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Course Description

Session L2: Building Analysis and Diaphragm Design

September 17, 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.

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-d and P-D effects.
- Describe the components of designing members at diaphragm openings.

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Seismic Design in Steel: Concepts and Examples

Session L2: Building Analysis and Diaphragm Design
September 17, 2018

Rafael Sabelli, SE

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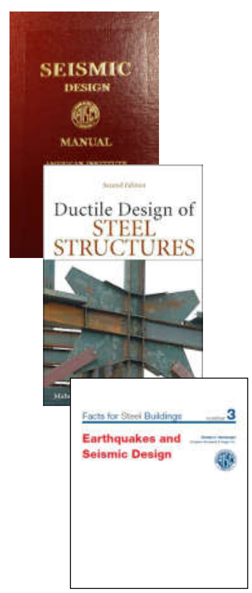
Course objectives

- 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.

9

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



10

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

R1. Introduction to effective seismic design

R2. Seismic design of moment frames

R3. Seismic design of braced frames

R4. Seismic design of buildings

Course outline

Part II: Application

L1. Planning the seismic design

L2. <u>Building analysis and diaphragm design</u>

L3. Design of the moment frames

L4. Design of the braced frames

13

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Session L2: 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

15

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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

17

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

18

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

19

Load cases

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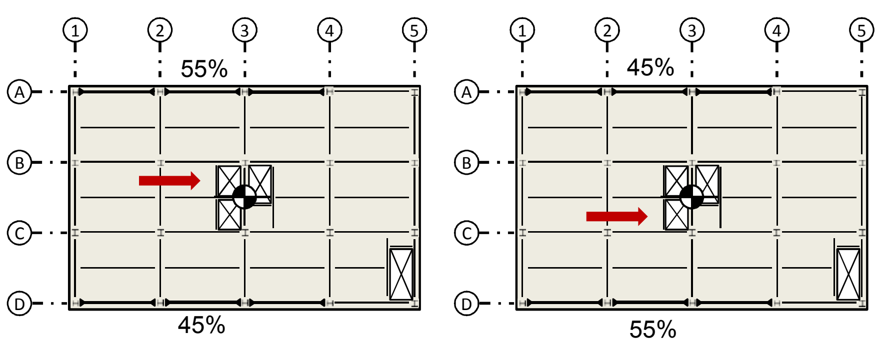
Load Cases

- 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

21

Load Cases



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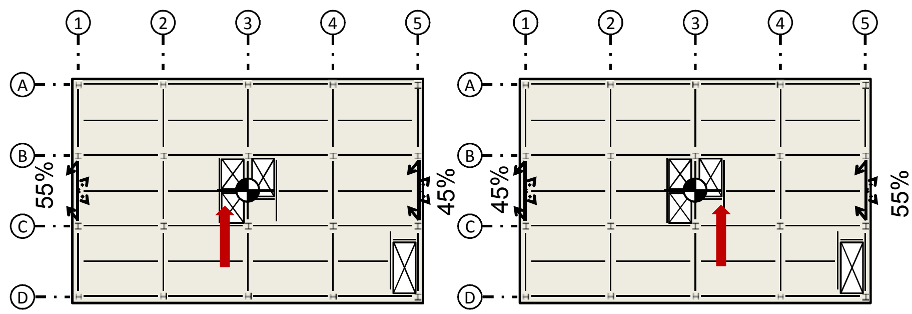
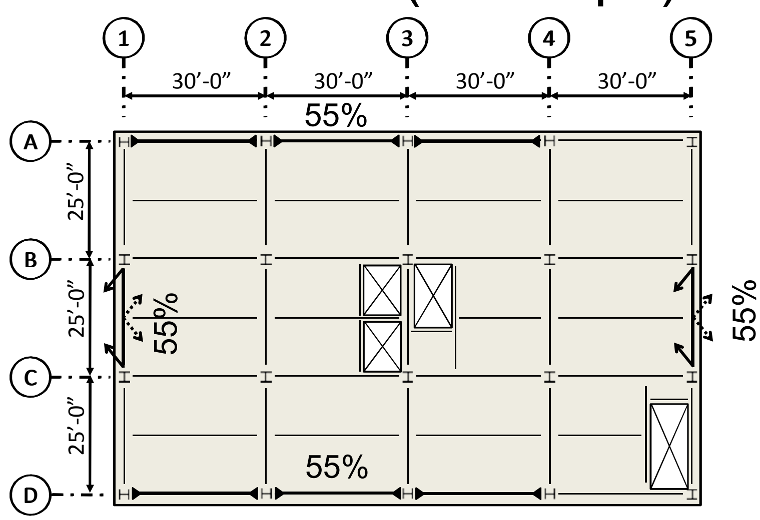
Load Cases



Load Cases (envelope)



Base Shear

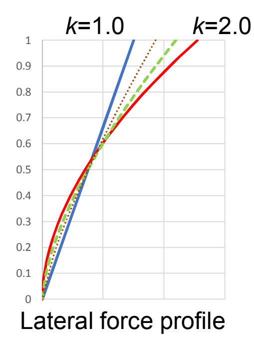
- From Session 5
  - SMF
    - V = 0.0818*3313K
      = 271K
  - BRBF
    - V = 0.100*3313K
      = 331K

25

Vertical Distribution

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

- T≤0.5s k=1.0
- T≥2.5s k=2.0
- 0.5s<T<2.5s interpolate



ASCE 7 §12.8.3

26

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

27

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

28

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Structural model

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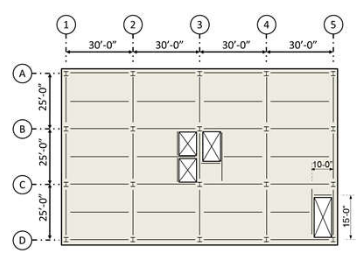
Structural Model

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



ASCE 7 §12.3.1

30

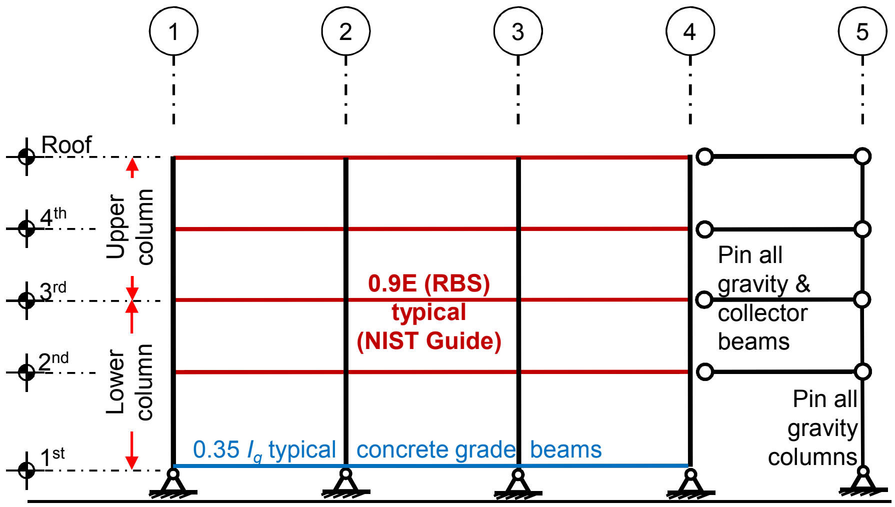
Structural Model

- 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

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

Moment-frames



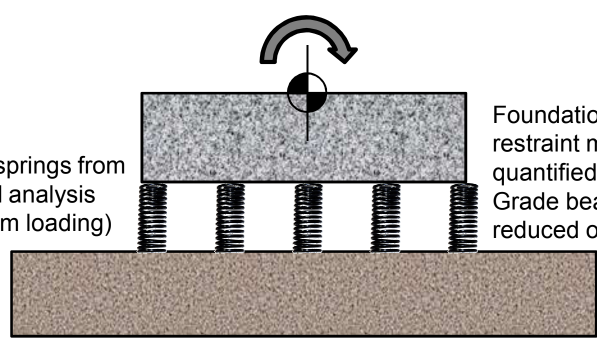
Base plate and foundation rotational restraint may be quantified for additional stiffness

33

Moment-frames

Foundation rotational restraint

Foundation springs from geotechnical analysis (for short-term loading)



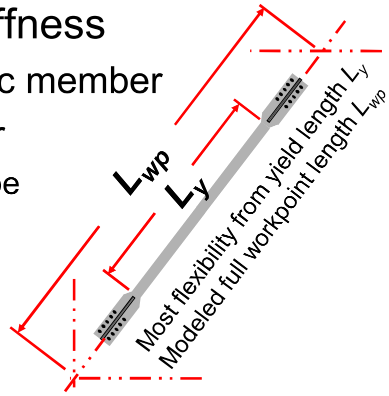
Foundation rotational restraint may be quantified footing sizing. Grade beam may be reduced or eliminated.

