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Course Description

Session 6: Building Analysis and Diaphragm Design

March 26, 2018

This session will review various analysis types and applicability to seismic design. The session will address effective structural modeling, including moment releases and effective stiffness. This session will also discuss second-order effects in the analysis, and calculating drift. The session will also address diaphragm design including determination of building-analysis forces, capacity-design forces and design of members at diaphragm openings.

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Learning Objectives

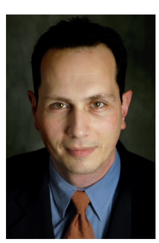
- Identify various analysis types and their applicability in seismic design.
- Identify how to properly model moment releases and effective stiffness.
- Describe the attributes of P-δ and P-Δ effects.
- Describe the components of designing members at diaphragm openings.

There's always a solution in steel.

Seismic Design in Steel: Concepts and Examples

Session 6: Building Analysis and Diaphragm Design

March 26, 2018



Rafael Sabelli, SE

structural STEEL

Course objectives

- Understand the principles of seismic design of steel structures.
- Understand the application of those principles to two common systems:
  - Special Moment Frames
  - Buckling-Restrained Braced Frames.
- Understand the application of design requirements for those systems.

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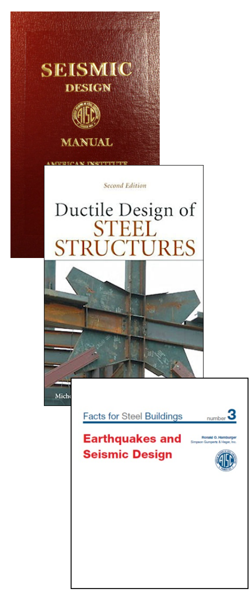
Resources

- AISC *Seismic Design Manual*
- *Ductile Design of Steel Structures*, Bruneau, Uang, and Sabelli, McGraw Hill.
- *Earthquakes and Seismic Design,* Facts for Steel Buildings #3. Ronald O. Hamburger, AISC.
- Other publications suggested in each session



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Other resources

- AISC Solutions Center
  - 866.ASK.*AISC* (866-275-2472)
  - Solutions@AISC.org
- AISC Night School
  - Nightschool@AISC.org

Course outline

Part I: Concepts

1. Introduction to effective seismic design
2. Seismic design of moment frames
3. Seismic design of braced frames
4. Seismic design of buildings

Course outline

Part II: Application

5. Planning the seismic design
6. **<u>Building analysis and diaphragm design</u>**
7. Design of the moment frames
8. Design of the braced frames

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Session 6: Building analysis and Diaphragm design

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Session topics

- Building Analysis
  - Lateral analysis methods
  - Load cases
  - Structural model
  - Design for stability & 2^{nd}-order analysis
  - Strength-design forces
- Diaphragm Design
  - Diaphragm forces
  - Capacity-design forces
  - Diaphragm analysis
  - Collector design
  - Collector-connection design
  - Diaphragm openings

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Building Analysis: Lateral analysis methods

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Lateral analysis methods

- Equivalent Lateral Force (ELF)
  - Standard procedure
  - Slightly conservative
- Modal Response Spectrum Analysis (MRSA)
  - ASCE 7-10: Scaled to 85% of ELF base shear
  - ASCE 7-16: Scaled to 100% of ELF base shear
  - Always: $(M_{ot}/V)_{MRSA} < (M_{ot}/V)_{ELF}$
  - Slightly cumbersome
- Response History Analysis
  - Cumbersome; only used for special conditions

ASCE 7 §12.9.4.1

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Lateral analysis methods

- 2D Analysis
  - 2D with flexible diaphragms
  - 2D with rigid or semi-rigid diaphragms
    - No torsional irregularity
    - Parallel and orthogonal frames (no skewed frames)
    - No out-of-plane offsets of the seismic system
- 3D
  - Everything else
  - Technically required in some cases for flexible diaphragms, but results are the same as 2D
    - Orthogonal combination requirements are not waived

ASCE 7 §12.7.3

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Lateral analysis methods

- This example
  - Diaphragms are rigid per ASCE 7 §12.3.1.2
    - Concrete-filled steel deck
    - Span-to-depth ratio <3
    - No Horizontal irregularities
  - Use ELF for simplicity & clarity
    - MRSA slightly more economical
  - Use 2D analysis for simplicity & clarity
    - 3D slightly more economical

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Load cases

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Load Cases

- 2 principal axes
  - Orthogonal combination not required
    - No skewed frames in this example
    - No columns shared by orthogonal frames
- Accidental torsion
  - Required for non-flexible diaphragms
  - 5% offset of mass in either direction
    - For simplicity, conservatively neglect torsional resistance of orthogonal frame (that is, 2D analysis)

ASCE 7 §12.8.4.2

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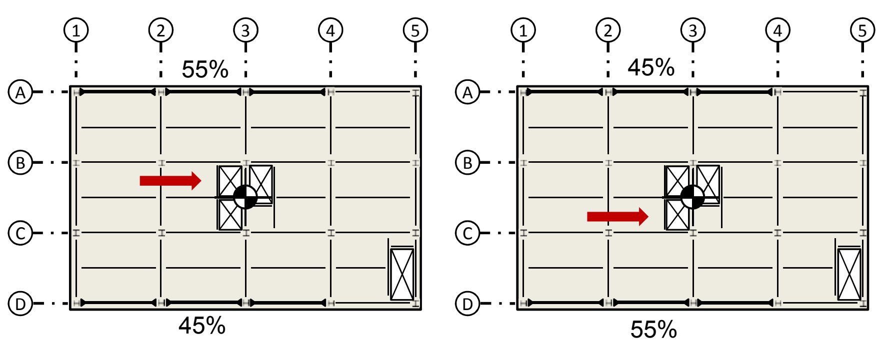
Load Cases



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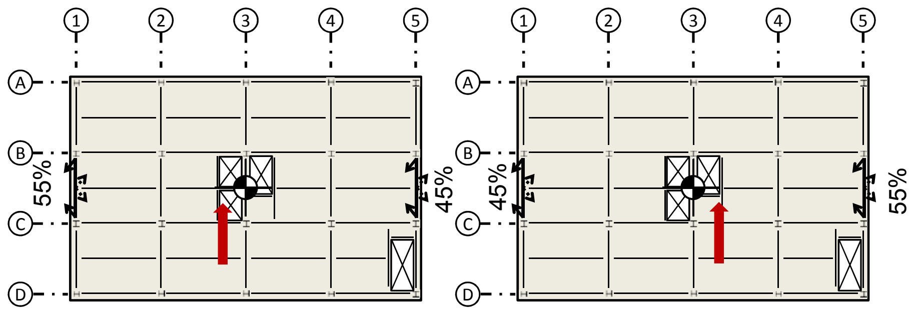
Load Cases



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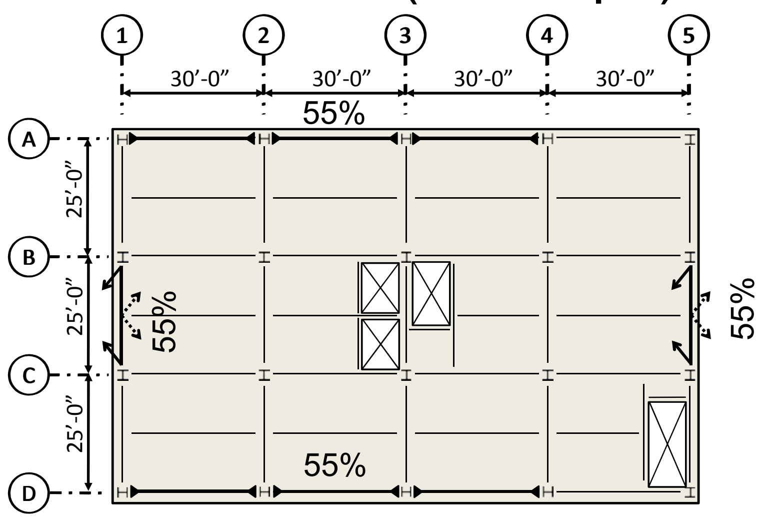
Load Cases (envelope)



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Base Shear

- From Session 5
  - SMF
    - V = 0.0818*3313K
      = 271K
  - BRBF
    - V = 0.100*3313K
      = 331K

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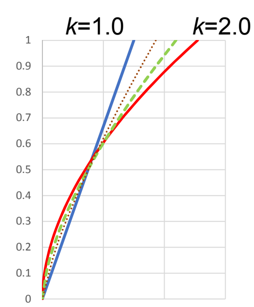
Vertical Distribution

$$C_{vx} = \frac{w_x h_x^{\ k}}{\sum_{i=1}^{n} w_i h_i^{\ k}}$$

- $T \leq 0.5$s k=1.0
- $T \geq 2.5$s k=2.0
- 0.5s<T<2.5s interpolate



Lateral force profile

ASCE 7 §12.8.3

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Vertical Distribution

$$C_{vx} = \frac{w_x h_x^{\ k}}{\sum_{i=1}^{n} w_i h_i^{\ k}}$$

- SMF
  - $C_u T_a$ =0.918s
  - k=1.21

Level	w_i, kips	h_i, (ft)	$w_i h_i^k$	C_{vx}	F_x, (kips)
Roof	708.4	51.5	83,115	0.373	101
4th	868.1	39	72,778	0.327	89
3rd	868.1	26.5	45,617	0.205	55
2nd	868.1	14	21,092	0.095	26
Total	3312.9		222,602	1.000	271

ASCE 7 §12.8.3

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Vertical Distribution

$$C_{vx} = \frac{w_x h_x^{\ k}}{\sum_{i=1}^{n} w_i h_i^{\ k}}$$

- BRBF
  - $C_u T_a$ =0.100s
  - k=1.125

Level	w_i, kips	h_i, (ft)	$w_i h_i^k$	C_{vx}	F_x, (kips)
Roof	708	51.5	59,687	0.362	120
4th	868	39.0	53,499	0.325	108
3rd	868	26.5	34,639	0.210	70
2nd	868	14.0	16,898	0.103	34
Total	3313		164,723	1.000	331

