + 14.4 k = **69.0 kip**

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Total Dead Load Stabilized by Lateral Load Resisting Systems

Roof: 22 psf x 211 ft x 121 ft	basic DL allowance
+ 2.5 psf x 90 ft x 30 ft	addtl. DL in mech. area
+ 30 psf x (90 ft x 2 + 30 ft x 2) x 6 ft	screen walls, mech. area
+ 60 psf x 90 ft x 30 ft	roof mech. equipment
+ 30 psf x (211.8 ft x 2 + 121.8 ft x 2) x 9.25 ft	ext. walls
= **959 kip**	

Floors: 82.5 psf x 211 ft x 121 ft	basic DL allowance
+ 30 psf x (211.8 ft x 2 + 121.8 ft x 2) x 13.5 ft	ext. walls
= **2376 kip (for each floor)**	

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Total Live Load Stabilized by Lateral Load Resisting Systems

Snow Load (uniform snow + rain on snow)
= 8.5 psf x 211 ft x 121 ft = **217 kip (roof)**

Floor Live Load = 80 psf x 2 x 45.5 ft x 211 ft
+ 100 psf x 2 x 30.5 ft x 30 ft
+ 125 psf x 150 ft x 30 ft = **2281 kip (for each floor)**

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Total Factored Vertical Load Stabilized by Lateral Systems

Loads applied at each level:

Y_i = 1.23 x 959 k + 0.2 x 217 k = **1223 kip at the roof**

Y_i = 1.23 x 2376 k + 0.5 x 2281 k = **4063 kip at each floor**

Total vertical load supported by bottom story:

P_{story} = ½ (1223 k + 3 x 4063 k) = **6706 kip (per braced frame)**

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Factored Dummy Column Loads

At Roof:

Dummy $_u$ = 1223 k /2 – 0.443 klf x 30 ft – 2 x 17.9 k
= **562 kip (per braced frame)**

At Each Floor:

Dummy $_u$ = 4063 k /2 – 1.331 klf x 30 ft – 2 x 69.0 k
= **1854 kip (per braced frame)**

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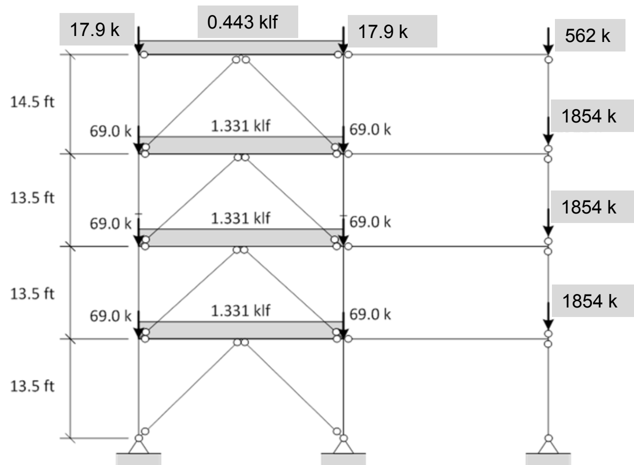
Factored Vertical Load Summary

Load Combination 5, 1.23D + 1.0E + 0.5L + 0.2S



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Upper-Bound B_2 Estimate (Bot. Story)

- Allowable 1st-order service wind drift
  $\Delta_{sW.max}$ = L/500 under 0.44W (10-Year MRI)

$$\therefore P_{Lstory.min} = \frac{H_{sW}}{\left(\Delta_{H_{sW}.max}/L\right)} = \frac{0.44 \times 147.2k}{(1/500)} = 32,400\,\frac{kip}{rad}$$

- Allowable seismic drift $\Delta_a = 0.025h_{sx} = L/40$ under H_E (ASCE 7 Sect. 12.8.6 & 12.12.1)

Given $C_d = 3$ → $\frac{\Delta_{H_E.max}}{L} = \frac{1}{120}$ elastic drift ratio under H_E

$$\therefore P_{Lstory.min} = \frac{H_E}{\left(\Delta_{H_E.max}/L\right)} = \frac{240.2k}{(1/120)} = 28,800\,\frac{kip}{rad}$$

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Upper-Bound B_2 Estimate (cont'd)

- Seismic P-Δ limit (ASCE 7 Sect. 12.8.7)

$$\left[\theta = \frac{P_x\left(\frac{\Delta}{C_d}\right)I_e}{V_x h_{sx}} = \frac{P_x}{\left\{\frac{H_E}{\left(\Delta_{H_E}/L\right)}\right\}} = \frac{P_x}{P_{Lstory.min}}\right] \le \left[\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25\right]$$

- $C_d = 3$, recommended $\beta = 1$ for braced frames, which typically will be governed by strength rather than stiffness → $\theta_{max} = 0.167$

  → $P_{Lstory.min} = 6P_x$

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P_x = Total Vertical Design Load

ASCE 7 Sect. 12.8.7: "when computing P_x, no individual load factor need exceed 1.0"

Common practice: use 1.0 Dead + 0.5 Live + zero Snow (for Snow $\le$ 30 psf)

At bottom story:

P_x = ½ { 959 k (roof) + 3 x 2376 k (floors) + 0.5 x [3 x 2281 k (floors)] }

= **5754 k per lateral load resisting frame**

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Upper-Bound B_2 Estimate (cont'd)

- Seismic P-Δ limit (ASCE 7 Sect. 12.8.7)

$$\left[\theta = \frac{P_x\left(\frac{\Delta}{C_d}\right)I_e}{V_x h_{sx}} = \frac{P_x}{\left\{\frac{H_E}{\left(\Delta_{H_E}/L\right)}\right\}} = \frac{P_x}{P_{Lstory.min}}\right] \le \left[\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25\right]$$

- C_d = 3, recommended β = 1 for braced frames, which typically will be governed by strength rather than stiffness → $\theta_{max} = 0.167$

→ $P_{Lstory.min} = 6P_x$ = 6 x 5754 kip = 34,520 kip/rad

⬆ **Governs**

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Upper-Bound B_2 Estimate (cont'd)

$R_M = 1.0$ since there are no moment frames in the N-S direction — **Eq. (A-8-8)**

$P_{estory.min} = R_M P_{Lstory.min}$ = 34,520 kip/rad — **Eq. (A-8-7)**

P_{story} = 6,706 kip (from Slide 43)

$$B_{2.max} = \frac{1}{1-\frac{P_{story}}{P_{estory}}} = 1.24$$ — **Eq. (A-8-6)**

But Wait! What about Live Load Reduction? Is it really necessary to assume that the building is packed to the gills in determining the total nominal live load?

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Live Load Reduction (Slide 1)

(Credits: Ziemian & McGuire (1992))

1) Determine all the factored *reducible* live loads that are applied to the braced frames and their dummy columns
   - ➢ The live loads between col. lines C & D are not reducible, and thus are not included in these calculations

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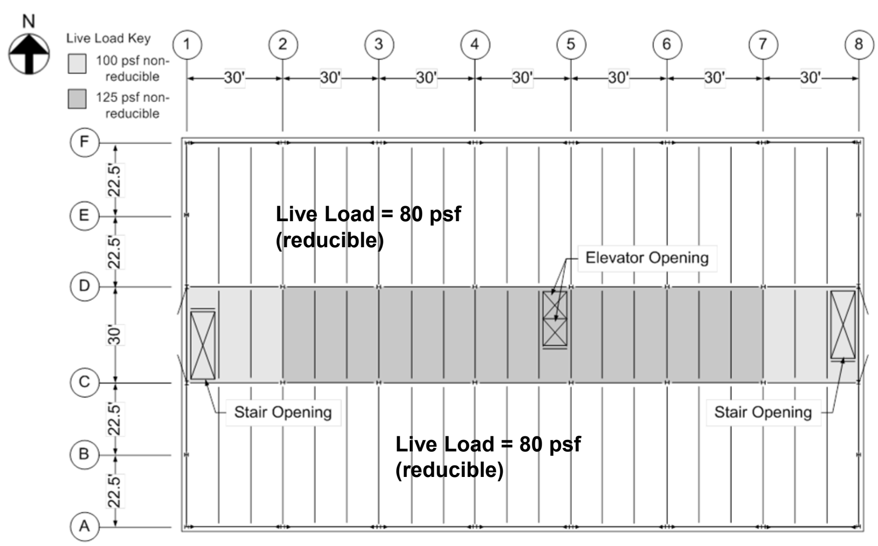
3rd & 4th Floor Framing Plan & Live Load