ASCE 7 §12.13.3

34

Braced frames

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



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

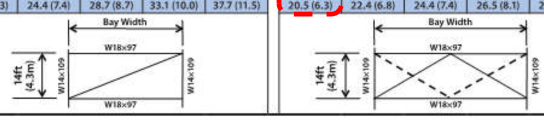
$$1.2 \le K_F \le 1.8$$

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.

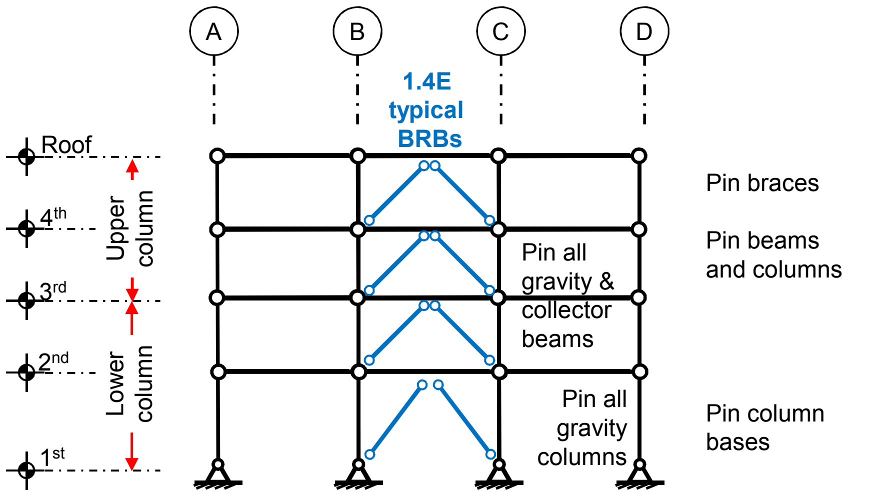
STORY HEIGHT: 14ft (4.3m)

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)



Brochure from Core Brace

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

39

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

40

Design for stability AISC 360

- First-Order Analysis Method (FOAM)
  - Analysis
    - 2^{nd}-order effects not included
    - Uses same model for strength, drift, period
  - 2^{nd}-order effects addressed by additional lateral load
    - $N_i = 2.1 \frac{\Delta}{h} Y_i \approx 0.008 Y_i$ 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 **≤ 0.5**

AISC 360 Appendix 7 §7.3

41

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 2^{nd} order analysis
  - Or use B_2

AISC 360 Appendix 7

42

Design for stability: Moment Frames

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

AISC 360 Appendix 7

43

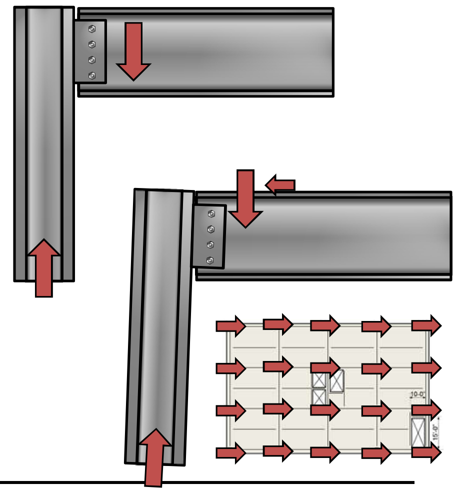
Second-order effects

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

44

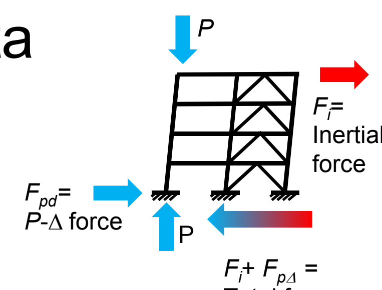
Second-order effects (P-Delta)

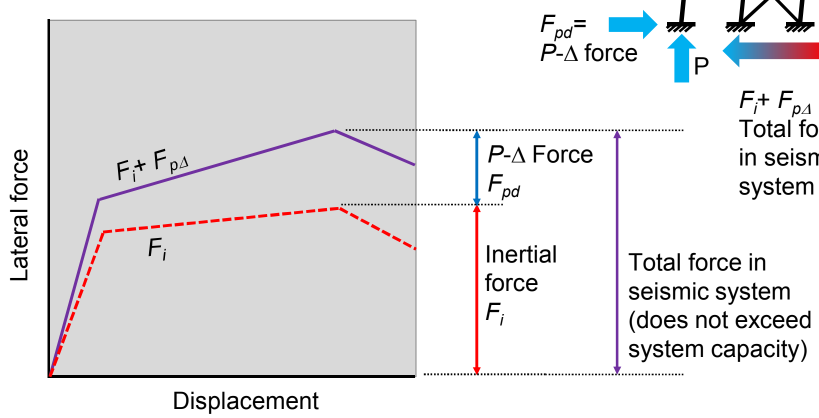
- Columns resist the story gravity force
- Lateral loads induce drift
  - Columns slope
  - Column axial force has horizontal component
  - Horizontal component is additional thrust on diaphragm



45

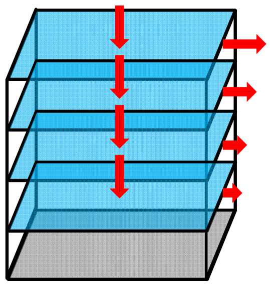
P-Delta





46

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

47

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

48

Second Order Effects: BRBF

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

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

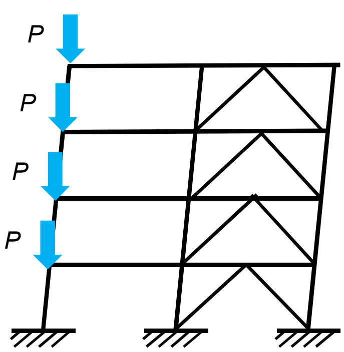
	B_2
Roof	1.02
4th	1.03
3rd	1.03
2nd	1.04

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

AISC 360 Appendix 8

49

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

50

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

  - $\theta \sim 1-1/B_2 = 0.04$ (max)

