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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



Course objectives

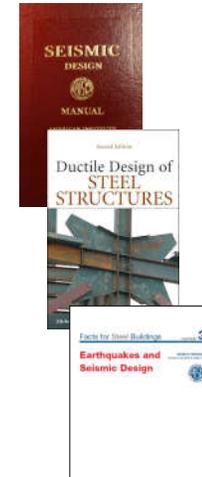
- Understand the principles of seismic design of steel structures.
- Understand the application of those principles to two common systems:
 - Special Moment Frames
 - Buckling-Restrained Braced Frames.
- Understand the application of design requirements for those systems.



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



11

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



12

Course outline

Part II: Application

- L1. Planning the seismic design
- L2. Building analysis and diaphragm design
- L3. Design of the moment frames
- L4. Design of the braced frames



13

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Session L2: Building analysis and Diaphragm design



Session topics

- Building Analysis
 - Lateral analysis methods
 - Load cases
 - Structural model
 - Design for stability & 2nd-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



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

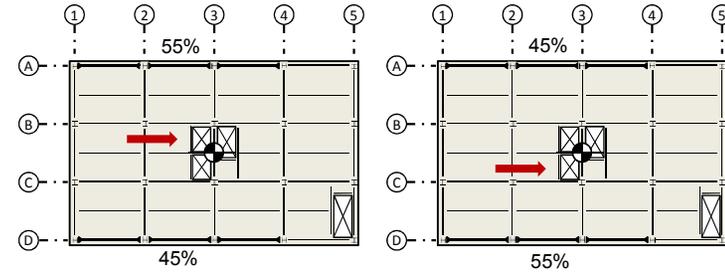
- 2 principal axes
 - Orthogonal combination not required
 - No skewed frames in this example
 - No columns shared by orthogonal frames
- Accidental torsion
 - Required for non-flexible diaphragms
 - 5% offset of mass in either direction
 - For simplicity, conservatively neglect torsional resistance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



ASCE 7 §12.8.4.2

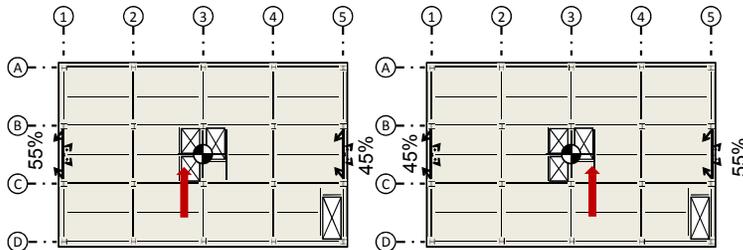
21

Load Cases



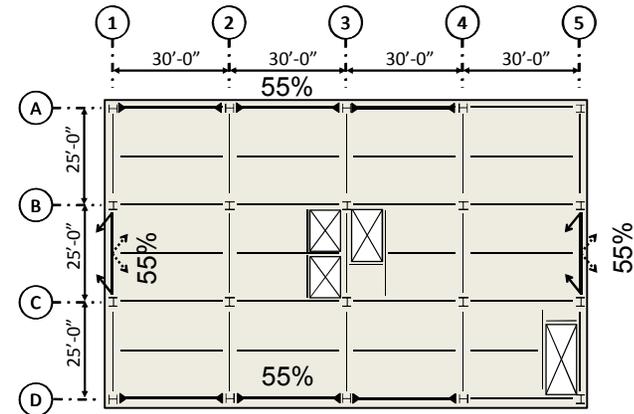
22

Load Cases



23

Load Cases (envelope)



24

Base Shear

- From Session 5
 - SMF
 - $V = 0.0818 \cdot 3313K$
 - $= 271K$
 - BRBF
 - $V = 0.100 \cdot 3313K$
 - $= 331K$