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Live Load Reduction (Slide 1)

(Credits: Ziemian & McGuire (1992))

1) Determine all the factored *reducible* live loads that are applied to the braced frames and their dummy columns
   - The live loads between col. lines C & D are not reducible, and thus are not included in these calculations
   - **$0.5V_{L1}$** = 0.5 x 80 psf x (20 ft / 2) x (45 ft /2) = **9.0 kip** transferred from the E-W girders on col. lines C & D
   - **$0.5V_{L2}$** = 0.5 x 80 psf x 5.5 ft x 22.5 ft / 2 = **2.5 kip** transferred from spandrels "outside" of col. lines C & D
   - **$0.5V_L$** = 2.5 k + 9.0 k = **11.5 kip**
   - **$0.5V_{L.dummy.col}$** = { 0.5 x 80 psf x 2 x 45.5 ft x 211 ft }
     x ½ (since there are two N-S braced frames)
     – 2 braced frame cols. x $0.5V_L$
     = 384 k – 2 x 11.5 k
     = **384 kip – 23 kip (at each floor)**

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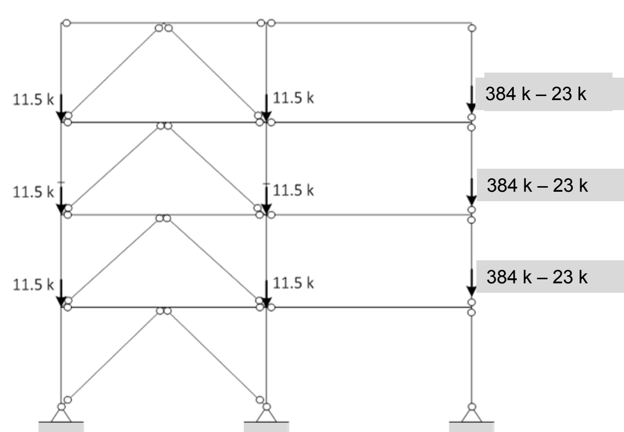
Live Load Reduction (Slide 2)

1) Determine all the factored *reducible* live loads that are applied to the braced frame and its dummy column



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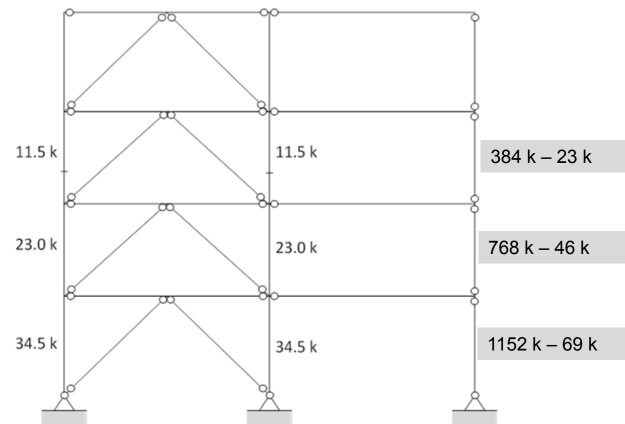
Live Load Reduction (Slide 3)

2) Determine the member live load internal forces (P_L) corresponding to the above applied loads



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Live Load Reduction (Slide 4)

3) Calculate the influence areas A_I (ft^2) for each of the column members, as well as for the dummy columns

- The areas that do not contain reducible live loads are not included in the influence area
- 3rd–story cols.: A_I = 30 ft x 45 ft + 0.5 ft x 22.5 ft = 1361 ft^2
- 2nd–story cols.: A_I = 2 x 1361 ft^2 = 2272 ft^2
- 1st–story cols.: A_I = 3 x 1361 ft^2 = 4083 ft^2
- Full 3rd–story: A_I = 2 x 211 ft x 45.5 ft = 19,201 ft^2
- Full 2nd–story: A_I = 2 x 19,201 ft^2 = 38,402 ft^2
- Full 1st–story: A_I = 3 x 19,201 ft^2 = 57,603 ft^2

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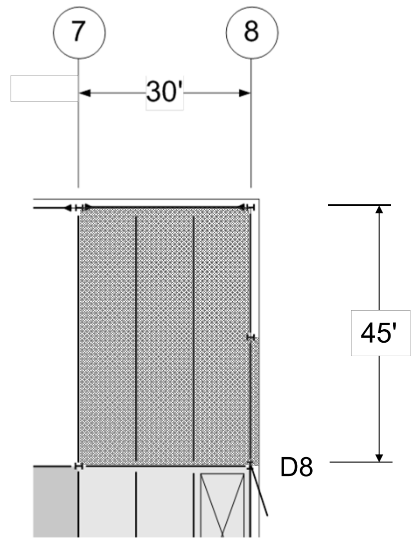
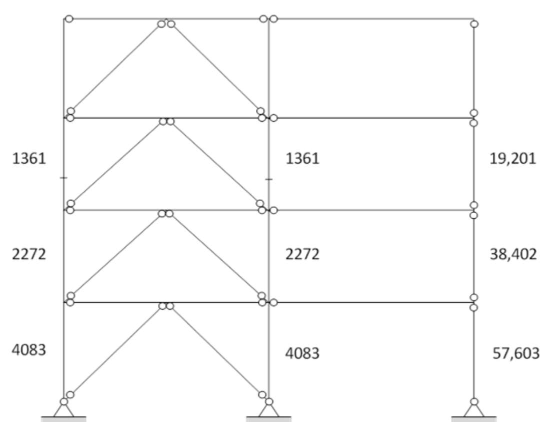
Influence Area for Column D8



57

Live Load Reduction (Slide 5)

3) Calculate the influence areas A_I (ft^2) for each of the column members, as well as for the dummy columns



58

Live Load Reduction (Slide 6)

4) Calculate the live load reduction factor

$$R_L = 0.25 + \frac{15}{\sqrt{A_I}} \geq 0.5$$ for members supporting one story

≥ 0.4 for members supporting multiple stories

- 3rd–story cols.: $R_L = 0.25 + 15 / (1361)^{0.5} = 0.66$
- 2nd–story cols.: $R_L = 0.25 + 15 / (2272)^{0.5} = 0.56$
- 1st–story cols.: $R_L = 0.25 + 15 / (4083)^{0.5} = 0.48$
- Full 3rd–story: $R_L = \max[0.5, 0.25 + 15 / (19{,}201)^{0.5}] = 0.5$
- Full 2nd–story: $R_L = \max[0.4, 0.25 + 15 / (38{,}402)^{0.5}] = 0.4$
- Full 1st–story: $R_L = 0.4$

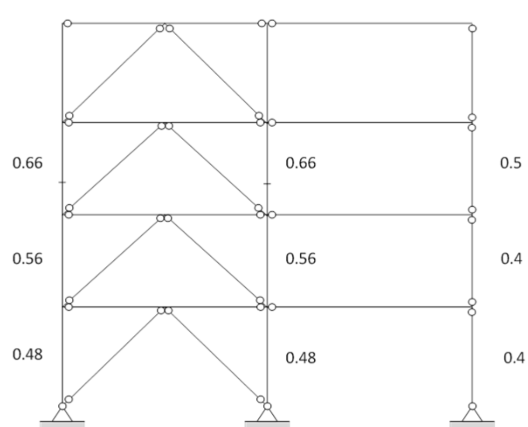
59

Live Load Reduction (Slide 7)