ASCE 7 §12.8.3

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Structural model

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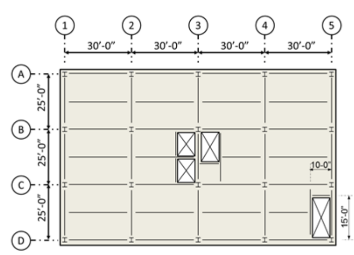
Structural Model

- Diaphragms are
  - o Flexible,
  - o Rigid, or
  - o Semi-rigid
- Rigid diaphragms assumed, if
  - o Concrete deck
  - o *L/d*≤3
    - 120'/75'=1.6 OK
  - o No horizontal irregularities



ASCE 7 §12.3.1

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Structural Model

- Gravity framing (if included in model)
  - o Design seismic system for 100% of lateral forces
  - o Prevent shear in gravity columns in seismic analysis
    - Pin columns top and bottom
      - o Except SMF
    - Pin non-frame beams connecting to SMF columns
- Column size change at floor

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Moment frames

- RBS connection
  - o Reduced beam stiffness
    - (90% if maximum RBS reduction)
  - o Prequalification limits on members
- No rigid-end offset
- Mesh columns into 4 segments (for P-δ effects, dependent on software capability)
  - o Or use B_1 factor
- Use non-composite beam stiffness
- Do not assume fully rigid bases

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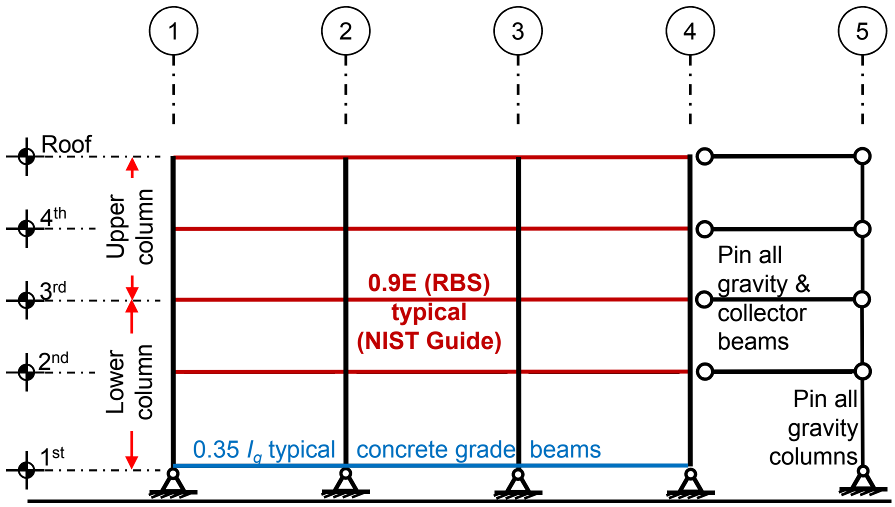
Moment-frames



Base plate and foundation rotational restraint may be quantified for additional stiffness

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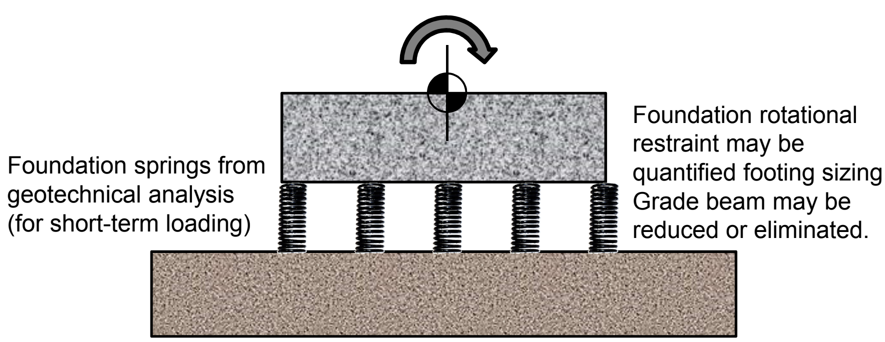
Moment-frames

Foundation rotational restraint



ASCE 7 §12.13.3

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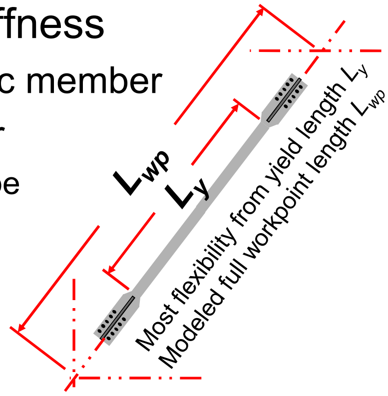
Braced frames

- Pin all members
- Use increased brace stiffness
  - o Represents non-prismatic member
  - o K_F =1.4 per manufacturer
    - Varies with connection type
    - Varies with area
    - Varies with length

$K \sim EA_{sc}/L_y = K_F EA_{sc}/L_{wp}$

$1.2 \leq K_F \leq 1.8$



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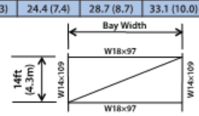
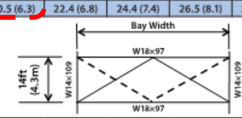
Approximate Stiffness Modification Factors, KF[1,2,7]

Sizes shown are representative of typical BRB sizes. Information on intermediate and larger sizes is available upon request.

F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in² (cm²)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	1.37	1.34	1.31	1.29	1.28	1.33	1.30	1.27	1.25	1.24
3.0 (19)	103 (448)	1.36	1.33	1.31	1.29	1.27	1.32	1.29	1.27	1.25	1.23
4.0 (26)	137 (613)	1.47	1.42	1.37	1.35	1.32	1.42	1.37	1.34	1.31	1.29
5.0 (32)	171 (754)	1.41	1.37	1.34	1.31	1.29	1.36	1.33	1.30	1.28	1.26
6.0 (39)	205 (919)	1.51	1.45	1.40	1.37	1.34	1.45	1.41	1.37	1.34	1.31
7.0 (45)	239 (1060)	1.48	1.42	1.38	1.35	1.33	1.43	1.38	1.35	1.32	1.30
8.0 (52)	274 (1225)	1.53	1.47	1.42	1.38	1.36	1.47	1.42	1.38	1.35	1.33
9.0 (58)	308 (1367)	1.54	1.47	1.42	1.39	1.36	1.48	1.43	1.39	1.36	1.33
10.0 (65)	342 (1532)	1.59	1.51	1.45	1.41	1.38	1.52	1.47	1.42	1.38	1.35
11.0 (71)	376 (1673)	1.60	1.51	1.46	1.41	1.38	1.53	1.47	1.43	1.39	1.36
12.0 (77)	410 (1814)	1.67	1.57	1.50	1.45	1.41	1.59	1.53	1.47	1.43	1.39
14.0 (90)	479 (2121)	1.60	1.52	1.46	1.42	1.39	1.53	1.47	1.43	1.40	1.37
16.0 (103)	547 (2427)	1.69	1.59	1.52	1.47	1.43	1.61	1.54	1.49	1.44	1.41
18.0 (116)	616 (2733)	1.69	1.59	1.52	1.47	1.43	1.61	1.54	1.49	1.45	1.41
20.0 (129)	684 (3040)	1.66	1.57	1.50	1.46	1.42	1.59	1.52	1.47	1.44	1.40
22.0 (142)	752 (3346)	1.76	1.65	1.57	1.51	1.47	1.68	1.60	1.53	1.48	1.44
24.0 (155)	821 (3652)	1.81	1.68	1.59	1.53	1.47	1.69	1.61	1.57	1.52	1.48
26.0 (168)	889 (3959)	1.82	1.69	1.60	1.54	1.46	1.66	1.58	1.59	1.53	1.48
28.0 (181)	958 (4265)	1.83	1.70	1.61	1.55	1.48	1.69	1.61	1.59	1.54	1.49
30.0 (194)	1026 (4571)	1.81	1.69	1.60	1.54	1.49	1.70	1.62	1.58	1.53	1.48
Workpoint Length, ft (m)		20.5 (6.3)	24.4 (7.4)	28.7 (8.7)	33.1 (10.0)	37.7 (11.5)	20.5 (6.3)	22.4 (6.8)	24.4 (7.4)	26.5 (8.1)	28.7 (8.7)





STORY HEIGHT: 14ft (4.3m)

Brochure from Core Brace

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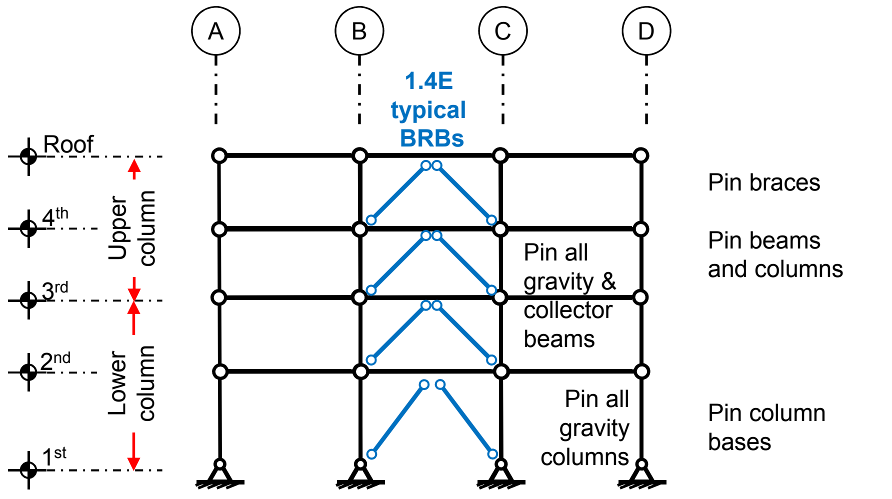
Braced frames