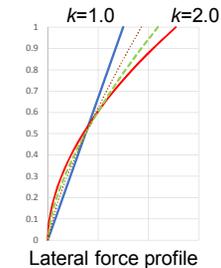


25

Vertical Distribution

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

- $T \leq 0.5s$ $k=1.0$
- $T \geq 2.5s$ $k=2.0$
- $0.5s < T < 2.5s$ 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_i} , (kips)
Roof	708.4	51.5	83,115	0.373	101
4 th	868.1	39	72,778	0.327	89
3 rd	868.1	26.5	45,617	0.205	55
2 nd	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_i} , (kips)
Roof	708	51.5	59,687	0.362	120
4 th	868	39.0	53,499	0.325	108
3 rd	868	26.5	34,639	0.210	70
2 nd	868	14.0	16,898	0.103	34
Total	3313		164,723	1.000	331



ASCE 7 §12.8.3

28

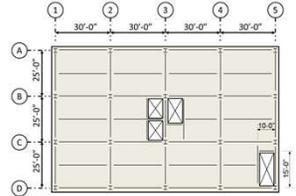
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Structural model



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



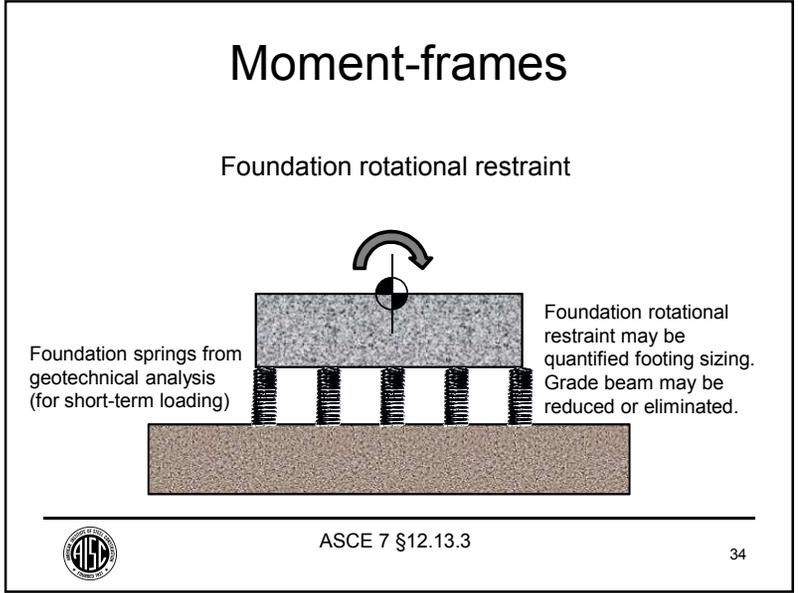
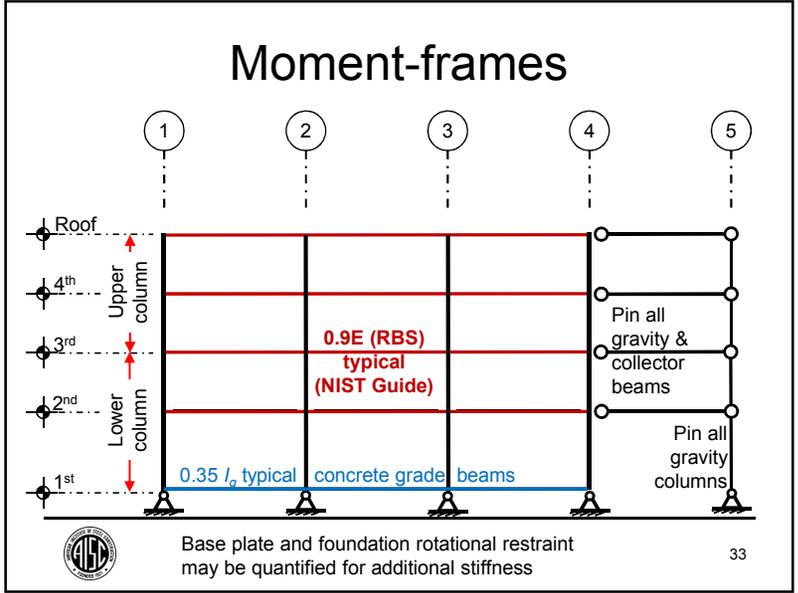
31