4) Calculate the live load reduction factor

$$R_L = 0.25 + \frac{15}{\sqrt{A_I}} \geq 0.5$$ for members supporting one story

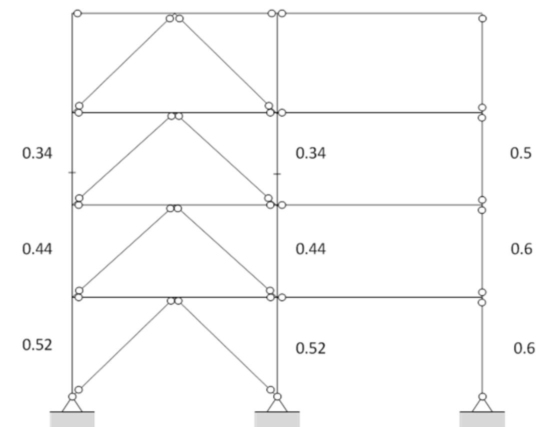
≥ 0.4 for members supporting multiple stories



60

Live Load Reduction (Slide 8)

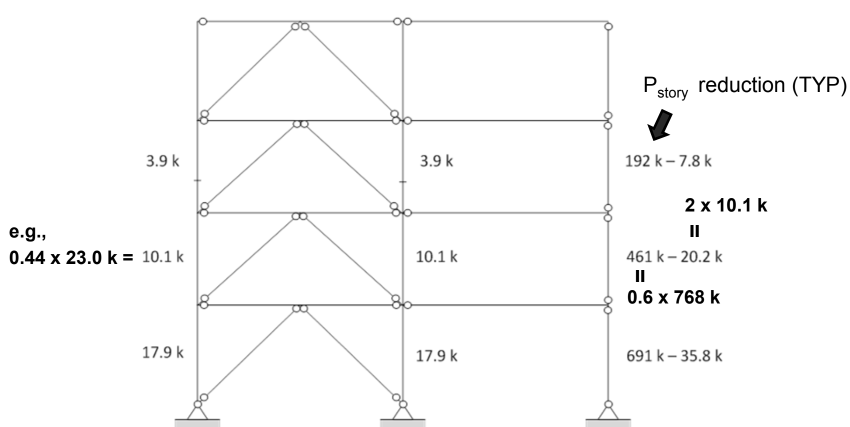
5) Calculate $(1-R_L)$ for each of the frame members



61

Live Load Reduction (Slide 9)

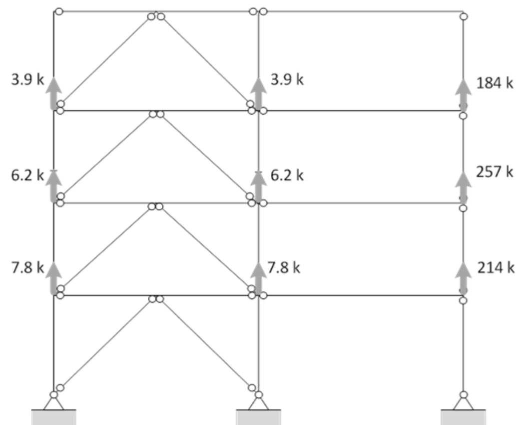
6) Calculate the internal member axial forces, $(1-R_L)\ P_L$, which may be deducted from the total factored loads



62

Live Load Reduction (Slide 10)

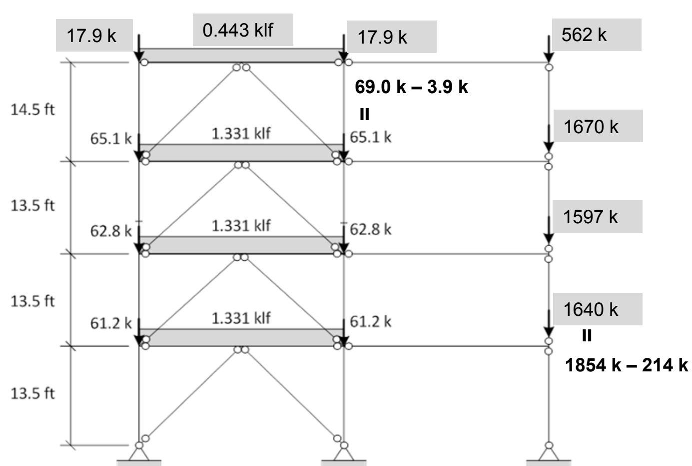
7) Calculate the upward vertical loads to be applied at the frame joints to produce the member internal axial force reductions determined in Step (6)



63

Live Load Reduction (Slide 11)

8) Add the above upward vertical loads to the load combination under consideration



64

Key Take Away (Live Load Reduction)

Reducing the live loads APPLIED to the structure, using the above approach, is fundamentally more correct than the customary live load reduction procedure. The customary live load reduction procedure is as follows:

- Apply the full unreduced live load to the structure
- Perform the global structural analysis based on this unreduced live load
- Determine the unreduced internal axial force in the columns caused by the full applied live loads
- Apply the appropriate live load reduction factors to the internal column axial force on a member by member basis

The recommended procedure gives reduced APPLIED live load forces in the global analysis. As such, the global second-order effects are smaller. The customary approach analyzes the structure for the full unreduced live load, thus producing unjustified larger second-order effects.

65

Upper-Bound B_2 Estimate (updated)

$$P_{estory.min} = R_M P_{Lstory.min} = 34{,}520 \text{ kip/rad} \qquad \text{Eq. (A-8-7)}$$

$$P_{story} = 6{,}706 \text{ k} - 691 \text{ k} = 6{,}015 \text{ kip}$$

$$B_{2.max} = \frac{1}{1 - \frac{P_{story}}{P_{estory}}} = 1.21 \qquad \text{Eq. (A-8-6)}$$

66

Bottom Story Column Design

$P_u = P_{nt} + B_2 P_{lt}$ **Eq. (A-8-2)**

P_u = 17.9 k (roof) + 65.1 k (3rd story) + 62.8 k (2nd story)
+ 61.2 k (1st story) + 0.443 klf x 15 ft (roof)
+ 2 x 1.331 klf x 15 ft (3rd & 2nd stories)
+ 1.331 klf x 0.375 x 15 ft (1st story) ← P_{nt}
+ 1.21 (B_2) x (50.7 k x 41.5 ft + 97.4 k x 27 ft + 62.7 k x 13.5 ft) / 30 ft ← P_{lt}
= 261.1 k + 1.21 x 186.0 k = **486 kip**

$KL_x = KL_y = KL_z$ = 13.5 ft

From **Table 4-1 of the AISC Manual**, a W10x54, W12x53 or W14x61 will work

Select a W12x53, $\phi_c P_n$ = 514 kip

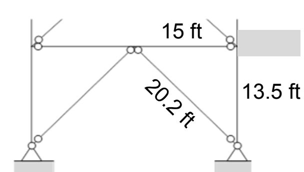
67

Bottom Story Diagonal Brace Design

$P_u = P_{nt} + B_2 P_{lt}$ **Eq. (A-8-2)**

P_u = 2 lengths/2 braces x 1.331 klf x 15 ft x 0.625 x (20.2 ft / 13.5 ft) ← P_{nt}
+ 1.21 (B_2) x ½ x 240.2 k x (20.2 ft / 15 ft) ← P_{lt}
= 18.6 k + 1.21 x 161.7 k = **214 kip**

$KL_x = KL_y = KL_z$ = 20.2 ft



From **Table 4-4 of the AISC Manual**, an HSS8x8x5/16 (31.8 plf), HSS9x9x1/4 (29.2 plf), or HSS10x10x1/4 (32.6 plf) will work

Select an HSS9x9x1/4, $\phi_c P_n$ = 243 kip

68

There's always a solution in Steel

Stability Design of Low- and Medium-Rise Steel Buildings — Don White

TOPIC 6.3

DESIGN OF CDS BUILDING MOMENT FRAMES

69

Outline: Moment Frame Calcs.