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Design for stability & Second-order analysis

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Design for stability AISC 360

- Direct Analysis Method (DAM)
  - Analysis
    - Decreased member stiffness
      - Different model used for drift, period
    - 2nd-order effects must be included
  - Can be used for any level of second-order effect
    - But B_2>1.5 is unusual
  - K=1 for columns
  - Requires minimum lateral load

AISC 360 §C1.1

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Design for stability AISC 360

- Effective Length Method (ELM)
  - Analysis
    - 2nd-order effects must be included
    - Uses same model for strength, drift, period
  - Reduced column strength
  - Can be used for B_2≤1.5
  - K=1 for
    - Braced-frame columns
    - Moment-frame columns with B_2≤1.1
  - Requires minimum lateral load

AISC 360 Appendix 7 §7.2

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Design for stability AISC 360

- First-Order Analysis Method (FOAM)
  - Analysis
    - 2nd-order effects not included
    - Uses same model for strength, drift, period
  - 2nd-order effects addressed by additional lateral load
    - $N_i = 2.1\frac{\Delta}{h}Y_i \approx 0.008Y_i$ for SMF at drift limit of $0.02h$
    - Increase C_s by 12% in this example (0.0818+0.0099=0.0917)
      - Not a penalty if the frame is governed by drift
  - Can be used for $B_2 \leq 1.5$
  - K=1 for columns
  - P_u/P_y for columns **$\leq$ 0.5**

AISC 360 Appendix 7 §7.3

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Design for stability: Braced Frames

- Typically governed by strength
- Use ELM (Appendix 7 §7.2)
  - Simpler to use same model for drift
- Use K=1
  - Always OK for braced frames
- Perform 2nd order analysis
  - Or use B_2

AISC 360 Appendix 7

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Design for stability: Moment Frames

- Typically governed by drift
- Use ELM if $B_2 \leq 1.1$
- If $B_2 > 1.1$
  - o Size frame for drift
  - o Check strength with 1st-order analysis (Appendix 7 §7.3)
    - Supplemental lateral load ~ 0.084 P_{story} for SMF at drift limit
      - o B_2 not required
    - Or DAM
  - o Don't try to calculate K factors!
- If $B_2 > 1.5$ redesign!
  - o Will not meet ASCE 7 §12.8.7 stability check

AISC 360 Appendix 7

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Second-order effects

- Second-order analysis
  - o Equilibrium in deformed condition
  - o Analysis must include all gravity load
- Approximate second-order analysis
  - o Appendix 8
  - o B_1 Beam-columns
  - o B_2 The entire lateral-load-resisting system
- ASCE 7 §12.8.7 requires consideration of second order effects for forces and displacements when $\theta > 0.1$ ($\theta \sim 1 - {}^{1}/B_2 < B_2 - 1$)

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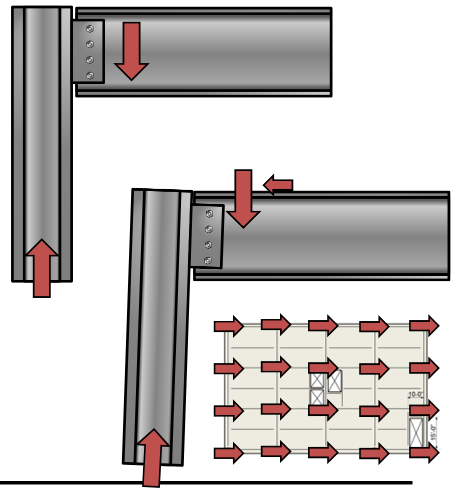
Second-order effects (P-Delta)

- Columns resist the story gravity force
- Lateral loads induce drift
  - Columns slope
  - Column axial force has horizontal component
  - Horizontal component is additional thrust on diaphragm



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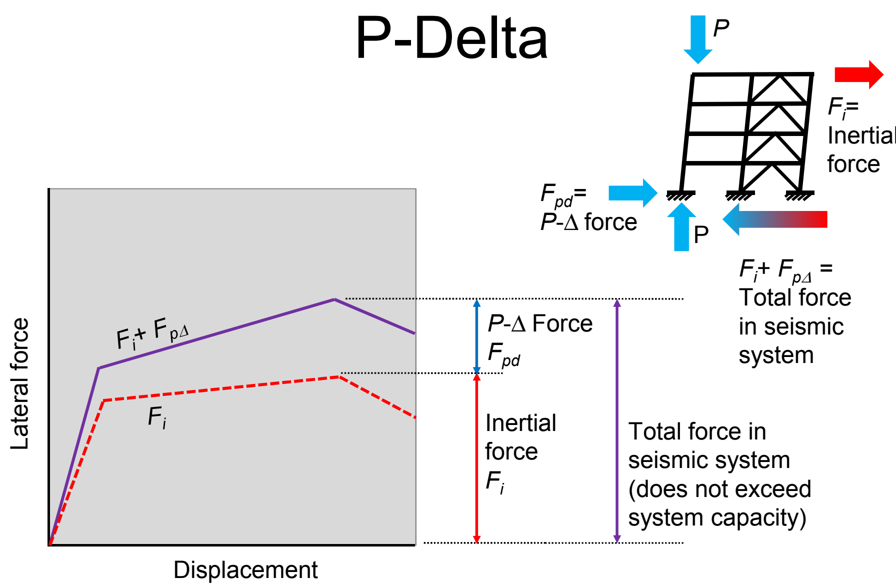
P-Delta



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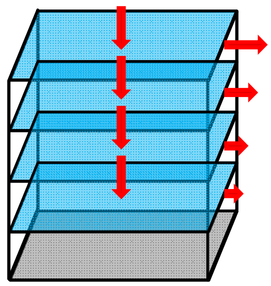
Second-order effects (approximate 2nd-order analysis)



$$B_2 = \frac{1}{1 - \frac{P_{Story}\Delta_{elastic}}{R_m V_{Story} h}}$$

Largest B_2 at base for seismic

- B_2 calculation
  - P_{story}
    - Gravity load acting on story
    - Includes gravity load above
    - Use consistent dead load with seismic mass
    - Use live load from load combination
  - V_{story}
    - Story shear

AISC 360 Appendix 8

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Second Order Effects: SMF

$$B_2 = \frac{1}{1 - \frac{P_{Story}\Delta_{elastic}}{\left[1 - 0.15\frac{P_{mf}}{P_{Story}}\right] V_{Story} h}}$$

$$B_2 = \frac{1}{1 - \frac{0.0036 P_{Story}}{0.93 V_{Story}}}$$

	B_2
Roof	1.02
4th	1.03
3rd	1.03
2nd	1.04

- Assume drift-controlled
  - $\Delta_{elastic}$ = 0.02h/C_d = 0.0036h
- Use P-Δ gravity load
  - P=1.0D+0.5L
- Assume P_{mf} = ½ P_{story}
- B_2 = 1.04 (largest) <1.1
  - Use ELM
  - K=1

AISC 360 Appendix 8

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Second Order Effects: BRBF

$$B_2 = \frac{1}{1 - \frac{P_{Story}\Delta_{elastic}}{V_{Story}h}}$$

$$B_2 = \frac{1}{1 - \frac{0.004P_{Story}}{V_{Story}}}$$

	B_2
Roof	1.02
4th	1.03
3rd	1.03
2nd	1.04

- Assume drift
  - $\Delta_{elastic}$ = 0.02*h*/C_d
    = 0.004*h*
- Use *P*-Δ gravity load
  - P=1.0D+0.5L
- B_2 = 1.04 (largest)

AISC 360 Appendix 8

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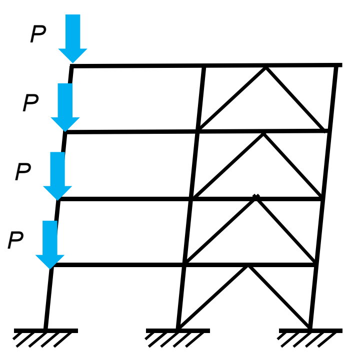
Second Order Effects: Analysis



- Use "leaner" column to resist all gravity-column axial forces
- Use software capable of performing second-order analysis
- Results
  - Amplified lateral-system forces
  - Amplified drift

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Second Order Effects

- ASCE 7 stability check

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} = \frac{P_x \Delta_{elastic}}{V_x h_{sx}} = \frac{P_x / h_{sx}}{K_x}$$

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

Where β is the ratio of shear demand to shear capacity

  - θ ~ 1−1/B_2 = 0.04 (max)

$$\theta_{max} = \frac{0.5}{0.9 * 5} = 0.11$$

  - OK

ASCE 7 §12.8.7

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Stability & 2nd-order analysis

- Due to high seismic demands at this location, system is required to be stiff enough so that second-order effects are minor
  - First-order effects are large
    - Second-order effects are relatively small
  - At sites with low seismic demands second-order effects are more important
    - i.e., B_2 and θ will be larger

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Stability & 2nd-order analysis

- Use ELM for SMF & BRBF
  - Same model for strength and drift
    - No DAM stiffness reduction
    - No FOAMy supplemental lateral force
- For clarity in this example, second-order analysis software not used
  - Approximate second-order analysis (Appendix 8)
    - Amplify lateral forces and displacements by B_2
    - Amplify non-sway (gravity) moments by B_1

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There's always a solution in steel.