Moment frames

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



32



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

Most flexibility from yield length L_y
Modeled full workpoint length L_{wp}

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

$$1.2 \leq K_F \leq 1.8$$

APPROXIMATE STIFFNESS MODIFICATION FACTORS, $K_F^{1,2,7}$

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

$F_y = 38 \text{ ksi (262 MPa)}$	A_{sc} in ² (cm ²)	P_y kip (kN)	Bay Width, ft (m)									
			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 (28)	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
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 (1397)		1.54	1.47	1.42	1.39	1.36	1.48	1.43	1.39	1.36	1.33
10.0 (68)	348 (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)	688 (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



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
 - $N_i = 2.1 \frac{\Delta}{h} Y_i \approx 0.008 Y_i$ 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_v 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 2nd 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 $\sim 0.084 P_{story}$ 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

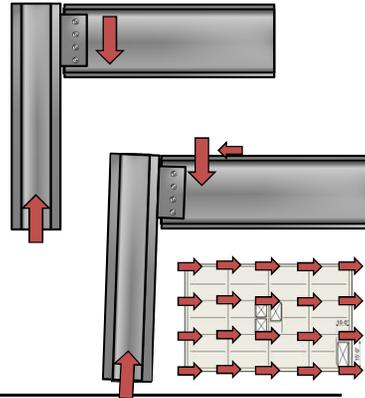
- Second-order analysis
 - Equilibrium in deformed condition
 - Analysis must include all gravity load
- Approximate second-order analysis
 - Appendix 8
 - B_1 Beam-columns
 - B_2 The entire lateral-load-resisting system
- ASCE 7 §12.8.7 requires consideration of second order effects for forces and displacements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



44

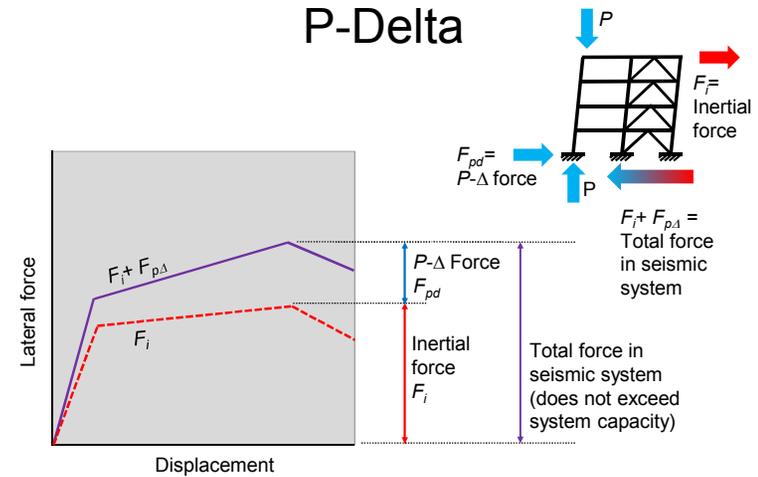
Second-order effects (P-Delta)

- 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



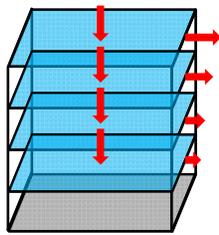
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P-Delta



46

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



- 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

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

Largest B_2 at base for seismic



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
4 th	1.03
3 rd	1.03
2 nd	1.04

- Assume drift-controlled
 - $\Delta_{elastic} = 0.02h/C_d = 0.0036h$
- Use $P-\Delta$ gravity load
 - $P = 1.0D + 0.5L$
- Assume $P_{mf} = \frac{1}{2} P_{story}$
- $B_2 = 1.04$ (largest) < 1.1
 - Use ELM
 - $K=1$