- Vertical load takedowns
- Back-calculation of 1st-order service wind drift limit necessary to satisfy an actual 2nd-order limit
- Upper-bound estimation of B_2

$$B_2 = \frac{1}{1 - \frac{P_{story}}{P_{estory}}} \qquad P_{estory} = R_M \frac{HL}{\Delta_H} \qquad R_M = 1 - \frac{P_{mf}}{P_{story}}$$

Eqs. (A-8-6) (A-8-7) (A-8-8)

- Characteristics of upper-bound B_2
- Bottom story column & spandrel design

$$M_u = 1.0 M_{nt} + B_2 M_{lt}$$

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

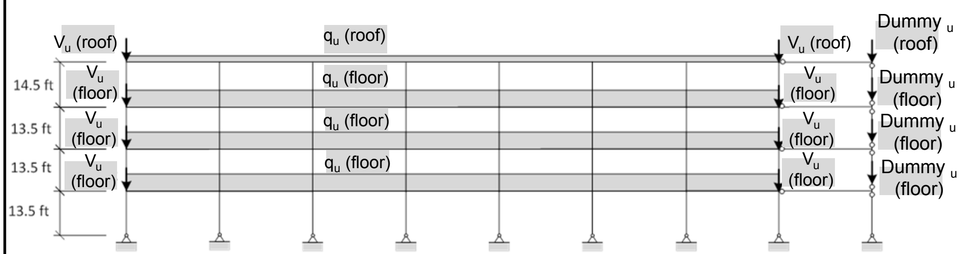
Eqs. (A-8-1) (A-8-2)

- Story stiffness design

70

Vertical Load Calcs., Moment Frames

Load Combination 5, 1.23D + 1.0E + 0.5L + 0.2S



71

E-W Spandrel Distributed Loads

Roof: q_D = 22 psf x (45 ft/2 + 0.5 ft) — DL allowance
+ 30 psf x 9.25 ft — façade
= 784 plf

q_S = 8.5 psf x (45 ft/2 + 0.5 ft) = 196 plf — snow + rain

$\mathbf{q_u}$ = 1.23 x 784 plf + 0.2 x 196 plf = **1.00 klf**

Floors: q_D = 82.5 psf x (45 ft/2 + 0.5 ft) — DL allowance
+ 30 psf x 13.5 ft — façade
= 2.30 klf

q_L = 80 psf x (45 ft/2 + 0.5 ft) = 1.84 klf

$\mathbf{q_u}$ = 1.23 x 2.30 klf + 0.5 x 1.84 klf = **3.75 klf**

(Handling all girder transverse loads as distributed loads for preliminary design)

72

Col. A1 & A8 Concentrated Loads

From Spandrels A1–B1 & A8–B8 (same as on Slide 40)

Roof: V_{u1} = 443 plf x 22.5 ft /2 = **5.0 kip**

Floors: V_{u1} = 1.277 klf x 22.5 ft /2 = **14.4 kip**

From slab & cladding panels at the building corners

Roof: V_{u2} = [1.23 x 22 psf + 0.2 x 8.5 psf] x 0.5ft x 0.5ft
+ 1.23 x 30 psf x 9.25 ft x 2 x 0.94 ft = **0.7 kip**

Floors: V_{u2} = [1.23 x 82.5 psf + 0.5 x 80 psf] x 0.5ft x 0.5ft
+ 1.23 x 30 psf x 13.5 ft x 2 x 0.94 ft = **1.0 kip**

Roof: $V_u = V_{u1} + V_{u2}$ = **5.7 kip** Floors: $V_u = V_{u1} + V_{u2}$ = **15.4 kip**

73

Total Factored Vertical Load Stabilized by Lateral Systems

Same as calculated for braced frames:

Roof: Y_i = 1.23 x 959 k + 0.2 x 217 k = **1223 kip**

Floors: Y_i = 1.23 x 2376 k + 0.5 x 2281 k = **4063 kip**

At bottom story:

P_{story} = ½ [1223 kip + 3 x 4063 kip] = **6706 kip (per frame)**

P_{mf} = 1.0 klf x 210 ft + 2 x 5.7 k
+ 3 x [3.75 klf x 210 ft + 2 x 15.4 k] = **2676 kip**

$$R_M = 1 - 0.15\frac{P_{mf}}{P_{story}} = 0.94$$ **Eq. (A-8-8)**

74

Factored Dummy Column Loads

At Roof:

$$\text{Dummy}_u = 1223 \text{ k}/2 - 1.0 \text{ klf} \times 210 \text{ ft} - 2 \times 5.7 \text{ k}$$
$$= 390 \text{ kip}$$

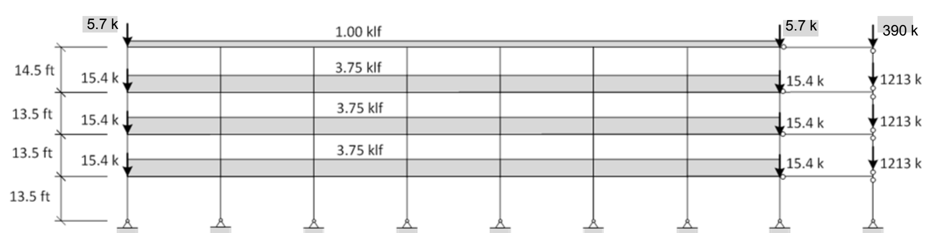
At Floors:

$$\text{Dummy}_u = 4063 \text{ k}/2 - 3.75 \text{ klf} \times 210 \text{ ft} - 2 \times 15.4 \text{ k}$$
$$= 1213 \text{ kip}$$

75

Factored Vertical Load Summary

Load Combination 5, 1.23D + 1.0E + 0.5L + 0.2S



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Total Service Vertical Load Stabilized by Lateral Systems

Used with service wind combination of 1.0D + 0.5L + 0.5S + 0.44W:

Roof: Y_i = 1.0 x 959 k + 0.5 x 217 k = **1068 kip**

Floors: Y_i = 1.0 x 2376 k + 0.5 x 2281 k = **3516 kip**

neglecting small vertical component of wind at the roof

At bottom story:

$P_{s.story}$ = ½ { 1068 k (roof) + 3 x 3516 k (floors) = **5808 kip**

per lateral load resisting frame

R_M for service wind is approximately the same as R_M for LC5:

R_M = **0.94**

77

Service Wind Drift Limit

(Credits: LeMessurier (1977))

- Back-calculate a limit on the *1st-order service wind drift* (ψ_1) given the a specified limit on the corresponding physical *2nd-order drift* ψ_2

$$\frac{(\Delta_{2nd})_s}{L} \le \psi_2 \Rightarrow B_{2s}\frac{(\Delta_{1st})_s}{L} \le \psi_2 \Rightarrow \frac{1}{1-\frac{P_{s.story}}{P_{Lstory}R_M}}\frac{H_s}{P_{Lstory}} \le \psi_2$$

$$\Rightarrow \frac{1}{\frac{P_{Lstory}}{H_s}-\frac{P_{s.story}}{R_M H_s}} \le \psi_2 \Rightarrow \frac{1}{\frac{1}{\psi_1}-\frac{P_{s.story}}{R_M H_s}} \le \psi_2 \Rightarrow \boxed{\psi_1 \le \frac{1}{\frac{1}{\psi_2}+\frac{P_{s.story}}{R_M H_s}}}$$

$$\psi_1 \le \frac{1}{400+\frac{5808\,\text{k}}{0.94 \times 0.44 \times 72.7\,\text{k}}} = \frac{1}{593}$$