Analysis forces

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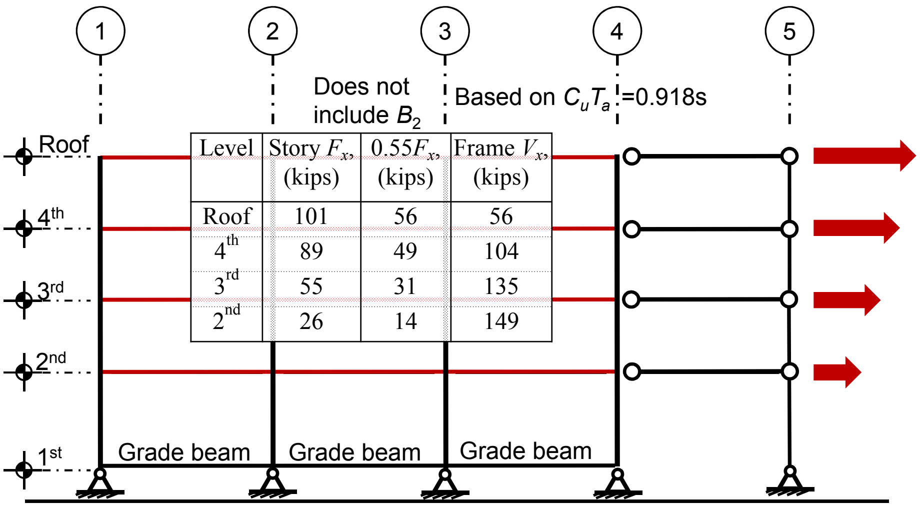
Moment-frames



Level	Story F_x, (kips)	$0.55F_x$, (kips)	Frame V_x, (kips)
Roof	101	56	56
4th	89	49	104
3rd	55	31	135
2nd	26	14	149

Design of SMF presented in Session 7
Forces derived in Session 5

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Moment frames

- Moment frames likely drift-controlled
  - Design for drift
  - Check strength after member selection
- Design base shear strength check subject to maximum period C_uT_a
- Drift not subject to maximum period
  - Design in Session 7 tracks period with iteration
  - Required stiffness (and thus period) can be approximated using spectrum & drift limit

ASCE 7 §12.8.2.1

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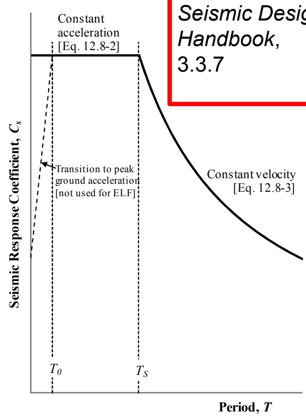
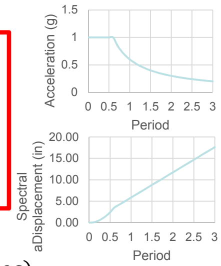
Drift-determined period

Adapted from Naeim's *Seismic Design Handbook*, 3.3.7

$$S_a = C_s \left(R/I_e\right) g = \frac{S_{D1} g}{T/sec}$$

$$S_d = \frac{S_a}{\omega^2} = \frac{S_a T^2}{4\pi^2} = \frac{S_{D1} g T(sec)}{4\pi^2}$$





$$S_d \approx \frac{2}{3}\Delta_{roof} = \frac{2}{3}0.02h = \frac{S_{D1} g T(sec)}{4\pi^2}$$

$$T = \frac{2}{3}\frac{0.02h4\pi^2}{S_{D1} g\ (sec)} = \frac{2}{3}\frac{0.025h\ (sec)}{S_{D1}(feet)}$$

$$T = \frac{2}{3}\frac{0.025(51.5')(sec)}{0.6(feet)} = 1.4sec$$

⅔ is an approximate correction factor
- Spectral displacement is not roof displacement
- 1st-mode mass participation <100%

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Moment frames

- Design base shear based on maximum period $C_u T_a$ =0.92 sec
- Drift-determined period =1.4 sec
  - Corresponds to **2.1** T_a
- Recommendation for SMF
  - For first iteration use either
    - Drift-determined period
    - 2.0 T_a
  - Use calculated period for subsequent iteration

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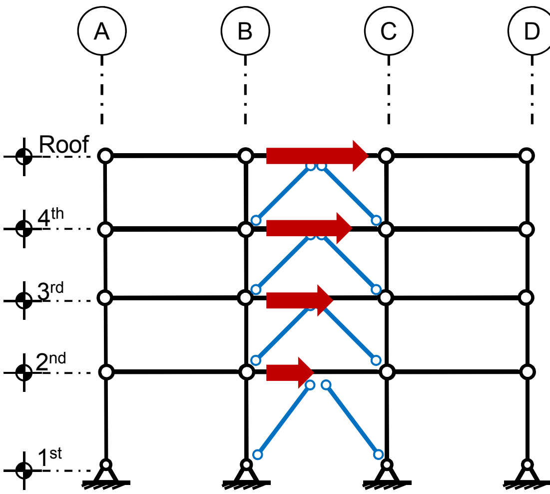
Braced frames



Level	Story F_x, (kips)	$0.55F_x$, (kips)	Frame V_x, (kips)
Roof	120	66	66
4th	108	59	125
3rd	70	38	164
2nd	34	19	182

Does not include ρ=1.3
Does not include B_2

Design of BRBF presented in Session 8
Forces and redundancy from Session 5

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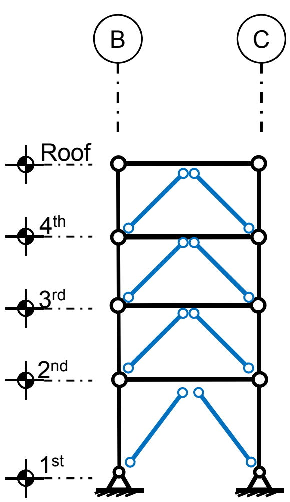
Braced-frame forces

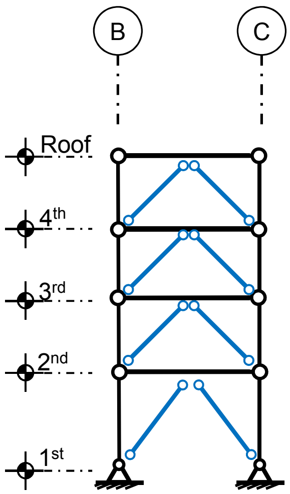


Level	Frame F_x, (kips)	B_2	ρ	Frame $\rho B_2 F_x$, (kips)	Frame $\rho B_2 V_x$, (kips)
Roof	66.0	1.02	1.30	87.6	88
4th	59.2	1.03	1.30	79.3	167
3rd	38.3	1.03	1.30	51.3	218
2nd	18.7	1.04	1.30	25.3	243

Design of BRBF presented in Session 8
Forces derived in Session 5

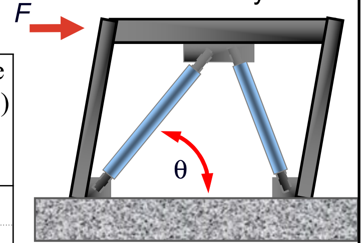
60

Braced-frame forces



Level	Brace Force $\rho B_2 P_x$, (kips)
4th	62
3rd	118
2nd	154
1st	183

Assume braces resist 100% of story shear



$$P_u = \frac{F}{2\cos\theta}$$

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There's always a solution in steel.

Diaphragm Design

structural STEEL

Diaphragm design

- Typically done after design of frames
  - Requires consideration of transfer forces between frames
    - 3D building analysis
    - Indeterminate analysis using designed member stiffness
  - Forces may be limited by yielding elements
- This example
  - Done now to allow full sessions for each system

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Diaphragm Forces

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad 0.2S_{DS}Iw_{px} \geq F_{px} \geq 0.4S_{DS}Iw_{px}$$

Level	Story F_{px}, (kips)	
Roof	141.7	$0.2S_{DS}Iw_{px}$
4th	173.6	$0.2S_{DS}Iw_{px}$
3rd	173.6	$0.2S_{DS}Iw_{px}$
2nd	173.6	$0.2S_{DS}Iw_{px}$

ρ=1.0 for diaphragm design (ASCE 7 §12.3.4.1)

B_2 (i.e., 2nd order amplification) applies to diaphragm design (ASCE 7 §12.8.7)

ASCE 7 §12.10.1

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There's always a solution in steel.

Diaphragm analysis

structural STEEL

Diaphragm analysis

- Determine diaphragm shear
- Determine collector forces
  - Apply Ω_o factor per ASCE 7 §12.10.2
- Determine chord forces
  - Diaphragm equivalent-beam moments
  - Divide by depth
- B_2 applies to entire lateral analysis
  - Incorporated in member-design forces

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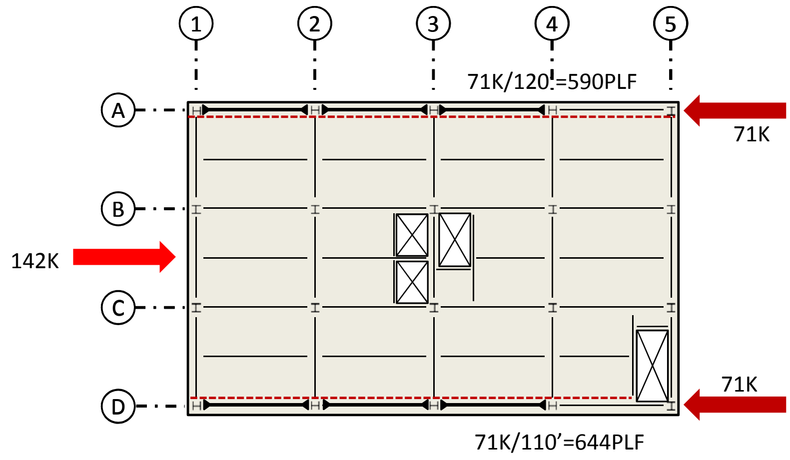
Diaphragm analysis: Roof X