AISC 360 Appendix 8

48

Second Order Effects: BRBF

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

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

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

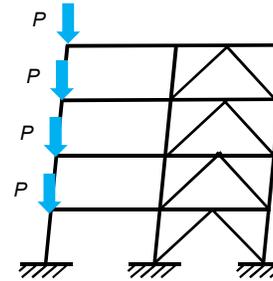
- Assume drift
 - $\Delta_{elastic} = 0.02h/C_d = 0.004h$
- Use $P-\Delta$ gravity load
 - $P=1.0D+0.5L$
- $B_2 = 1.04$ (largest)



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

$$\theta = \frac{P_x \Delta_e}{V_x h_{sx} C_d} = \frac{P_x \Delta_{elastic}}{V_x h_{sx}} = \frac{P_x / h_{sx}}{K_x} \quad \theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

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

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

- OK

Where β is the ratio of shear demand to shear capacity



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



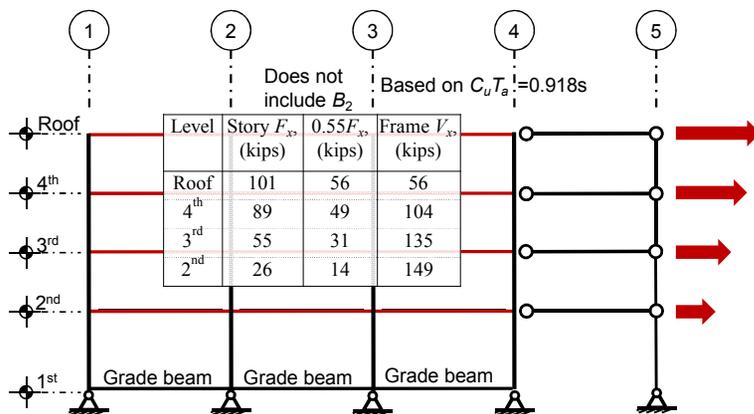
53

Analysis forces

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Moment-frames



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 $C_u T_a$
- 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

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

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

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

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

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

% 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_{xs} (kips)	$0.55F_{xs}$ (kips)	Frame V_{xs} (kips)
Roof	120	66	66
4 th	108	59	125
3 rd	70	38	164
2 nd	34	19	182

Does not include $\rho=1.3$
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_{xs} (kips)	B_2	ρ	Frame $\rho B_2 F_{xs}$ (kips)	Frame $\rho B_2 V_{xs}$ (kips)
Roof	66.0	1.02	1.30	87.6	88
4 th	59.2	1.03	1.30	79.3	167
3 rd	38.3	1.03	1.30	51.3	218
2 nd	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)
4 th	62
3 rd	118
2 nd	154
1 st	183

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

Assume braces resist 100% of story shear

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

63

Diaphragm Forces

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

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

$\rho=1.0$ 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 64

There's always a solution in steel.