78

Upper-Bound B_2 Estimate (Bot. Story)

- Allowable 1st-order service wind drift $\Delta_{sW.max}$ = L/593 under 0.44W

$$\therefore P_{Lstory.min} = \frac{H_{sW}}{\left(\Delta_{H_{sW}.max}/L\right)} = \frac{0.44 \times 72.7\,k}{(1/593)} = 18{,}970\,\frac{kip}{rad}$$

- Allowable seismic drift $\Delta_a = 0.025h_{sx}$ = L/40 under H_E (ASCE 7 Sect. 12.8.6 & 12.12.1)

Given C_d = 3 → $\frac{\Delta_{H_E.max}}{L} = \frac{1}{120}$ elastic drift ratio under H_E

$$\therefore P_{Lstory.min} = \frac{H_E}{\left(\Delta_{H_E.max}/L\right)} = \frac{126.9\,k}{(1/120)} = 15{,}230\,\frac{kip}{rad}$$

79

Upper-Bound B_2 Estimate (cont'd)

- Seismic P-Δ limit (ASCE 7 Sect. 12.8.7)

$$\left[\theta = \frac{P_x\left(\frac{\Delta}{C_d}\right)I_e}{V_x h_{sx}} = \frac{P_x}{\left\{\frac{H_E}{\left(\Delta_{H_E}/L\right)}\right\}} = \frac{P_x}{P_{Lstory.min}}\right] \le \left[\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25\right]$$

- C_d = 3, assuming β ≤ 0.67 for moment frames → θ_{max} = 0.25

→ $P_{Lstory.min} = 4P_x$

80

P_x = Total Vertical Design Load

(Same as for braced frames)

ASCE 7 Sect. 12.8.7: "when computing P_x, no individual load factor need exceed 1.0"

Common practice: use 1.0 Dead + 0.5 Live + zero Snow (for Snow $\leq$ 30 psf)

At bottom story:

P_x = ½ { 959 k (roof) + 3 x 2376 k (floors) + 0.5 x [3 x 2281 k (floors)] }

= **5754 k per lateral load resisting frame**

81

Upper-Bound B_2 Estimate (cont'd)

- Seismic P–Δ limit (ASCE 7 Sect. 12.8.7)

$$\left[\theta = \frac{P_x\left(\frac{\Delta}{C_d}\right)I_e}{V_x h_{sx}} = \frac{P_x}{\left\{\frac{H_E}{\left(\Delta_{H_E}/L\right)}\right\}} = \frac{P_x}{P_{Lstory.min}}\right] \leq \left[\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25\right]$$

- C_d = 3, assuming $\beta \leq 0.67$ for moment frames → θ_{max} = 0.25

→ $P_{Lstory.min} = 4P_x$ = 4 x 5754 kip = 23,020 kip/rad

⬆ **Governs**

82

Upper-Bound B_2 Estimate (cont'd)

$R_M = 0.94$ $\quad P_{Lstory.min} = 23{,}020$ kip/rad **Eq. (A-8-8)**

$P_{estory.min} = 0.8\, R_M\, P_{Lstory.min} = 17{,}310$ kip/rad **Eq. (A-8-7)**

(using the 0.8 elastic stiffness reduction of the DM, assuming $\tau_b = 1$)

$P_{story} = 6{,}706$ kip per frame (from Slide 74)

$$B_{2.max} = \frac{1}{1 - \frac{P_{story}}{P_{estory}}} = 1.63$$ ... less than 1.7, ∴ the frame is not stability critical **Eq. (A-8-6)**

83

Generalization, Upper-Bound B_2 Est.

- Determine required minimum story lateral stiffnesses ($P_{Lstory.min}$) from:
  - $H_{sW} / \left(\Delta_{H_{sW}.max} / L\right)$
  - $H_E / \left(\Delta_{H_E.max} / L\right)$ (NA for many single story frames)
  - $6P_x$ for braced frames (taking $\beta = 1$)
  - $4P_x$ for moment frames (assuming $\beta \leq 0.67$)
- $P_{estory.min} = 0.8\, R_M\, P_{Lstory.min}$ **Eq. (A-8-7)**
- Determine P_{story} from applicable load combination
- $B_2 = \frac{1}{1 - \frac{P_{story}}{P_{estory}}}$ **Eq. (A-8-6)**

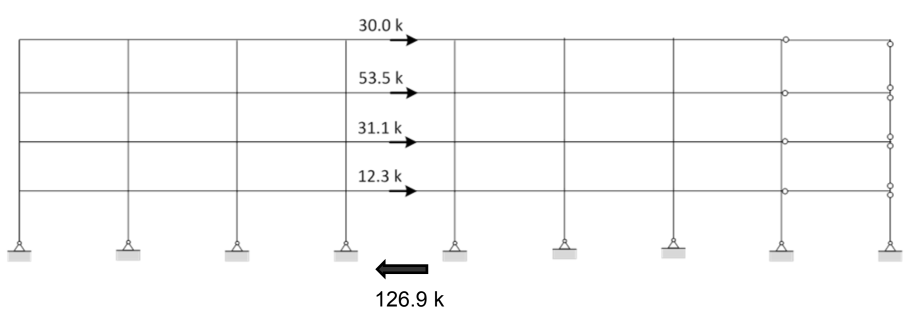
84

Characteristics, Upper-Bound B_2 (DM)

- $B_2 \leq 1.7$ under the wind & earthquake load combinations for practically all multi-story buildings
- It is not uncommon for B_2 to be greater than 1.7, & therefore for the structure to be stability critical, under the Dead + Maximum Live Load combinations in low- to medium-rise multi-story buildings
- B_2 can be significantly larger than 1.7 for LRFD LC3 (1.2D + 1.6S + 0.5W) in cases with large S
- Single-story buildings with interior walls, partitions, ceilings, & exterior wall systems designed to accommodate larger story drifts can have B_2 significantly greater than 1.7 if the gravity load is relatively large.

85

Earthquake Lateral Loads



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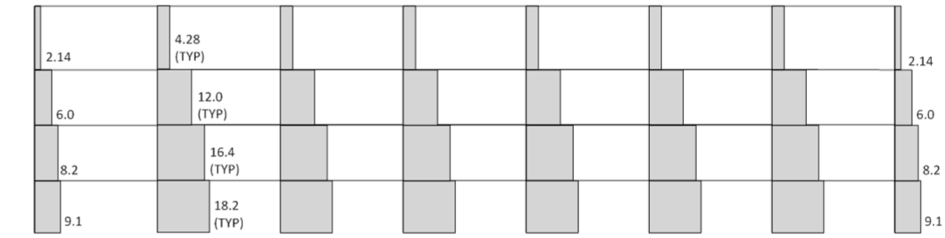
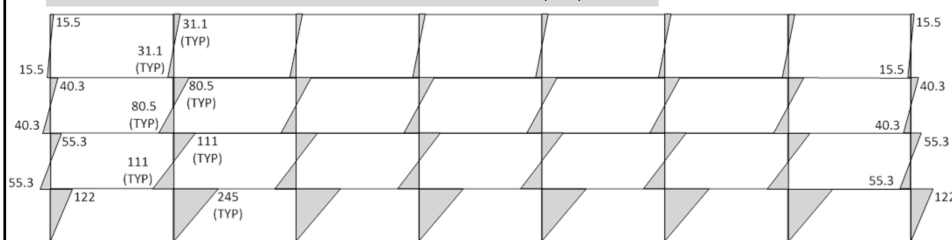
Prelim. Lateral Analysis (Portal Method)

1st –order column shear forces (k)



1st –order column moments, drawn on tension side (ft-k)