Does not include B_2

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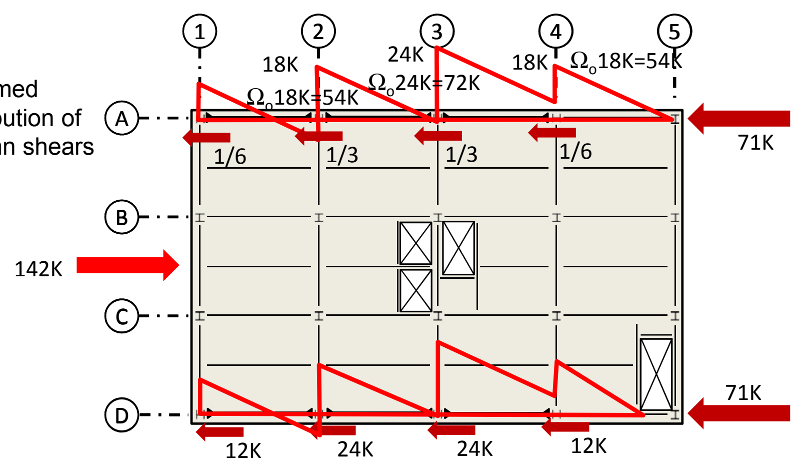
Diaphragm analysis: Roof X

Assumed distribution of column shears



Ω_o= 3.0 for SMF

Does not include B_2

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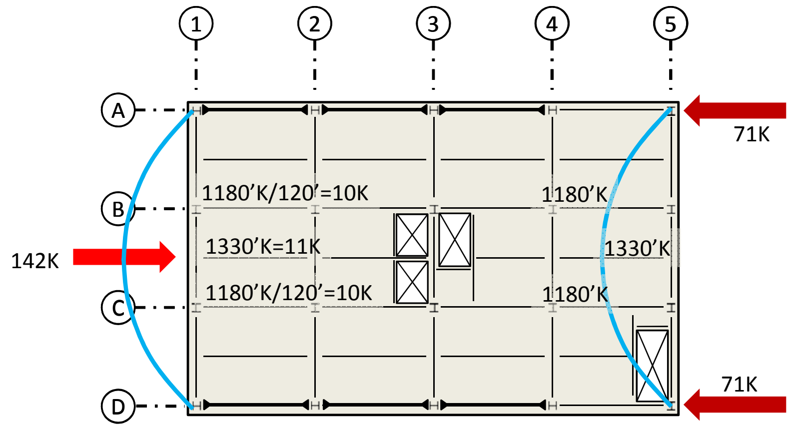
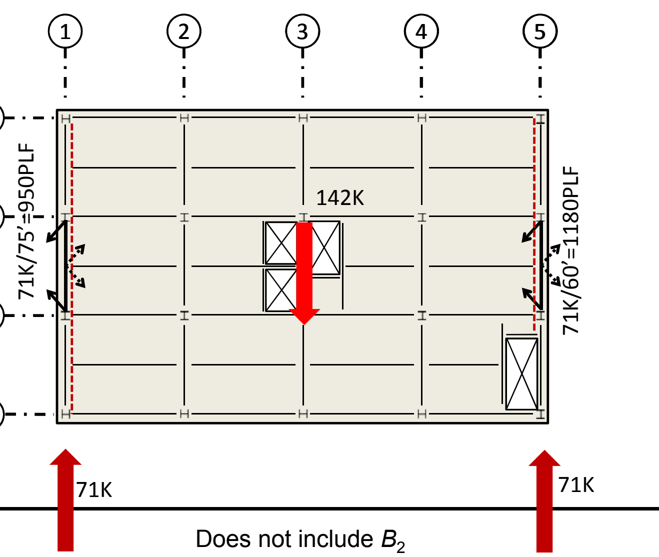
Diaphragm analysis: Roof X



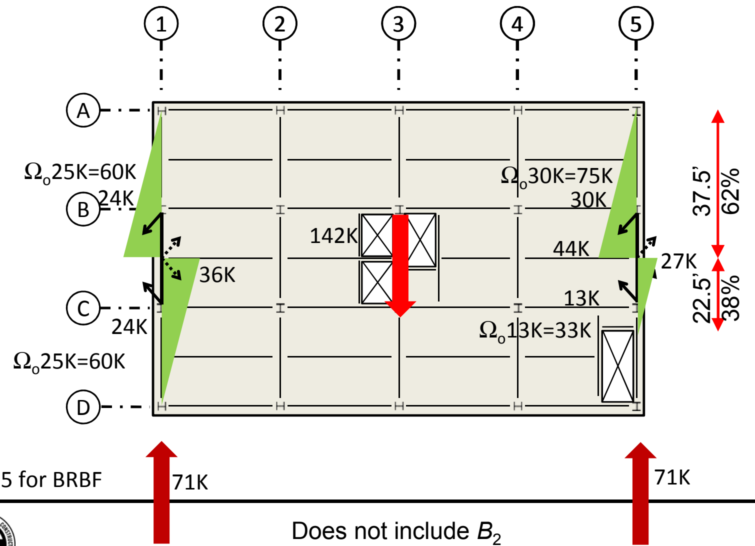
Does not include B_2

Diaphragm analysis: Roof Y



Does not include B_2

Diaphragm analysis: Roof Y



Ω_o= 2.5 for BRBF

Does not include B_2

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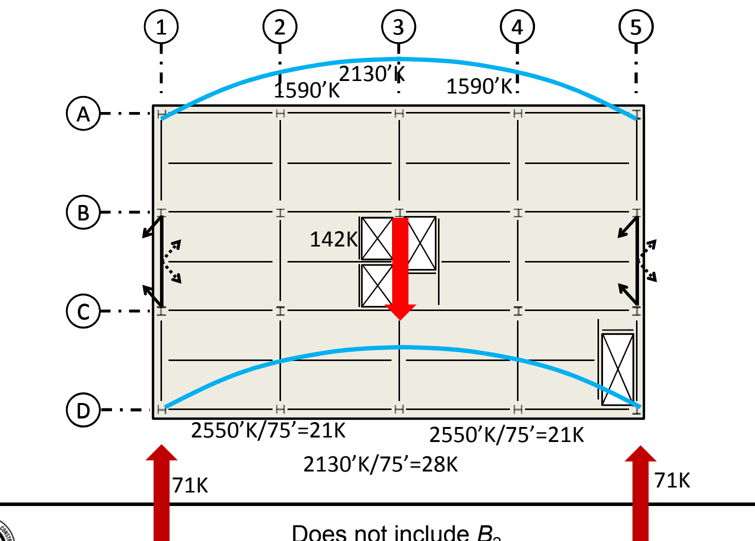
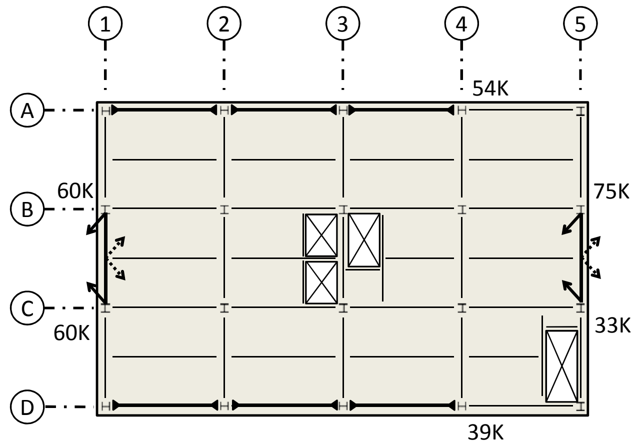
Diaphragm analysis: Roof Y



Does not include B_2

72

Chord/collector forces



Does not include B_2

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Capacity-design forces

There's always a solution in steel.

Capacity-design forces

- Per ASCE 7 §12.4.3 the overstrength seismic load, $\Omega_o E_h$, need never be taken as greater than the capacity-limited seismic load effect, (E_{cl} in ASCE 7 2016)
- Capacity design can only happen after frames designed
- In this example we will show capacity-design forces prior to showing frame design

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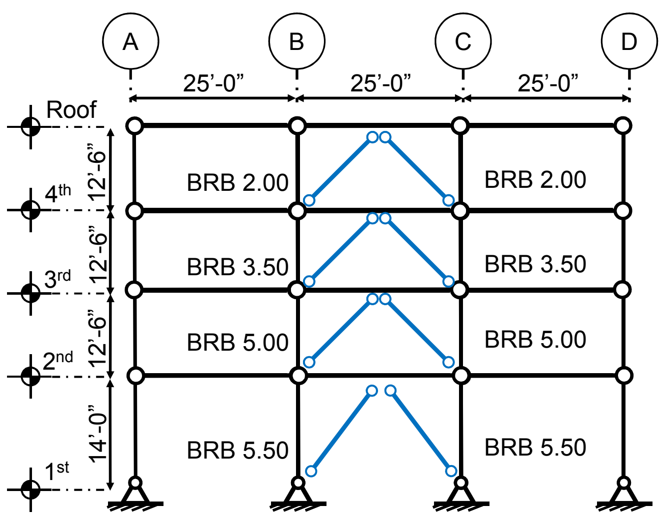
Brace sizes



Design of BRBF presented in Session 8

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Brace capacity (4th floor)

- Tension
  - $\omega R_y F_{ysc} A$
  - $1.4(42ksi)2.00in^2$ = 118K
- Compression
  - $\beta\omega R_y F_{ysc} A$
  - $1.15*1.4(42ksi)2.00in^2$ = 135K
- Horizontal component:
  - $(118K+135K)*\cos\theta$ = 179K
  - $< B_2 \Omega_o V_{frame} = 1.02*2.5*66K$ = 168k
- Use capacity forces for roof collectors

AISC 341 §F4.2a

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There's always a solution in steel.