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

  - OK

ASCE 7 §12.8.7

51

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

52

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

53

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Analysis forces

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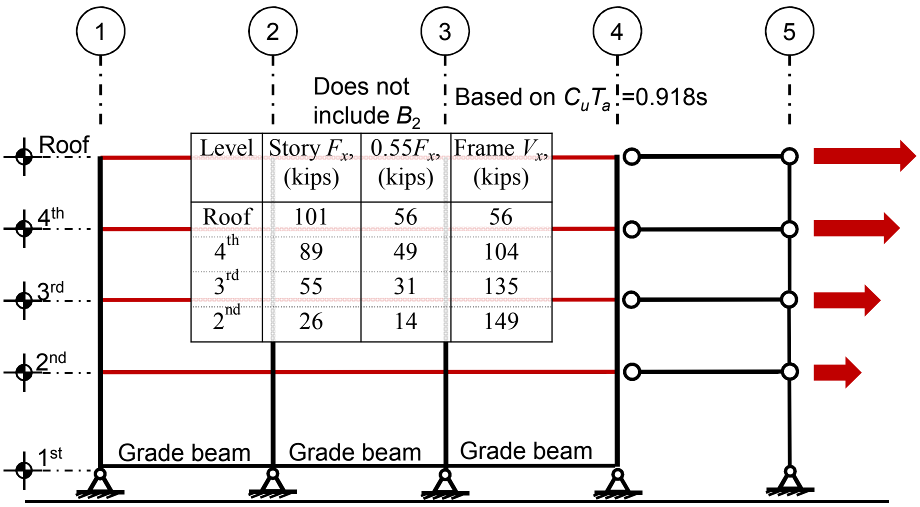
Moment-frames



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

55

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

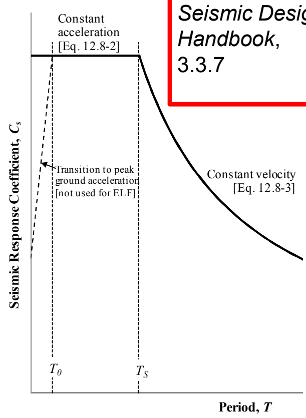
56

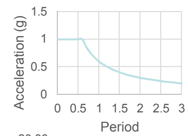
Drift-determined period

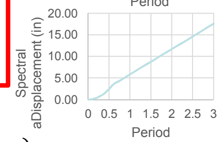
Adapted from Naeim's *Seismic Design Handbook*, 3.3.7

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

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







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

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

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

⅔ is an approximate correction factor

- Spectral displacement is not roof displacement
- 1st-mode mass participation <100%

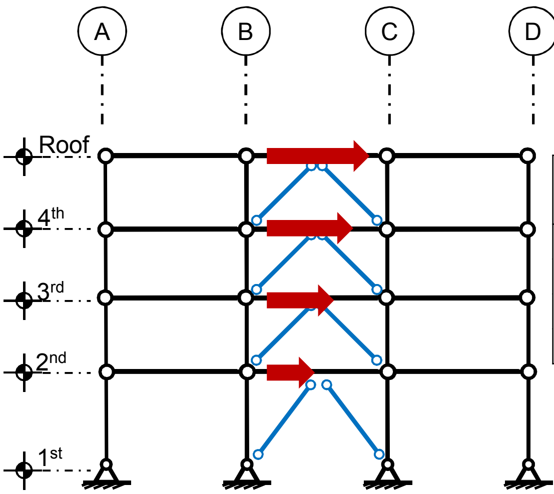
57

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

58

Braced frames



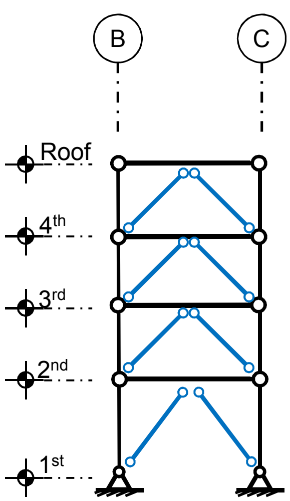
Level	Story F_x, (kips)	$0.55F_x$, (kips)	Frame V_x, (kips)
Roof	120	66	66
4th	108	59	125
3rd	70	38	164
2nd	34	19	182

Does not include ρ=1.3
Does not include B_2

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

59

Braced-frame forces

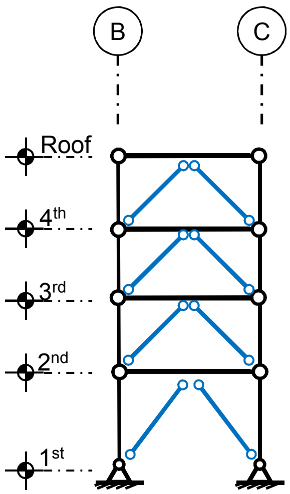


Level	Frame F_x, (kips)	B_2	ρ	Frame $\rho B_2 F_x$, (kips)	Frame $\rho B_2 V_x$, (kips)
Roof	66.0	1.02	1.30	87.6	88
4th	59.2	1.03	1.30	79.3	167
3rd	38.3	1.03	1.30	51.3	218
2nd	18.7	1.04	1.30	25.3	243

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

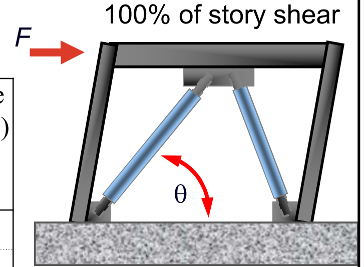
60

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



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

61

Diaphragm Design

There's always a solution in steel.

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

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

There's always a solution in steel.