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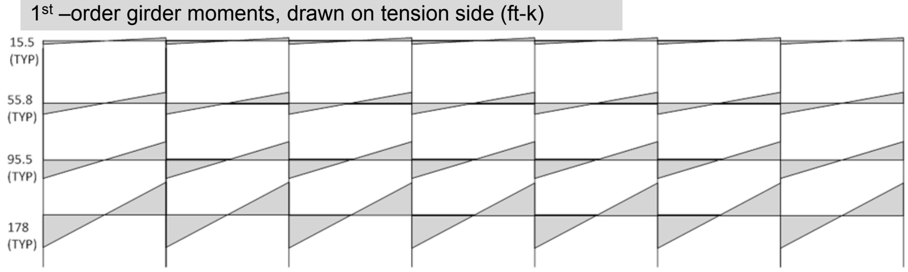
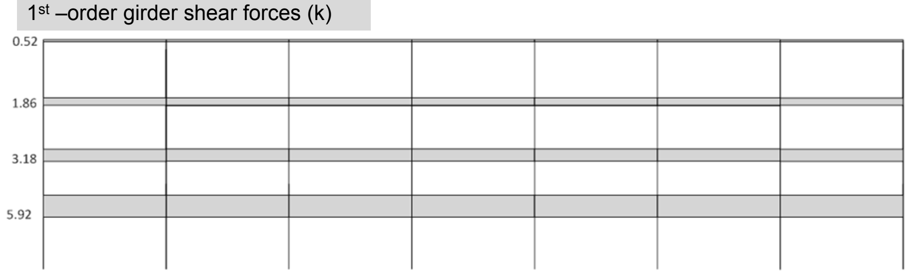
Prelim. Lateral Analysis (Portal Method)

1st –order girder moments, drawn on tension side (ft-k)



1st –order girder shear forces (k)



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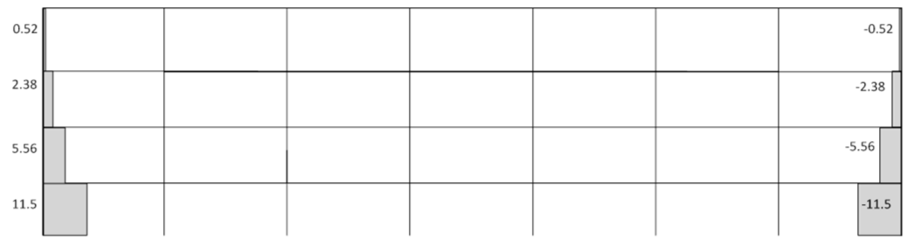
Prelim. Lateral Analysis (Portal Method)

1st –order column axial forces, positive in tension (k)



89

Bottom Story Exterior Column Design

$P_u = P_{nt} + B_2 P_{lt}$ — Eq. (A-8-2)

P_u = 5.7 k + 3 x 15.4 k + 1.0 klf x 15 ft + 3 x 3.75 klf x 15 ft (P_{nt})
\+ 1.63 (B_2) x 11.5 k (P_{lt})
= 235.6 k + 18.7 k = **254 kip**

$M_u = 1.0\,M_{nt} + B_2\,M_{lt}$ — Eq. (A-8-1)

$$= \frac{3.75\,\text{klf} \times (30\,\text{ft})^2}{12} \times \frac{1}{3} + 1.63 \times 122\,\text{ft-kip}$$

($1.0M_{nt}$; B_2; M_{lt})

Conservative estimate of girder end NT moment, assumed distributed one part to the bottom story column and two parts to the second story column (due to the simply-supported base conditions); 1st-order LT moment estimated using portal method

= 94 ft-kip + 199 ft-kip = **293 ft-kip**

90

Bottom Story Exterior Column Design

P_u = 254 kip, M_u = 293 ft-kip

$KL_x = KL_y = KL_z$ = 13.5 ft

Try a W14x68, p = 1.52, b_{x0} = 2.06, b_x (13.5 ft) = 2.26

(AISC Manual Table 6-1)

$$b_x = \max\left(\frac{b_x(13.5\,\text{ft})}{C_b}, b_{x0}\right) = \max\left(\frac{2.26}{1.75}, 2.06\right) = 2.06$$ **Eqs. (F2-1) (F2-2)**

C_b = 1.75 **Eq. (C-F1-1)**

$$pP_u = \frac{1.52 \times 254}{1000} = 0.388 > 0.2$$

➡ $pP_u + b_x M_u = 0.388 + 2.06\frac{293}{1000} = 0.388 + 0.604 = 0.992$ OK **Eq. (H1-1a)**

91

Bottom Story Spandrel Design

M_u = 1.0 M_{nt} + B_2 M_{lt} **Eq. (A-8-1)**

$$= \underbrace{\frac{3.75\,\text{klf} \times (30\,\text{ft})^2}{12}}_{1.0M_{nt}} + \underbrace{1.63}_{B_2} \times \underbrace{178\,\text{ft-kip}}_{M_{lt}}$$

Conservative estimate of the maximum NT moment + amplified LT moment at the leeward column face

= 281 ft-kip + 290 ft-kip = **571 ft-kip**

L_b = 10 ft

Neglect axial load in spandrel for preliminary design (the axial load is small and is resisted predominantly by the floor slab)

Neglect torsion from the exterior walls (assume torsion on the spandrels is resisted largely by the slab & by the infill beams)

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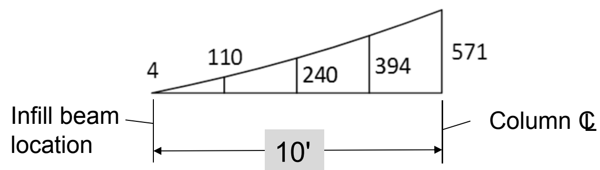
Bottom Story Spandrel Design

Second-order shear force at leeward end of beam, $V_u = V_{nt} + B_2 V_{lt}$:

V_u = 3.75 klf x 30 ft/ 2 + 1.63 x 178 ft-k x 2 / 30 ft = 75.5 k

Moments (ft-k) in critical leeward unbraced length:



$$C_b = \frac{12.5 \times 571}{2.5 \times 571 + 3 \times 110 + 4 \times 240 + 3 \times 394} = 1.83$$

Eq. (F1-1)

From the **AISC Manual Table 3-2**,
select a W24x62, $\phi_b M_n = \phi_b M_p$ = 574 ft-k, (L_r = 14.4 ft) > (L_b = 10 ft)
… flexural capacity = plateau strength by inspection

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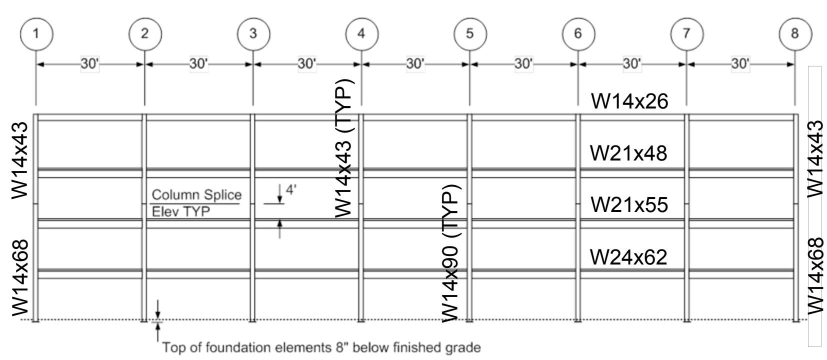
Strength Design of Moment Frames



94

Check Story Stiffness Requirements

Assume incidental rotational restraint at the simply-supported column bases based on the traditional assumption of G = 10, where:

$K_{rotation} = (6EI_c/L_c)/10 = 0.6 \times 29000$ ksi $\times$ 722 in^4 / 162 in
= 77,500 in-kip/rad for the W14x68 columns

$K_{rotation} = 0.6 \times 29000$ ksi $\times$ 999 in^4 / 162 in
= 107,300 in-kip/rad for the W14x90 columns

(Note: no 0.8 stiffness reduction here!)