Diaphragm design

structural STEEL

Deck selection

- Maximum shear
  - 1.18KLF
  - B_2*1.18KLF = 1.20KLF
- Design composite deck
  - Reinforced concrete section
  - Consider only topping above steel deck
    - 3.25” light weight concrete topping
    - #3 A614 Gr. 60 bars @12” each way

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Deck selection

- Design composite deck
  - v_c $= 2\lambda d \sqrt{f'_c}$
    = (2)0.75(3.25”)(4000)½ (12”/ft)
    = 3700 plf
  - v_s $= A_s f_y$
    = 0.11in²/ft * 60 ksi
    = 6600 plf
  - ϕV_n $= \phi (v_c + v_s)$
    = 0.75*(3700plf+6600plf)
    = 7725 plf > 1200 plf

ACI 318 §11.2

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Shear transfer

- Design for Ω_o & B_2
  - SMF
    - 1.02*3*0.644KLF = 1.97KLF
  - BRBF
    - 2.5*1.20KLF=3.00KLF
- Provide ¾"x4" studs @ 24" on collectors

$$l_{s,req'd} = 2\text{ in} + 1.5\text{ in} = 3.5\text{ in} < 4\text{ in}$$

- 4" stud projects above flute 2"

AISC 341 §B5.1

81

Shear transfer

- Stud strength:
  - R_g = 1.0 for one row with deck perpendicular (worst case)
  - R_p = 0.6 for one row with deck perpendicular (worst case)
  - F_u = 65 ksi

$$Q_n = 0.5A_{sc}\sqrt{f'_c E_c}$$
$$Q_n \le R_g R_p A_{sc} F_u$$
$$A_{sc} = \frac{\pi}{4}\left(\tfrac{3}{4}in\right)^2 = 0.44in^2$$
$$f_c' = 4000psi$$
$$E_c = w_c^{1.5}\sqrt{f'_c}$$
$$E_c = (115pcf)^{1.5}\sqrt{4ksi}$$
$$E_c = 2466ksi$$

AISC 360 §I8.2a

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Shear transfer

- Stud strength:

$$Q_n = 0.5 \times 0.44 \text{ in}^2 \sqrt{4 \text{ ksi} \times 2466 \text{ ksi}} = 21.85 \text{ k}$$

$$Q_n \leq 1.0 \times 0.6 \times 0.44 \text{ in}^2 \times 65 \text{ ksi} = 17.2 \text{ k}$$ Typically governs

$$Q_n = 17.2 \text{ k}$$

$$\phi Q_n = 0.65 \times 17.2 \text{ k} = 11.2\text{k}$$

- Spaced @ 24”
  - 5.6 KLF>3.0KLF

AISC 360 §I8.2a

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Collector design

There's always a solution in steel.

structural STEEL

Collector design

- Combined flexure and axial
  - o Compression governs over tension for collector member
    - Include P-δ (in the form of B_1)
    - Perform Chapter H interaction
- Tension may govern for collector connection
  - o Compression path through deck typically neglected

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Collector design

- Flexure
  - o Composite strength
  - o Continuously braced for LTB
- Compression
  - o Flexural buckling
    - Major axis
    - Minor axis braced by composite deck
  - o Torsional or flexural-torsional buckling
    - Twisting about restrained top flange

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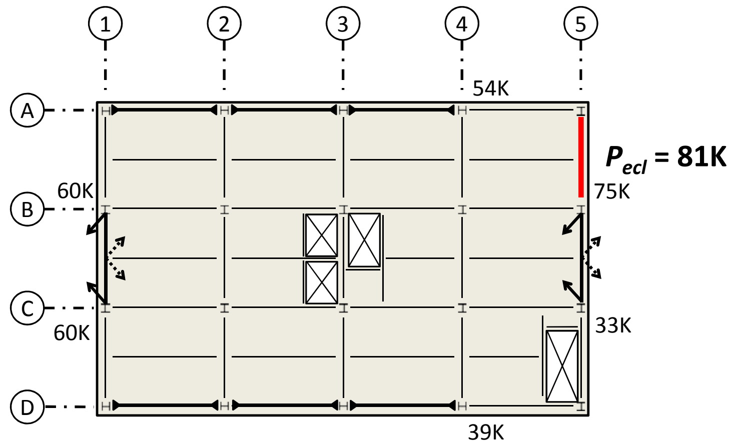
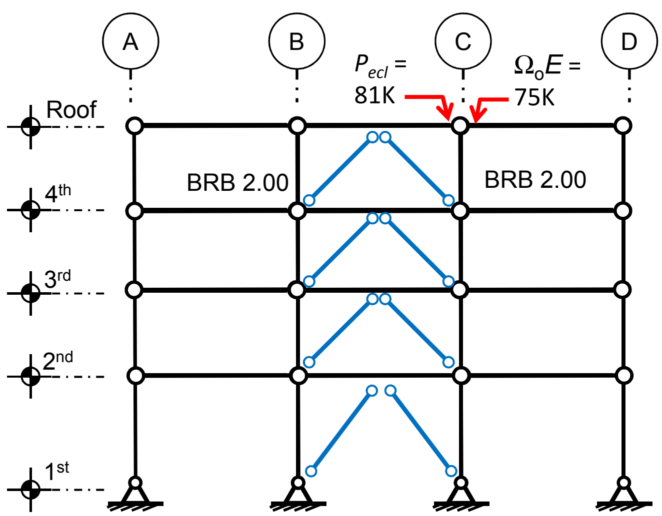
Collector design



Capacity design: P_{ecl} = 75K*BRB_Capacity/$\Omega_o V_{frame}$ =75K*179K/(2.5*66K)= 81K

Collector forces



Design both connections for **81K**

Design of BRBF presented in Session 8

Collector Loads

- $M_u = 1.4D + 0.5L + E_{cl}$ combo CLLC-1
  - M_u = 1.4(98.7'K) + 0.5(0'K)+ 0'k = 138'K
- $P_u = 1.4D + 0.5L + E_{cl}$ combo CLLC-1
  - P_u = 1.4(0K) + 0.5(0K) + 81K = 81K
- $V_u = 1.4D + 0.5L + E_{cl}$ combo CLLC-1
  - V_u = 1.4(9.4K) + 0.5(0K) + 0K = 13.1K
  - Beam shear design not presented
- Use W18x50 (per Seismic Design Manual)

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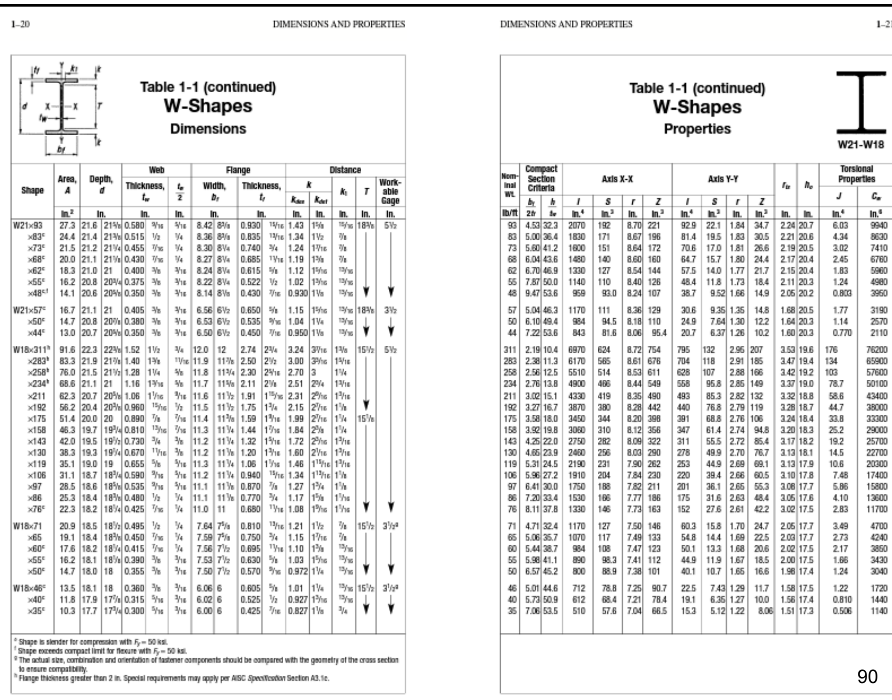

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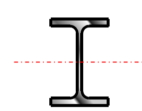
Collector design

- Use W18x50 (from Seismic Design Manual Example 8.4.1)
  - o Member properties (units per Manual)

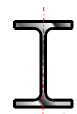
W18x50							
A 14.7	d 18.0	t_w 0.355	b_f 7.50	t_f 0.570	k_1 13/16	$b_f/2t_f$ 6.57	h/t_w 45.2
I_x 800	Z_x 101	S_x 88.9	r_x 7.38	$h/t_w > 1.49\sqrt{E/F_y} = 35.9$			
I_y 40.1	Z_y 16.6	S_y 10.7	r_y 1.65	Web is not compact		J 1.24	C_w 3040

Collector design

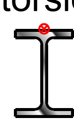
- Compressive Strength
  - o Major axis
    - $(KL)_x$ = 25’-0”



  - o Minor axis
    - $(KL)_y$ = 0’-0”



  - o Constrained-axis flexural-torsional buckling
    - $(KL)_{CAFT}$ = 12’-6”



Collector design

- Compressive Strength
  - Major axis buckling
    - KL/r_x = 300"/7.38" = 40.7
    - From Table 4-22: KL/r=41: ϕF_{cr} = 39.8ksi
      - ϕF_{cr} A= 39.8ksi (14.7in^2)= 585K

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Collector design

- Compressive Strength
  - CAFT buckling
    - $F_e = 0.9\left[\frac{\pi^2 E\left[C_w + I_y\left(d/2\right)^2\right]}{(K_z L)^2} + GJ\right]\frac{1}{I_x + I_y + \left(d/2\right)^2 A_g}$
    - AISC EJ (2013 Q4)
      - differs by factor of 0.9 from 2nd Edition Seismic Design Manual
    - $F_e = 0.9(46.2ksi)$ =41.6ksi
    - QF_y/F_e = 1(50ksi)/41.6ksi = 1.20<2.25

Torsional and Constrained-Axis Flexural-Torsional Buckling Tables for Steel W-Shapes in Compression