Diaphragm analysis

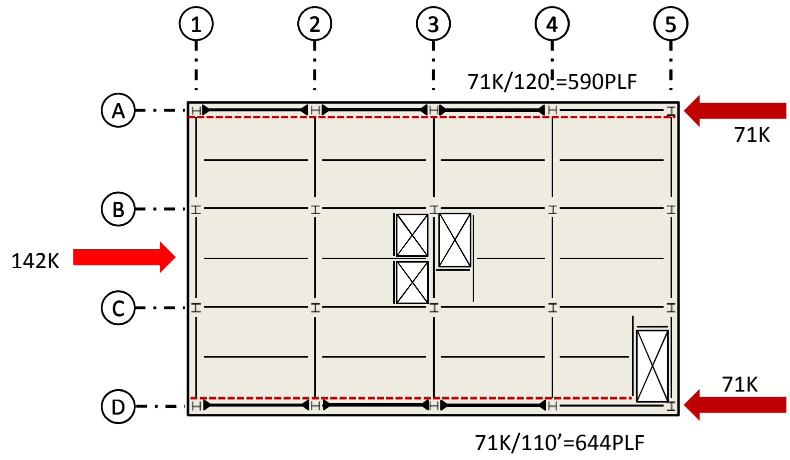
structural
STEEL

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

66

Diaphragm analysis: Roof X

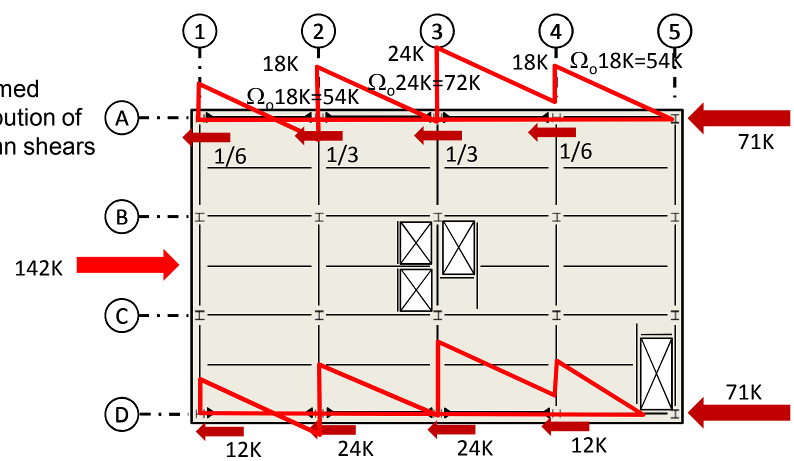


Does not include B_2

67

Diaphragm analysis: Roof X

Assumed distribution of column shears

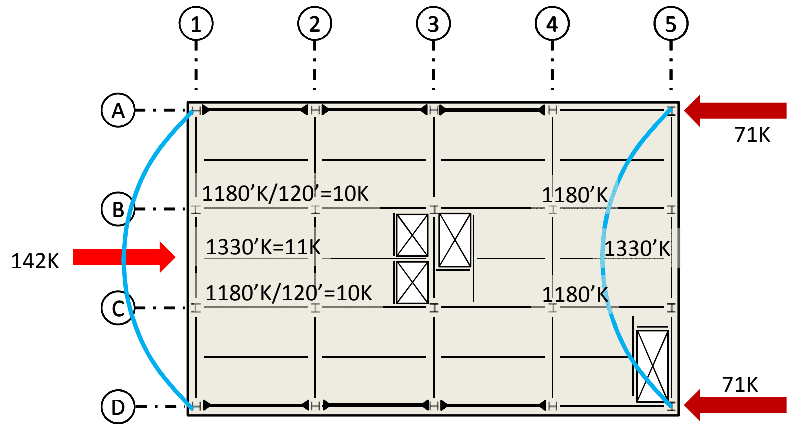


Ω_o= 3.0 for SMF

Does not include B_2

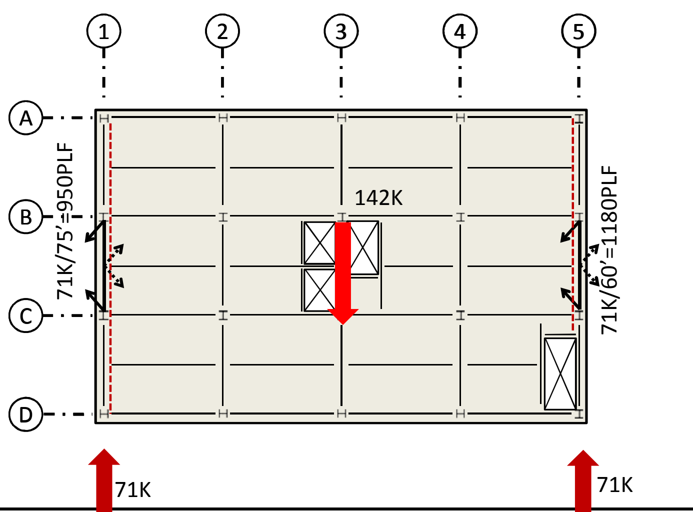
68

Diaphragm analysis: Roof X



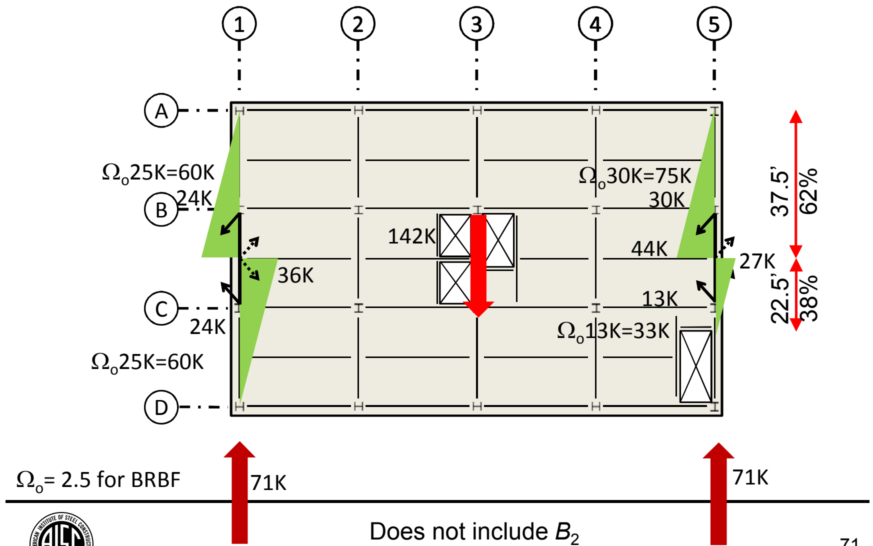
Does not include B_2

Diaphragm analysis: Roof Y



Does not include B_2

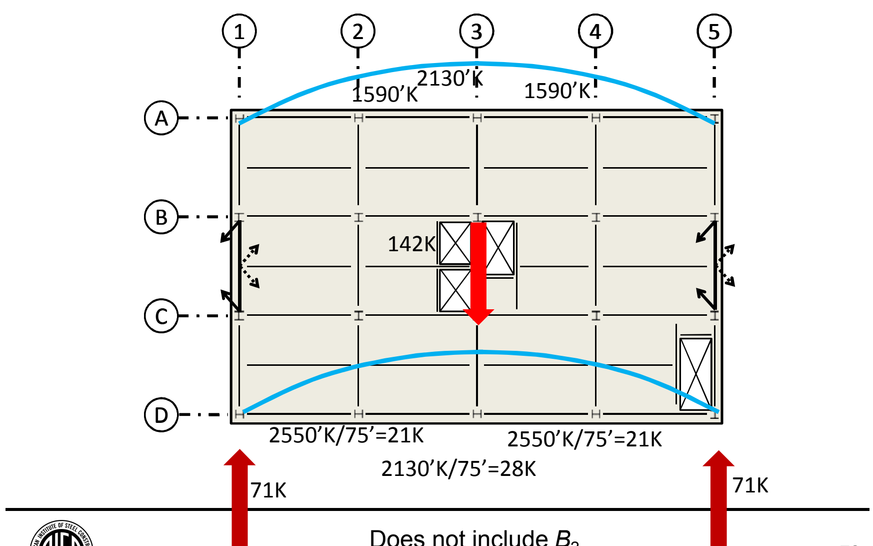
Diaphragm analysis: Roof Y



Does not include B_2

71

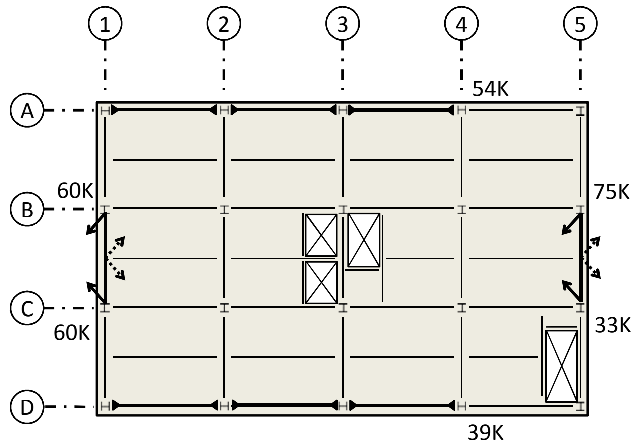
Diaphragm analysis: Roof Y



Does not include B_2

72

Chord/collector forces



Does not include B_2

73

Capacity-design forces

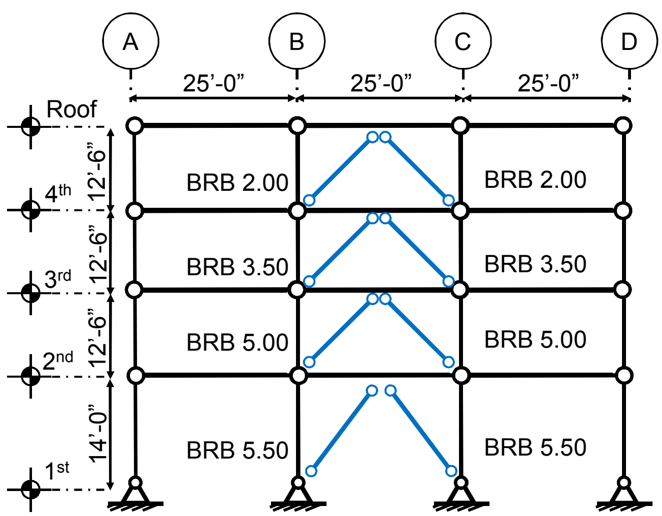
There's always a solution in steel.

structural
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

Brace sizes



Design of BRBF presented in Session 8

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)*cosθ = 179K
  - $< B_2 \Omega_o V_{frame}$ = 1.02*2.5*66K = 168k
- Use capacity forces for roof collectors

AISC 341 §F4.2a

77

Diaphragm design

There's always a solution in steel.