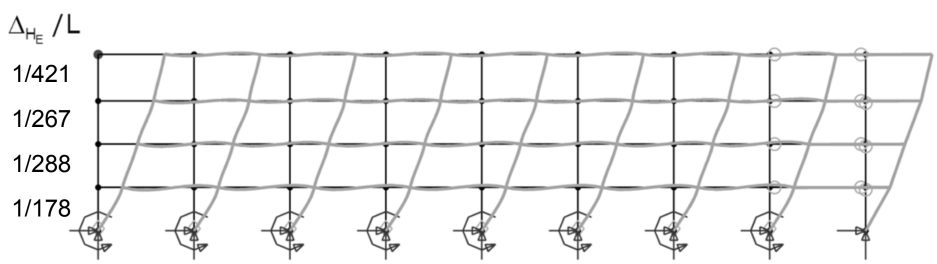
$$G = \frac{\Sigma(EI/L)_c}{\Sigma(EI/L)_g}$$

Eq. (C-A-7-3)

95

Mastan to the Rescue

1st-order elastic sidesway under 1.0E



Seismic Drift Limit of $\Delta_{H_E}/L \leq$ 1/120 is satisfied

Bottom story P_{Lstory} = 126.9 k / (1/178) = 22,590 kip/rad

slightly smaller than the minimum requirement
$P_{Lstory.min}$ = 23,020 kip/rad from the ASCE7 Seismic P-Δ limit

∴ The story stiffness needs to be increased

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Design for Story Stiffness

Useful rule of thumb:

- Increase the column sizes when $G << 1$
- Increase the girder sizes when $G >> 1$
- Increase both the columns and the girders, if necessary, when $G \cong 1$

For our 1st–story level:

$G = (2 \times 722\ in^4 / 13.5\ ft) / (1550\ in^4 / 30\ ft) = 2.07$ for W14x68 cols.

$G = (2 \times 999\ in^4 / 13.5\ ft) / (1550\ in^4 / 30\ ft) = 2.86$ for W14x90 cols.

∴ based on the **AISC Manual Table 3-3**, increase the spandrel sizes in the 1st–story level to a W24x68, $I_x = 1830\ in^4$

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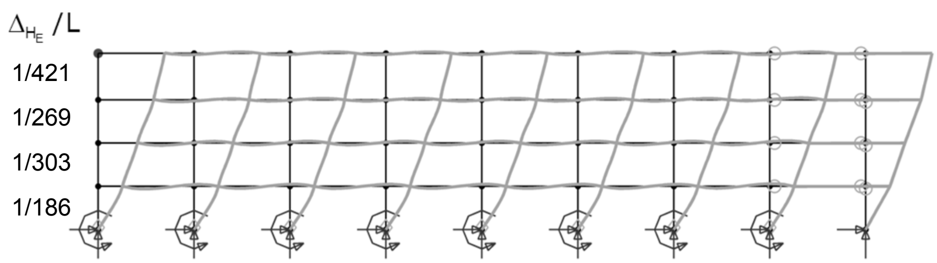
Mastan to the Rescue

1st-order elastic sidesway under 1.0E



Seismic Drift Limit of $\Delta_{H_E}/L \leq 1/120$ is satisfied

Bottom story P_{Lstory} = 126.9 k / (1/186) = 23,600 kip/rad

slightly larger than the minimum requirement
$P_{Lstory.min}$ = 23,020 kip/rad from the ASCE7 Seismic P-Δ limit

∴ **The seismic drift & seismic P-Δ limits are satisfied**

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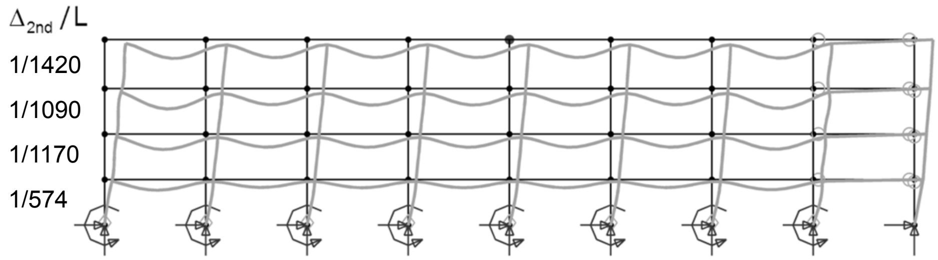
Check 2^{nd}-Order Service Wind Drift

Under 1.0D + 0.5L + 0.5S + 0.44W



The 2^{nd}-order service wind drift limit of 1/400 is satisfied

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Other Considerations to Limit Drift

- Composite Beam Action
- Finite Joint Size & Panel Zone Deformation
- Column Base Stiffness

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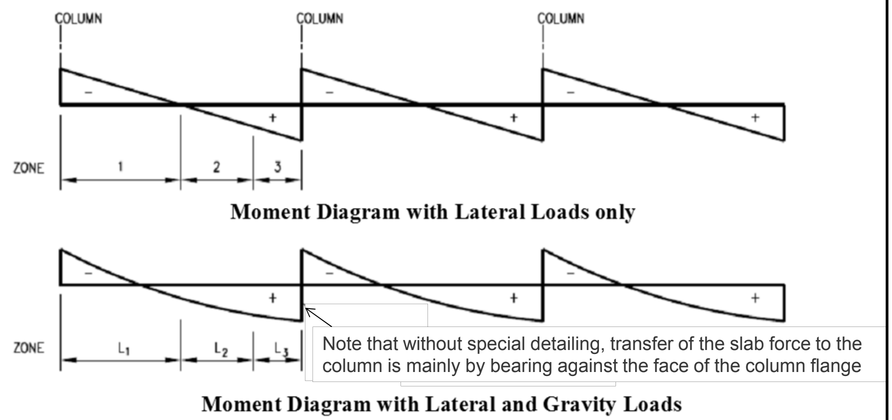
Composite Beam Stiffnesses

(Credits: Larry Griffis, W.P. Moore & Associates; Schaffhausen & Wegmuller,1977)

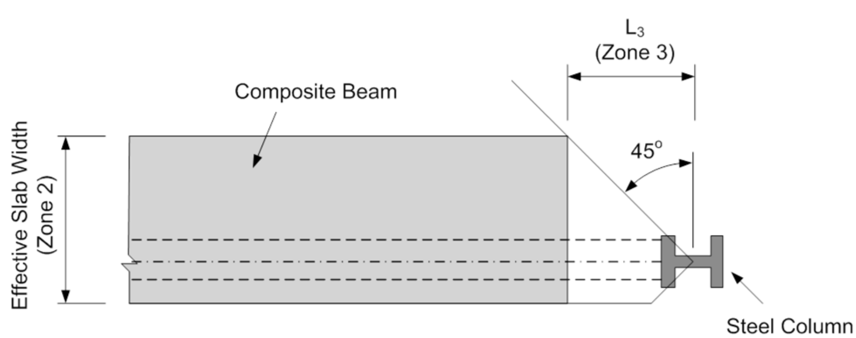


L_1 = 0.5 L_b = 15 ft (in our frame)

L_3 = maximum length from 45° projection at column centerline to outside of deck effective width