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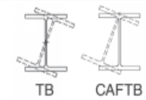
Collector design

F_y = 50 ksi

Table 1. (continued)
Torsional Buckling Design Strength in Axial Compression
$\phi_c P_n$, kip



Shape		W18x								W16x					
lb / ft		50		46		40		35		100		89		77	
Unbraced Length $K_z L$ (ft)	10	509	465	435	366	361	306	301	252	1190	1140	1050	1010	898	864
	11	492	439	418	336	345	280	286	229	1170	1110	1030	981	880	838
	12	475	413	401	308	329	255	271	205	1150	1080	1010	953	863	812
	13	459	386	386	281										786
	14	443	360	371	257										760
	15	428	335	357	235										734
	16	414	311	344	216	276	170	217	131	1090	969	948	843	796	709
	17	399	289	332	199	264	156	205	120	1070	943	934	818	781	684
	18	385	269	322	185	254	145	195	110	1060	918	920	793	767	660
	19	373	249	313	174	244	135	185	102	1050	895	908	770	754	638
	20	361	232	304	164	235	126	175	95.1	1040	873	896	748	742	616
	22	340	204	289	148	220	113	160	84.0	1020	832	876	706	719	575

$P_u / \phi P_n$ = **81**K/386K=**0.21**

Torsional and Constrained-Axis Flexural-Torsional Buckling Tables for Steel W-Shapes in Compression

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Collector Design

- Required flexural strength
  - o P-δ amplification
    - $P_E = \pi^2 EI / (KL_x)^2 = 2540\text{K}$
    - $B_1 = \frac{C_m}{1 - P_u/P_E} = \frac{1}{1 - 81K/2540K} = 1.03$
    - M_u = 138'K*1.03=142'K

AISC 360 Appendix 8 §8.2.1

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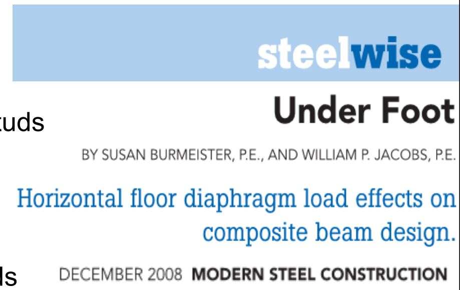
Collector Design

- Flexural strength
  - AISC Manual Table 3-19
    - *Y*2=3.5in
    - Use ΣQ_n=184K
      - 184K/11.2^{K}/$_{stud}$ =16.4 studs (each side of midpoint)
      - 32.8 studs
    - Collector studs
      - **81**K/11.2^{K}/$_{stud}$ =**7.2** studs
      - Not additive to flexure studs
  - ϕM_n = 516 kip-ft


steelwise
Under Foot
BY SUSAN BURMEISTER, P.E., AND WILLIAM P. JACOBS, P.E.
Horizontal floor diaphragm load effects on composite beam design.
DECEMBER 2008 MODERN STEEL CONSTRUCTION

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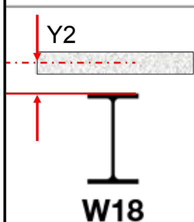

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

Shape	M_p/Ω_b kip-ft ASD	$\phi_b M_p$ kip-ft LRFD	PNA[c]	Y1[a] in.	ΣQ_n kip	Y2[b] = 2 ASD	2 LRFD	2.5 ASD	2.5 LRFD	3 ASD	3 LRFD	3.5 ASD	3.5 LRFD
W18×50	252	379	TFL	0	735	403	606	422	634	440	662	458	689
			2	0.143	628	392	590	408	613	424	637	439	660
			3	0.285	521	381	572	394	592	407	611	420	631
			4	0.428	414	368	553	378	569	389	584	399	600
			BFL	0.570	308	355	533	362	545	370	556	378	568
			6	2.08	246	345	518	351	527	357	537	363	546
			7	3.82	184								516

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90

[a] Y1 = distance from top of the steel beam to plastic neutral axis
[b] Y2 = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3c for PNA locations.

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Collector design

- Try non-composite flexural strength
  - Manual Table 3-2
  - ϕM_n = 379 kip-ft

- $P_u / \phi P_n$ = **81K**/**386**K= **0.21>**0.2
- $P_u / \phi P_n + {}^8/_9 M_u / \phi M_n$ (H1-1b)
- = (**0.21**)+ $^8/_9$(142'K)/(379'K) = **0.55** OK
  - Provide studs @ 24" (12 studs)

Collector connections

There's always a solution in steel.

structural STEEL

Collector connection

- Single-plate connection
  - ○ (4)⅞" Ø A325N bolts
    - 3" spacing
    - 1.5" edge distance top & bottom
    - 2.5" side edge distance
  - ○ ⅜" A36 plate
    - ¼" double-sided fillets

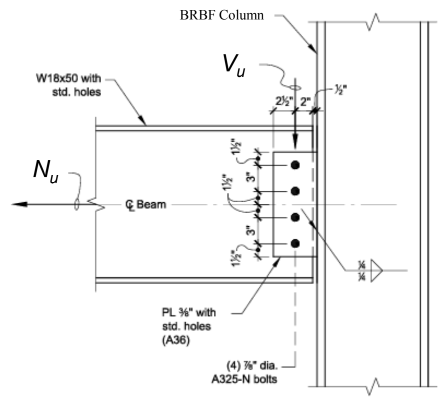
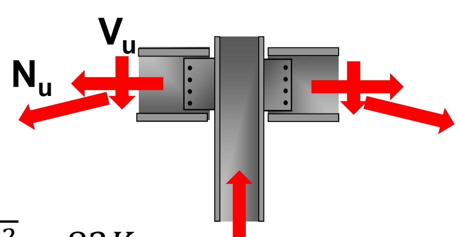


Fig. 8-6. Collector connection investigated in Example 8.4.2.

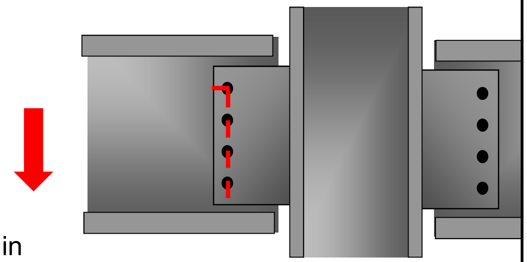
Required strength

- V_u = 13K
- N_u = 81K
- $R_u = \sqrt{V_u^2 + N_u^2}$
  - ○ $= \sqrt{(13K)^2 + (81K)^2} = 82K$
- $\theta = \tan^{-1}\frac{13K}{81k} = 9^o$



Collector connection

- Alternative approaches
  - Evaluate as a load at 9°
    - This approach shown in SDM
  - Evaluate shear and tension separately
    - SRSS interaction
    - This approach taken here



- Shear strength
  - From Table 10-10a
    - ϕV_n = 78.3K

Table 10-10a (continued)

Plate
F_y = 36 ksi

Single-Plate Connections

Bolt, Weld and Single-Plate Available Strengths, kips

7/8-in.-diameter bolts

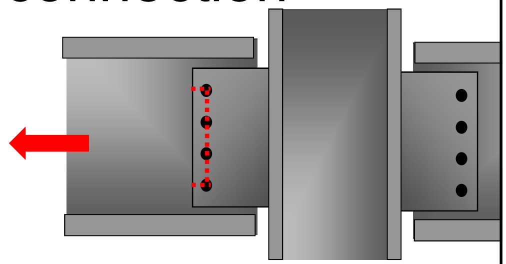
n	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
4 (L = 12)	Group A	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
			SSLT	34.8	52.2	43.5	65.3	52.2	78.3	56.5	64.8	56.5	64.8	56.5	64.8
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	—	—
			SSLT	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	71.2	107
	Group B	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	—	—
			[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	104	71.2	107
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	104	—	—
			[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	104	78.3	117

$$\frac{V_u}{\varnothing V_n} = \frac{13K}{78.3K} = 0.176$$

$$\sqrt{\left(\frac{V_u}{\varnothing V_n}\right)^2 + \left(\frac{T_u}{\varnothing T_n}\right)^2} \leq 1 \qquad \frac{T_u}{\varnothing T_n} \leq \sqrt{1 - \left(\frac{V_u}{\varnothing V_n}\right)^2} = 0.98$$

$$\varnothing T_n \geq {}^{T_u}/_{0.98} \qquad \varnothing T_n \geq {}^{81K}/_{0.98} = 82K$$

Collector connection



- Tension strength
  - Bolt (AISC 360 §J3)
    - Shear
    - Bearing
    - Tearout
  - Plate (AISC 360 §J4)
    - Yield
    - Rupture
    - Block shear
  - Weld (AISC 360 §J2)
    - Exceeds plate strength
  - Beam block shear, bearing, tearout
    - t_w(65ksi/58ksi)=
      - 0.355”(65/58)=0.39”>⅜”
    - t_w(50ksi/36ksi)=
      - 0.355”(50/36)=0.49” ”>⅜”
    - Plate governs

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Collector connection

- Plate limit states
  - Yield: $\phi F_y A$
    - = 0.9(36ksi)(⅜”)(12”)
    - = 146K
  - Rupture: $\phi F_u A_e = F_u A_n$
    - = 0.75(58ksi)(⅜”)(12”-4”)
    - = 130K
  - Block shear $\phi R_n \leq$
    - $\phi(0.6F_u A_{nv} + U_{BS} F_u A_{nt})$
      - 0.75(⅜”)(0.6*58ksi*2*2”+ 1.0*58ksi*6”)= 137K
    - $\phi(0.6F_y A_{gv} + U_{BS} F_u A_{nt})$
      - 0.75(⅜”)(0.6*50ksi*2*2.5” +1.0*58ksi*6”)= 140K

AISC 360 §J4

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Collector connection

- Bolt limit states
  - Shear: Table 7-1
    - 4*24.3K = 97.2K
  - Bearing (spacing)
    - Table 7-4
    - 4(⅜")91.4K/in = 137K
  - Bearing (edge distance)
    - Table 7-5
    - 4(⅜") 79.9/in = 120K
- Governing strength:
  - ϕR_n =97.2K
  - $R_u / \phi R_n$ =81K/97K=0.84

$$\sqrt{\left(\frac{V_u}{\emptyset V_n}\right)^2 + \left(\frac{T_u}{\emptyset T_n}\right)^2} = \sqrt{(0.18)^2 + (0.84)^2} = 0.85 \quad OK$$

AISC 360 §J3

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Diaphragm openings

There's always a solution in steel.