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

Deck selection

- Design composite deck
  - v_c $= 2\lambda d \sqrt{f'_c}$
    $= (2)0.75(3.25")(4000)^{1/2}$ (12"/ft)
    = 3700 plf
  - v_s $= A_s f_y$
    $= 0.11in^2$/ft * 60 ksi
    = 6600 plf
  - ϕV_n $= \phi (v_c + v_s)$
    = 0.75*(3700plf+6600plf)
    = 7725 plf > 1200 plf

ACI 318 §11.2

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

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

AISC 360 §I8.2a

82

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

83

Collector design

There's always a solution in steel.

structural STEEL

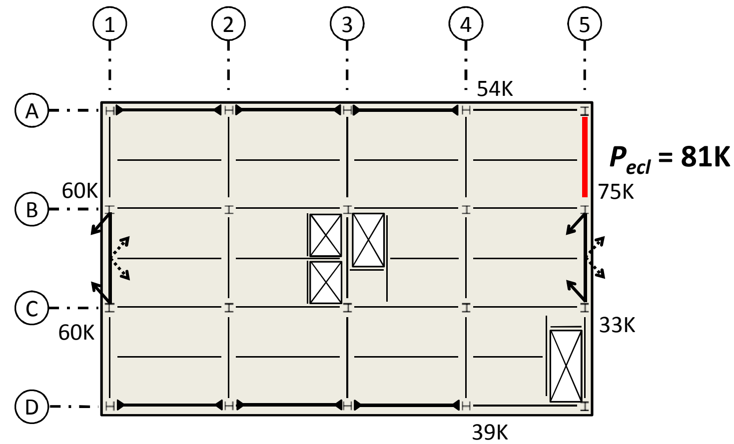
Collector design

- Combined flexure and axial
  - 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
  - Compression path through deck typically neglected

Collector design

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

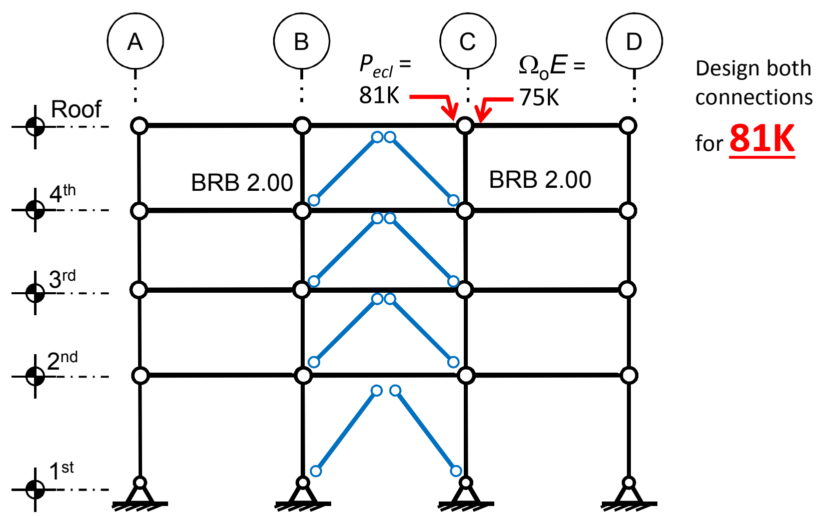
Collector design



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

87

Collector forces



Design both connections for **81K**

Design of BRBF presented in Session 8

88

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)

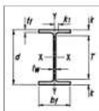


Table 1-1 (continued) W-Shapes Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	k k_{det}	k_1	T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W21×93	27.3	21.6	21 5/8	0.580	9/16	5/16	8.42	8 3/8	0.930	15/16	1.43	1 5/8	15/16	18 3/8	5 1/2
×83[c]	24.4	21.4	21 3/8	0.515	1/2	1/4	8.36	8 3/8	0.835	13/16	1.34	1 1/2	7/8		
×73[c]	21.5	21.2	21 1/4	0.455	7/16	1/4	8.30	8 1/4	0.740	3/4	1.24	1 7/16	7/8		
×68[c]	20.0	21.1	21 1/8	0.430	7/16	1/4	8.27	8 1/4	0.685	11/16	1.19	1 3/8	7/8		
×62[c]	18.3	21.0	21	0.400	3/8	3/16	8.24	8 1/4	0.615	5/8	1.12	1 5/16	13/16		
×55[c]	16.2	20.8	20 3/4	0.375	3/8	3/16	8.22	8 1/4	0.522	1/2	1.02	1 3/16	13/16		
×48[c,f]	14.1	20.6	20 5/8	0.350	3/8	3/16	8.14	8 1/8	0.430	7/16	0.930	1 1/8	13/16		
W21×57[c]	16.7	21.1	21	0.405	3/8	3/16	6.56	6 1/2	0.650	5/8	1.15	1 5/16	13/16	18 3/8	3 1/2
×50[c]	14.7	20.8	20 7/8	0.380	3/8	3/16	6.53	6 1/2	0.535	9/16	1.04	1 1/4	13/16		
×44[c]	13.0	20.7	20 5/8	0.350	3/8	3/16	6.50	6 1/2	0.450	7/16	0.950	1 1/8	13/16		
W18×311[h]	91.6	22.3	22 3/8	1.52	1 1/2	3/4	12.0	12	2.74	2 3/4	3.24	3 7/16	1 3/8	15 1/2	5 1/2
×283[h]	83.3	21.9	21 7/8	1.40	1 3/8	11/16	11.9	11 7/8	2.50	2 1/2	3.00	3 3/16	1 5/16		
×258[h]	76.0	21.5	21 1/2	1.28	1 1/4	5/8	11.8	11 3/4	2.30	2 5/16	2.70	3	1 1/4		
×234[h]	68.6	21.1	21	1.16	1 3/16	5/8	11.7	11 5/8	2.11	2 1/8	2.51	2 3/4	1 3/16		
×211	62.3	20.7	20 5/8	1.06	1 1/16	9/16	11.6	11 1/2	1.91	1 15/16	2.31	2 9/16	1 3/16		
×192	56.2	20.4	20 3/8	0.960	15/16	1/2	11.5	11 1/2	1.75	1 3/4	2.15	2 7/16	1 1/8		
×175	51.4	20.0	20	0.890	7/8	7/16	11.4	11 3/8	1.59	1 9/16	1.99	2 7/16	1 1/4	15 1/8	
×158	46.3	19.7	19 3/4	0.810	13/16	7/16	11.3	11 1/4	1.44	1 7/16	1.84	2 3/8	1 1/4		
×143	42.0	19.5	19 1/2	0.730	3/4	3/8	11.2	11 1/4	1.32	1 5/16	1.72	2 3/16	1 3/16		
×130	38.3	19.3	19 1/4	0.670	11/16	3/8	11.2	11 1/8	1.20	1 3/16	1.60	2 1/16	1 3/16		
×119	35.1	19.0	19	0.655	5/8	5/16	11.3	11 1/4	1.06	1 1/16	1.46	1 15/16	1 3/16		
×106	31.1	18.7	18 3/4	0.590	9/16	5/16	11.2	11 1/4	0.940	15/16	1.34	1 13/16	1 1/8		
×97	28.5	18.6	18 5/8	0.535	9/16	5/16	11.1	11 1/8	0.870	7/8	1.27	1 3/4	1 1/8		
×86	25.3	18.4	18 3/8	0.480	1/2	1/4	11.1	11 1/8	0.770	3/4	1.17	1 5/8	1 1/16		
×76[c]	22.3	18.2	18 1/4	0.425	7/16	1/4	11.0	11	0.680	11/16	1.08	1 9/16	1 1/16		
W18×71	20.9	18.5	18 1/2	0.495	1/2	1/4	7.64	7 5/8	0.810	13/16	1.21	1 1/2	7/8	15 1/2	3 1/2[g]
×65	19.1	18.4	18 3/8	0.450	7/16	1/4	7.59	7 5/8	0.750	3/4	1.15	1 7/16	7/8		
×60[c]	17.6	18.2	18 1/4	0.415	7/16	1/4	7.56	7 1/2	0.695	11/16	1.10	1 3/8	13/16		
×55[c]	16.2	18.1	18 1/8	0.390	3/8	3/16	7.53	7 1/2	0.630	5/8	1.03	1 5/16	13/16		
×50[c]	14.7	18.0	18	0.355	3/8	3/16	7.50	7 1/2	0.570	9/16	0.972	1 1/4	13/16		
W18×46[c]	13.5	18.1	18	0.360	3/8	3/16	6.06	6	0.605	5/8	1.01	1 1/4	13/16	15 1/2	3 1/2[g]
×40[c]	11.8	17.9	17 7/8	0.315	5/16	3/16	6.02	6	0.525	1/2	0.927	1 3/16	13/16		
×35[c]	10.3	17.7	17 3/4	0.300	5/16	3/16	6.00	6	0.425	7/16	0.827	1 1/8	3/4		