101

Composite Beam Stiffnesses

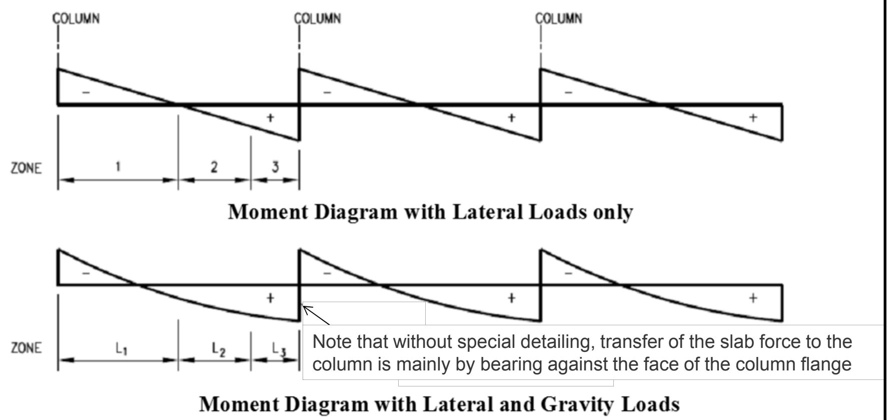


L_1 = 0.5 L_b = 15 ft (in our frame)

L_3 = maximum length from 45° projection at column centerline to outside of deck effective width = L_b / 8 = 3.75 ft (in our frame)

102

Composite Beam Stiffness



$L_1 = 0.5\ L_b = 15$ ft (in our frame)

L_3 = maximum length from 45° projection at column centerline to outside of deck effective width = $L_b / 8 = 3.75$ ft (in our frame)

$L_2 = L_g - (L_1 + L_2) = 11.25$ ft (in our frame)

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Composite Beam Stiffness

$L_3 = c\,L_g/2 = 3.75$ ft, $c = 0.25$ for our frame

$\alpha = I_{LB}/I_s$

$$I_{EQ} = \frac{I_s}{\left\{0.5\left[(c^3+7)+(1-c^3)/\alpha\right]-0.375\dfrac{\left[(c^2+3)+(1-c^2)/\alpha\right]^2}{\left[(c+1)+(1-c)/\alpha\right]}\right\}}$$

This equation gives the effective moment of inertia of a prismatic member that has the same net stiffness, for resisting the overall sidesway of the frame, as the 3-zone non-prismatic member

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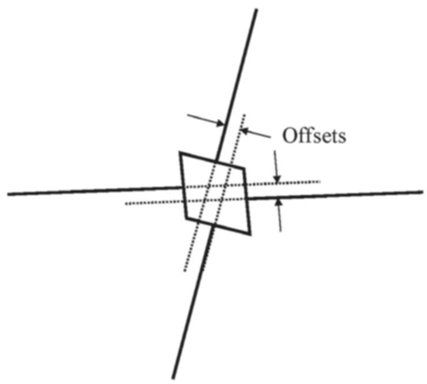
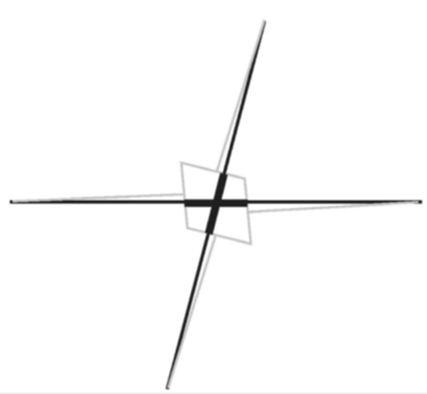
Finite Jt Size & Panel Zone Deformation

(Credits: Aswegan et al. 2015; Charney & Pathak 2008; Charney & Marshall 2006)



Krawinkler Panel Zone Model



Scissors Panel Zone Model

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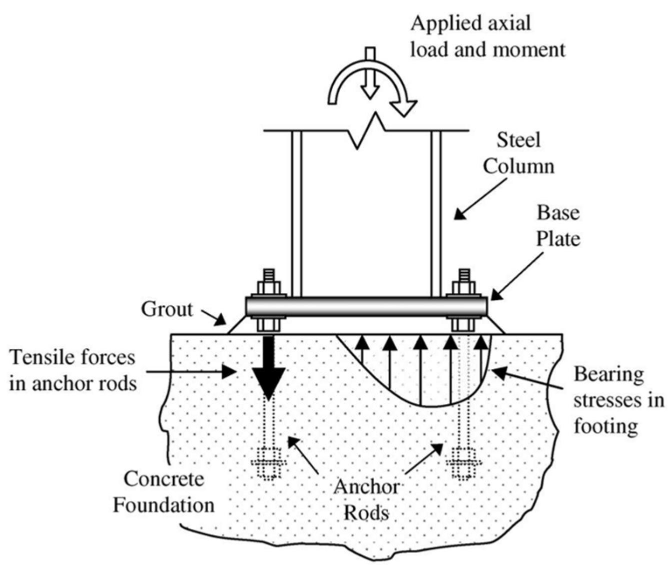
Column Base Stiffness

(Credits: Kanvinde et al. 2012)



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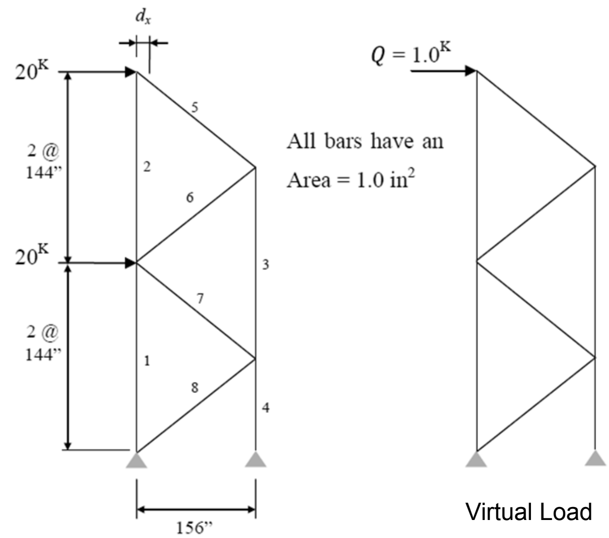
VW Methods & Lateral Stiffness Design

(Credits: Barrar 2009)



Real Loading

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Truss DISPAR Factors

Bar Number	Length (in)	Area (in^2)	Volume (in^3)	Real Force (kips)	Virtual Force (kips)	DPF (in.)	SI*1000 (in.$^{-2}$)
1	288	1.0	288.0	73.8	2.8	2.03	7.1
2	288	1.0	288.0	18.5	0.9	0.17	0.6
3	288	1.0	288.0	-36.9	-1.9	0.68	2.3
4	144	1.0	144.0	-110.8	-3.7	2.03	14.1
5	212	1.0	212.3	-27.2	-1.4	0.27	1.3
6	212	1.0	212.3	27.2	1.4	0.27	1.3
7	212	1.0	212.3	-54.4	-1.4	0.54	2.5
8	212	1.0	212.3	54.4	1.4	0.54	2.5
Summation			1857.3			6.53	

$$w_i = p_i \frac{P_i L_i}{A_i E} \qquad DPF_i = \frac{1}{Q}\int_0^L \left[\frac{M(x)_i m(x)_i}{EI_i} + \frac{P(x)_i p(x)_i}{A_i E} + \frac{V(x)_i v_i}{A_{v,i} E} + \frac{T(x)_i t(x)_i}{GJ_i}\right] dx$$

$$SI_i = \lim_{\Delta V \to 0}\left(\frac{\Delta DPF_i}{\Delta V_i}\right) = \left(\frac{dDPF_i}{dV_i}\right) = \frac{DPF_i}{V_i}$$

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Stability Design of Low- and Medium-Rise Steel Buildings — Don White

THAT'S IT!

Thanks For Attending!!

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Up Next…

- Session 7: March 16 –
  Stability Design – Advanced Applications
  by D.W. White, PhD
- This module investigates the application of modern structural analysis methods to several challenging stability design problems, including the sizing of general stability bracing for beams, beam-columns and frames, design of steel arches, and checking of stability during erection. The Direct Analysis Method as well as rigorous inelastic buckling analysis capabilities will be demonstrated.

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Thank You

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