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Diaphragm openings

- Local shears
- Local collector forces
- Local chord forces

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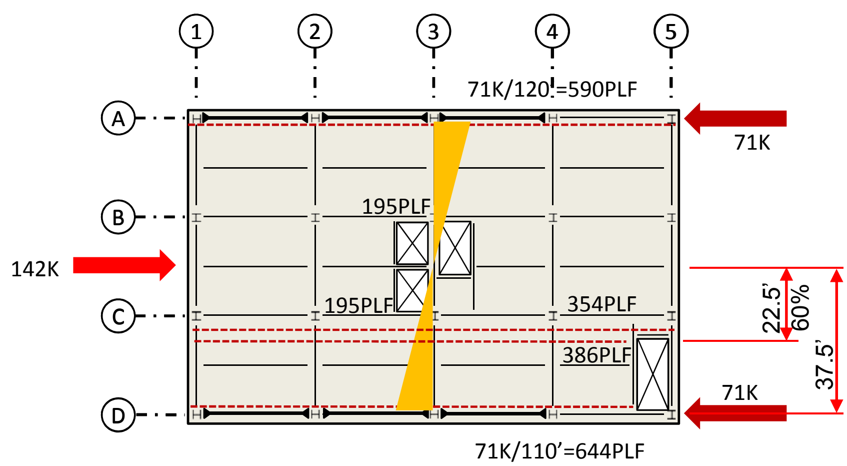
Diaphragm analysis: Roof X



Does not include B_2

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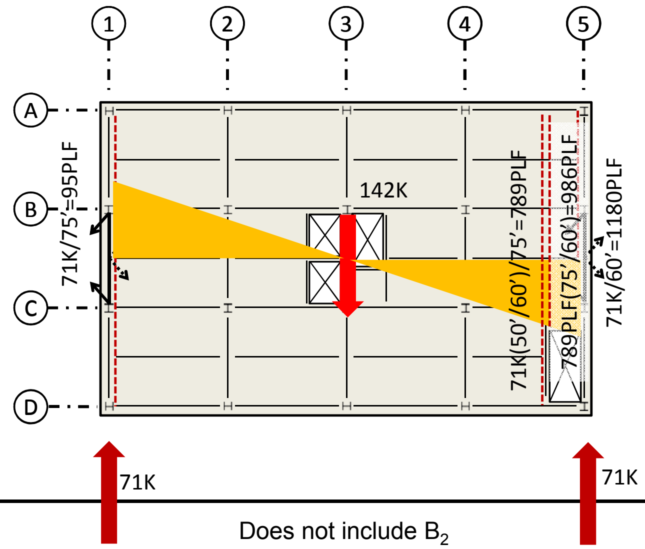
Diaphragm analysis: Roof Y



Does not include B_2

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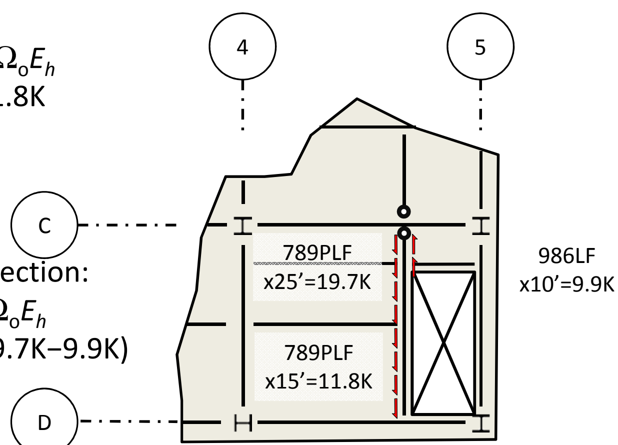
Local collector forces: Y

Collector:
Design for $B_2\Omega_o E_h$
= 1.02*2.5*11.8K
=30.2K

Collector connection:
Design for $B_2\Omega_o E_h$
= 1.02*2.5*(19.7K–9.9K)
=25.0K



Does not include B_2

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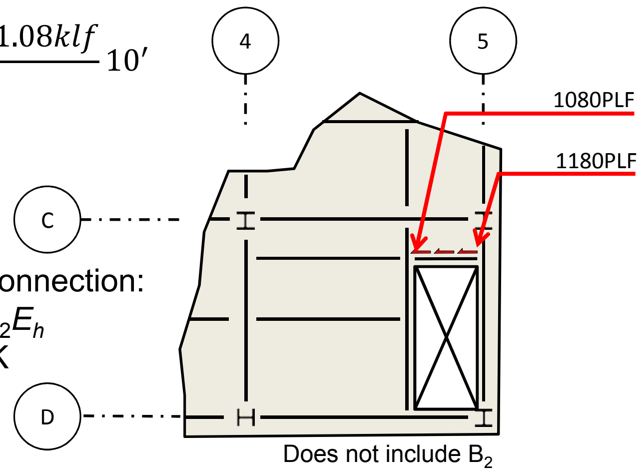
Local chord forces: Y

$$\frac{1.18klf + 1.08klf}{2} 10'$$

=11.3K



Chord and connection:
Design for $B_2 E_h$
= 1.02*11.3K
= 11.5K

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Summary

There's always a solution in steel.

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Summary

- Simple design methods presented
- Methods of accounting for second-order effects presented
- Forces generated for design of SMF & BRBF
- Diaphragm forces generated
- Roof diaphragm analyzed
- Deck designed
- Example collector designed
- Example collector connection designed

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There's always a solution in steel.

End of session 6

Next:

Design of the Moment frames

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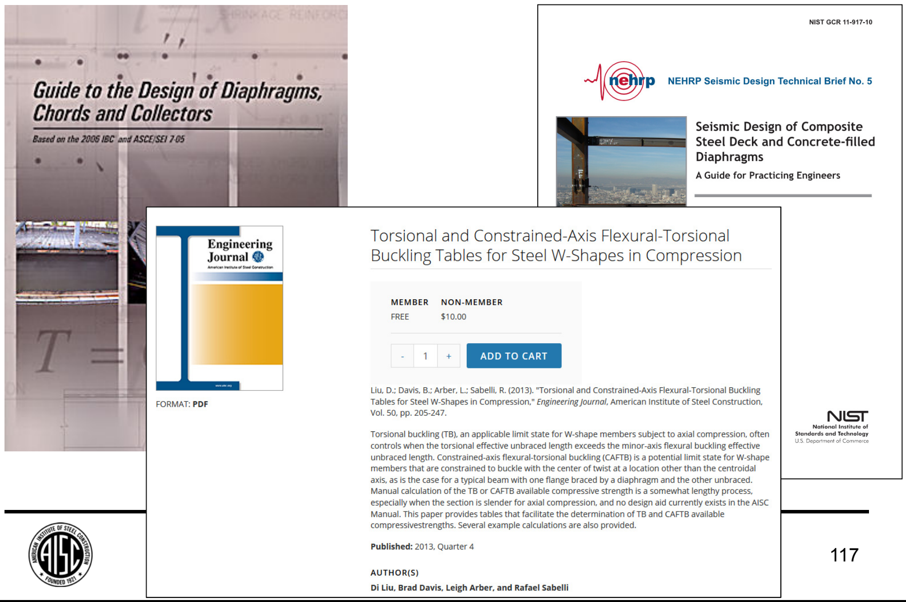
Additional resources



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Question time

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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!

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

- New reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

8-Session Registrants

CEU/PDH Certificates

One certificate will be issued at the conclusion of all 8 sessions.

8-Session Registrants

Access to the quiz: Information for accessing the quiz will be emailed to you by Wednesday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG

Quiz and Attendance records: Posted Tuesday mornings. www.aisc.org/nightschool - click on Current Course Details.

Reasons for quiz:

- EEU – must take all quizzes and final to receive EEU
- CEUs/PDHS – If you watch a recorded session you must take quiz for CEUs/PDHs.
- REINFORCEMENT – Reinforce what you learned tonight. Get more out of the course.

NOTE: If you attend the live presentation, you do not have to take the quizzes to receive CEUs/PDHs.

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Find all your handouts, quizzes and quiz scores, recording access, and attendance information all in one place!

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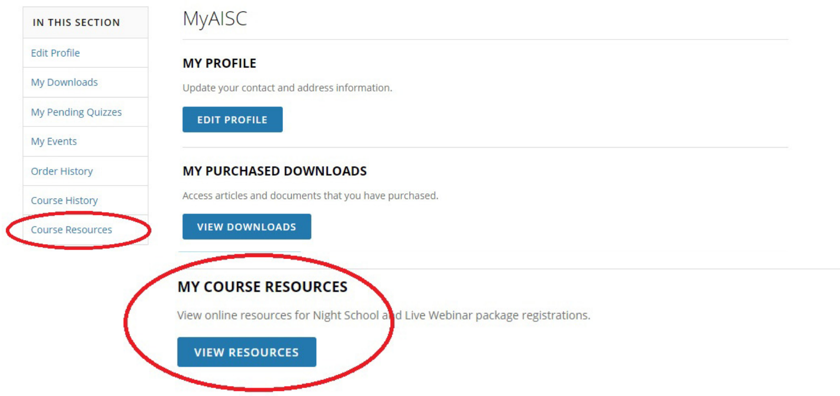

There's always a solution in steel.

Thank You

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