[c] Shape is slender for compression with F_y = 50 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 1-1 (continued) W-Shapes Properties

W21-W18

Nom-inal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	Axis X-X I	S	r	Z	Axis Y-Y I	S	r	Z	r_{ts}	h_o	Torsional Properties J	C_w
lb/ft			in.⁴	in.³	in.	in.³	in.⁴	in.³	in.	in.³	in.	in.	in.⁴	in.⁶
93	4.53	32.3	2070	192	8.70	221	92.9	22.1	1.84	34.7	2.24	20.7	6.03	9940
83	5.00	36.4	1830	171	8.67	196	81.4	19.5	1.83	30.5	2.21	20.6	4.34	8630
73	5.60	41.2	1600	151	8.64	172	70.6	17.0	1.81	26.6	2.19	20.5	3.02	7410
68	6.04	43.6	1480	140	8.60	160	64.7	15.7	1.80	24.4	2.17	20.4	2.45	6760
62	6.70	46.9	1330	127	8.54	144	57.5	14.0	1.77	21.7	2.15	20.4	1.83	5960
55	7.87	50.0	1140	110	8.40	126	48.4	11.8	1.73	18.4	2.11	20.3	1.24	4980
48	9.47	53.6	959	93.0	8.24	107	38.7	9.52	1.66	14.9	2.05	20.2	0.803	3950
57	5.04	46.3	1170	111	8.36	129	30.6	9.35	1.35	14.8	1.68	20.5	1.77	3190
50	6.10	49.4	984	94.5	8.18	110	24.9	7.64	1.30	12.2	1.64	20.3	1.14	2570
44	7.22	53.6	843	81.6	8.06	95.4	20.7	6.37	1.26	10.2	1.60	20.3	0.770	2110
311	2.19	10.4	6970	624	8.72	754	795	132	2.95	207	3.53	19.6	176	76200
283	2.38	11.3	6170	565	8.61	676	704	118	2.91	185	3.47	19.4	134	65900
258	2.56	12.5	5510	514	8.53	611	628	107	2.88	166	3.42	19.2	103	57600
234	2.76	13.8	4900	466	8.44	549	558	95.8	2.85	149	3.37	19.0	78.7	50100
211	3.02	15.1	4330	419	8.35	490	493	85.3	2.82	132	3.32	18.8	58.6	43400
192	3.27	16.7	3870	380	8.28	442	440	76.8	2.79	119	3.28	18.7	44.7	38000
175	3.58	18.0	3450	344	8.20	398	391	68.8	2.76	106	3.24	18.4	33.8	33300
158	3.92	19.8	3060	310	8.12	356	347	61.4	2.74	94.8	3.20	18.3	25.2	29000
143	4.25	22.0	2750	282	8.09	322	311	55.5	2.72	85.4	3.17	18.2	19.2	25700
130	4.65	23.9	2460	256	8.03	290	278	49.9	2.70	76.7	3.13	18.1	14.5	22700
119	5.31	24.5	2190	231	7.90	262	253	44.9	2.69	69.1	3.13	17.9	10.6	20300
106	5.96	27.2	1910	204	7.84	230	220	39.4	2.66	60.5	3.10	17.8	7.48	17400
97	6.41	30.0	1750	188	7.82	211	201	36.1	2.65	55.3	3.08	17.7	5.86	15800
86	7.20	33.4	1530	166	7.77	186	175	31.6	2.63	48.4	3.05	17.6	4.10	13600
76	8.11	37.8	1330	146	7.73	163	152	27.6	2.61	42.2	3.02	17.5	2.83	11700
71	4.71	32.4	1170	127	7.50	146	60.3	15.8	1.70	24.7	2.05	17.7	3.49	4700
65	5.06	35.7	1070	117	7.49	133	54.8	14.4	1.69	22.5	2.03	17.7	2.73	4240
60	5.44	38.7	984	108	7.47	123	50.1	13.3	1.68	20.6	2.02	17.5	2.17	3850
55	5.98	41.1	890	98.3	7.41	112	44.9	11.9	1.67	18.5	2.00	17.5	1.66	3430
50	6.57	45.2	800	88.9	7.38	101	40.1	10.7	1.65	16.6	1.98	17.4	1.24	3040
46	5.01	44.6	712	78.8	7.25	90.7	22.5	7.43	1.29	11.7	1.58	17.5	1.22	1720
40	5.73	50.9	612	68.4	7.21	78.4	19.1	6.35	1.27	10.0	1.56	17.4	0.810	1440
35	7.06	53.5	510	57.6	7.04	66.5	15.3	5.12	1.22	8.06	1.51	17.3	0.506	1140

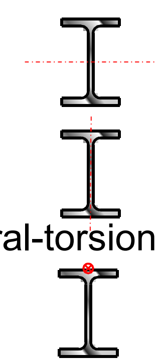
Collector design

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

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

Collector design

- Compressive Strength
  - Major axis
    - $(KL)_x$ = 25'-0"
  - Minor axis
    - $(KL)_y$ = 0'-0"
  - 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

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

Collector design

F_y = 50 ksi

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

TB CAFTB

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

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

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

95

Collector Design

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

AISC 360 Appendix 8 §8.2.1

96

Collector Design

- Flexural strength
  - AISC Manual Table 3-19
    - $Y2$=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**

97

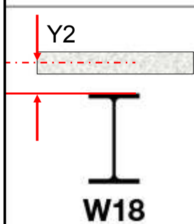


Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	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	329	495	334	502	339	509	343	516

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[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.

98

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)

99

There's always a solution in steel.

Collector connections

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

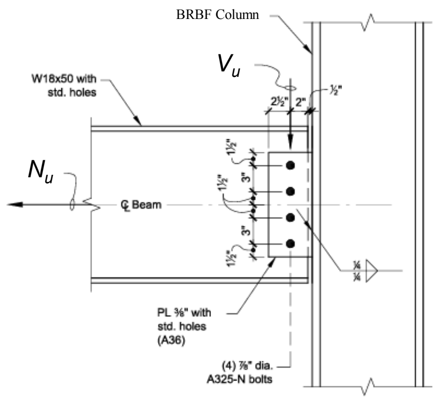
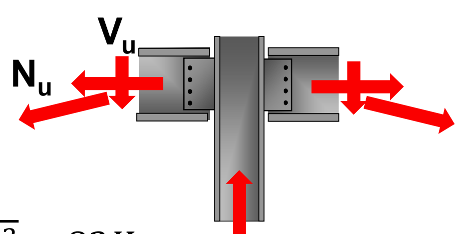


Fig. 8-6. Collector connection investigated in Example 8.4.2.

101

Required strength

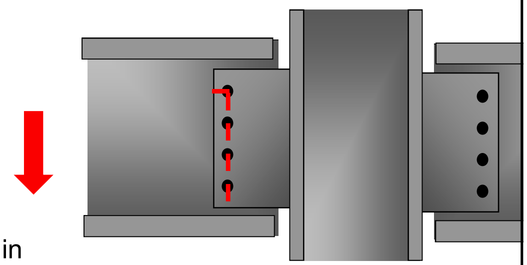
- V_u = 13K
- N_u = 81K
- $R_u = \sqrt{V_u^2 + N_u^2}$
  - $= \sqrt{(13K)^2 + (81K)^2} = 82K$
- $\theta = \tan^{-1}\frac{13K}{81k} = 9^o$



102

Collector connection

- Alternative approaches
  - Evaluate as a load at 9^o
    - 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

103

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	Plate Thickness, in. 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	[illegible]	[illegible]	[illegible]	[illegible]	84.8
		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

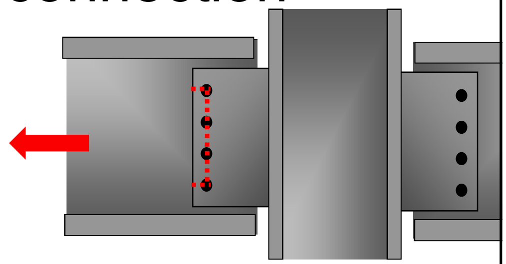
$$\frac{V_u}{\emptyset V_n} = \frac{13K}{78.3K} = 0.176$$

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

$$\emptyset T_n \ge {}^{T_u}/_{0.98} \qquad \emptyset T_n \ge {}^{81K}/_{0.98} = 82K$$

104

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

105

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 $\phi R_n \leq$
  - $\phi(0.6F_uA_{nv} + U_{BS}F_uA_{nt})$
    - 0.75(⅜”)(0.6*58ksi*2*2”+ 1.0*58ksi*6”)= 137K
  - $\phi(0.6F_yA_{gv} + U_{BS}F_uA_{nt})$
    - 0.75(⅜”)(0.6*50ksi*2*2.5” +1.0*58ksi*6”)= 140K

AISC 360 §J4

106

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

107

Diaphragm openings

There's always a solution in steel.

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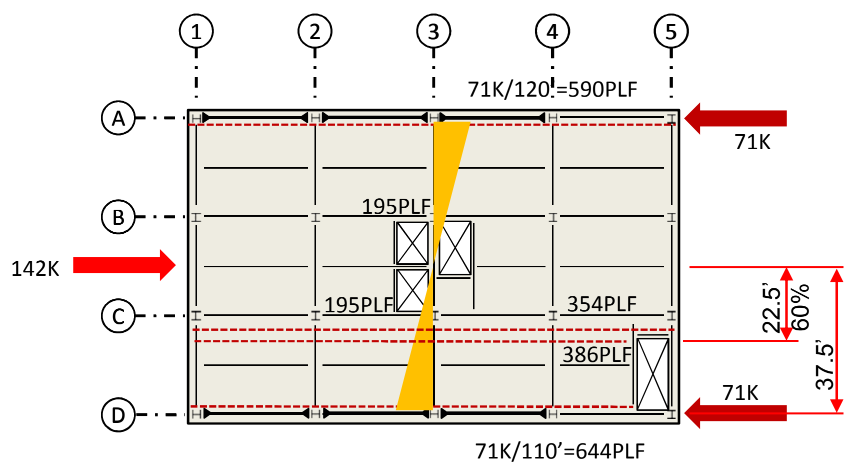
Diaphragm openings

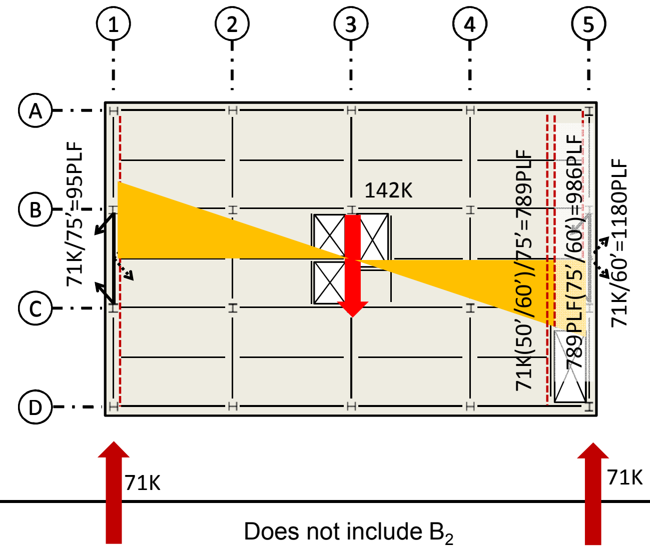
- Local shears
- Local collector forces
- Local chord forces

Diaphragm analysis: Roof X



Does not include B_2

Diaphragm analysis: Roof Y



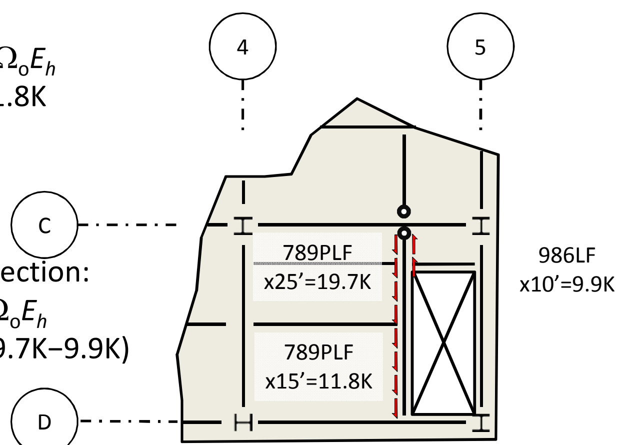
Does not include B_2

111

Local collector forces: Y

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

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



Does not include B_2

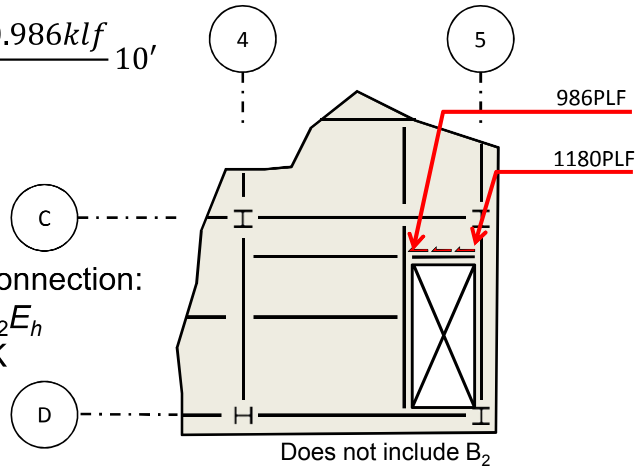
112

Local chord forces: Y

$$\frac{1.18klf + 0.986klf}{2}10'$$

=10.8K

Chord and connection:
Design for B_2E_h
= 1.02*10.8K
= 11.0K



There's always a solution in steel.

Summary

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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

115

There's always a solution in steel.

End of session L2

Next:

Design of the Moment frames

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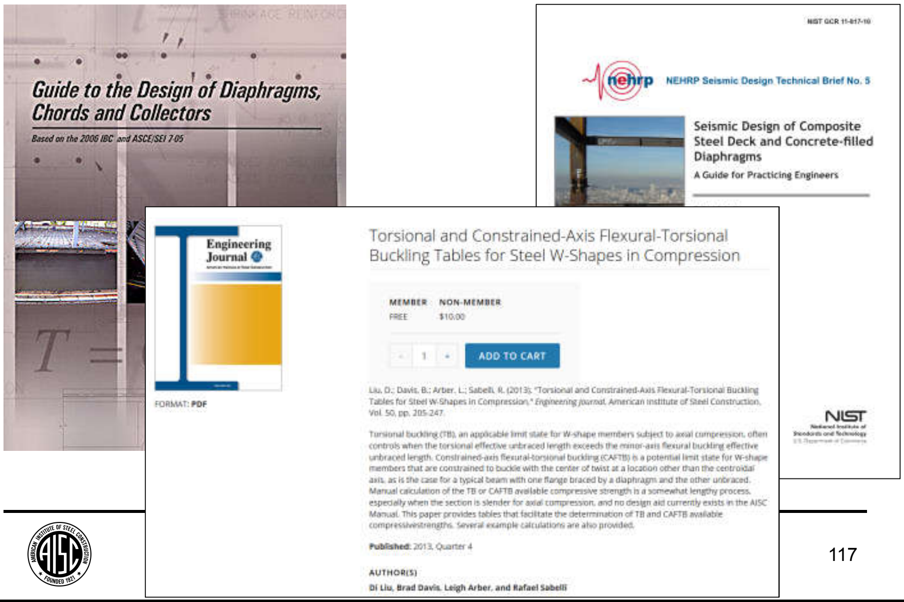
Additional resources



117

Question time

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- 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
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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 Package Registrants CEU/PDH Certificates

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

8-Session Package Registrants CEU/PDH Certificates

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- For Sessions R1 – R4, access has been available since July 16.
- For Sessions L1 – L4, find access within two days after the live air date. (An email will be sent from webinars@aisc.org.)
- All video recordings are available through October 22.
- All quizzes are due October 22.
- A final exam will also be given. It will be available October 5 and due October 22.
- Quiz scores are displayed in the Course Resources table.

8-Session Package Registrants
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Attendance and PDH Certificates

- For Sessions R1 – R4, you must pass the quiz to receive credit for the session.
- For Sessions L1 – L4, you have two options to receive credit for the session.
  - Option 1: Watch the session live. Credit for live attendance will be displayed in the Course Resources table within two days of the session.
  - Option 2: Watch the recording and pass the quiz.

EEU Certificates – Certificate of Completion

- In addition to PDH certificates earned for each individual session, an EEU (Equivalent Education Unit) certificate of completion will be issued for participants who complete the full course. Participants must pass at least 7 of 8 quizzes and the final exam to earn the EEU.

Distribution of Certificates

- All certificates (PDH and EEU) will be issued after the course is completed (the week of October 22). Only the registrant will receive certificates for the course.

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Course Resources

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

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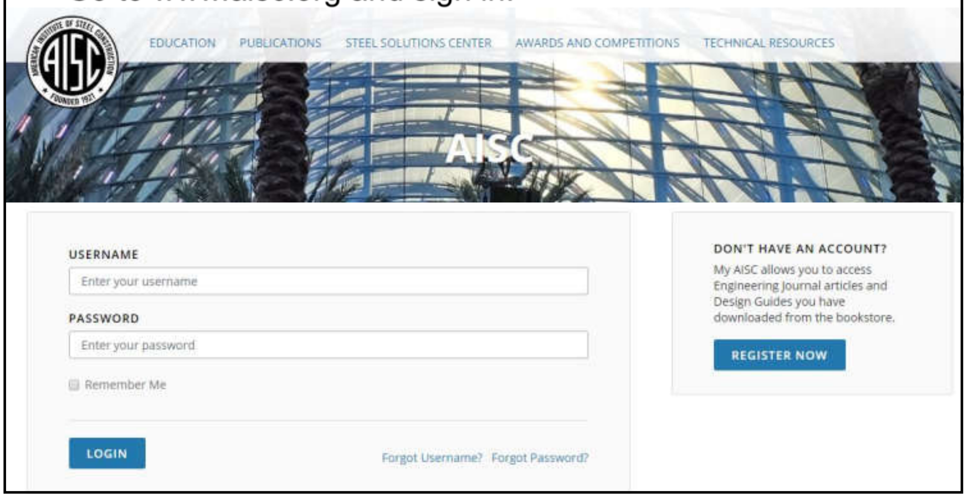

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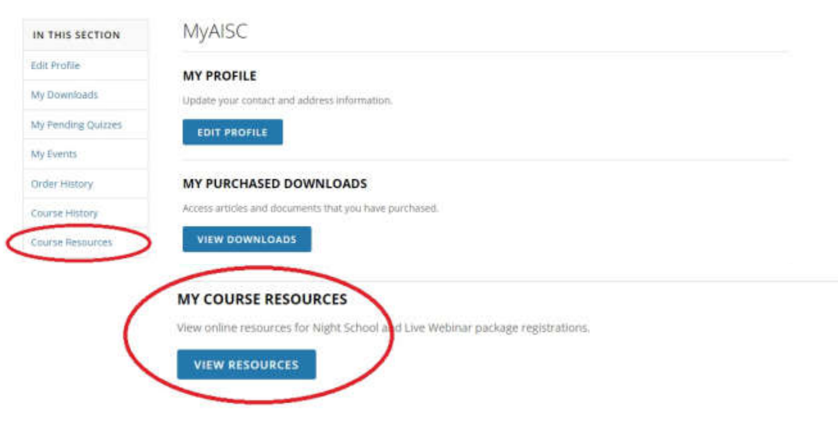

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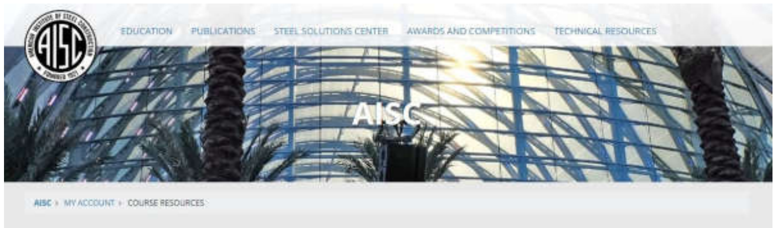
Course Resources



Course Resources

Event	Start Date
Test Course	1/1/1900 12:00:00 AM
NS 15 8-Session Package-Night School 15 - Fundamentals of Connection Design	10/3/2017 7:00:00 PM
NS 16 8-Session Package-Night School 16 - Seismic Design in Steel	2/5/2018 7:00:00 PM
NS 17 4-Session Package-Night School 17: Design of Facade Attachments	7/16/2018 7:00:00 PM
8-Session Package-Seismic Design in Steel - Concepts & Examples	7/16/2018 1:30:00 PM

8-Session Package Registrants
Course Resources

Seismic Design in Steel

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
R1: Introduction To Effective Seismic Design	N/A	Handouts	View Passcode: 18NSRS1	Pass Score: 100	N/A
R2: Seismic Design Concepts - Moment Frames	N/A	Handouts	View Passcode: 18221218	Pass Score: 100	N/A
R3: Seismic Design Concepts - Braced Frames	N/A	Handouts	View Passcode: 3610305	Pass Score: 100	N/A
R4: Seismic Design Concepts - Design	N/A	Handouts	View Passcode: N/A	Pass Score: 100	N/A
L1: Application - Planning the Seismic Design	Sep 10 2018 1:30PM EDT	Handouts	Available 09/12/2018 5:00PM EDT	Available 09/12/2018 5:00PM EDT	Pending
L2: Application - Building Analysis/Diaphragms	Sep 17 2018 1:30PM EDT	Handouts	Available 09/19/2018 5:00PM EDT	Available 09/19/2018 5:00PM EDT	Pending
L3: Application - Moment Frames	Sep 24 2018 1:30PM EDT	Handouts	Available 09/26/2018 5:00PM EDT	Available 09/26/2018 5:00PM EDT	Pending
L4: Design of the Braced Frames	Oct 1 2018 1:30PM EDT	Handouts	Available 10/03/2018 5:00PM EDT	Available 10/03/2018 5:00PM EDT	Pending
Seismice Design in Steel - Final Exam	Oct 3 2018 12:00AM EDT			Available 10/05/2018 5:00PM EDT	

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Thank You

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

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