Diaphragm analysis



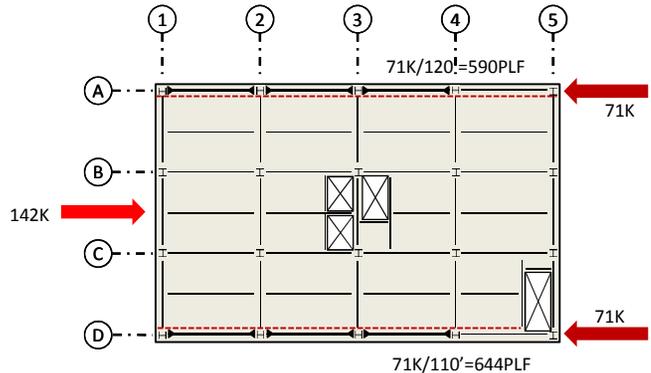
Diaphragm analysis

- 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

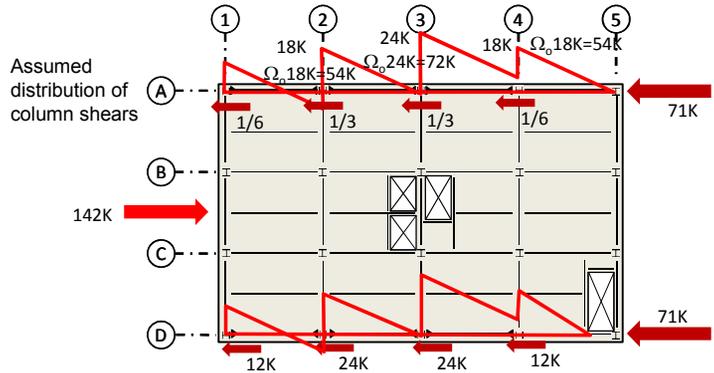


Does not include B_2



67

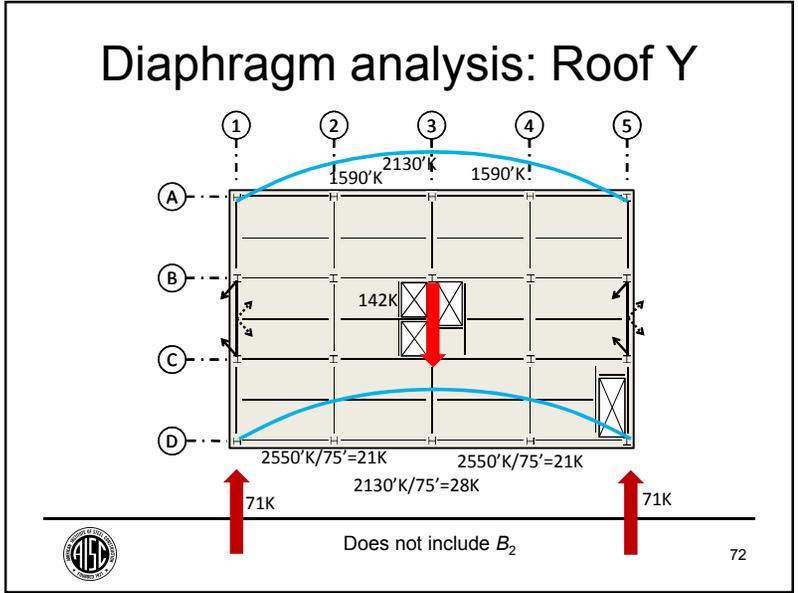
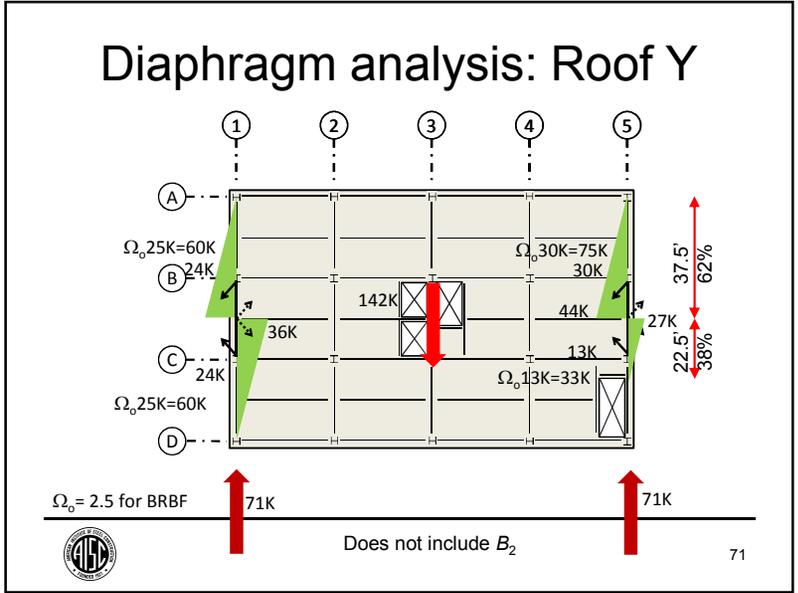
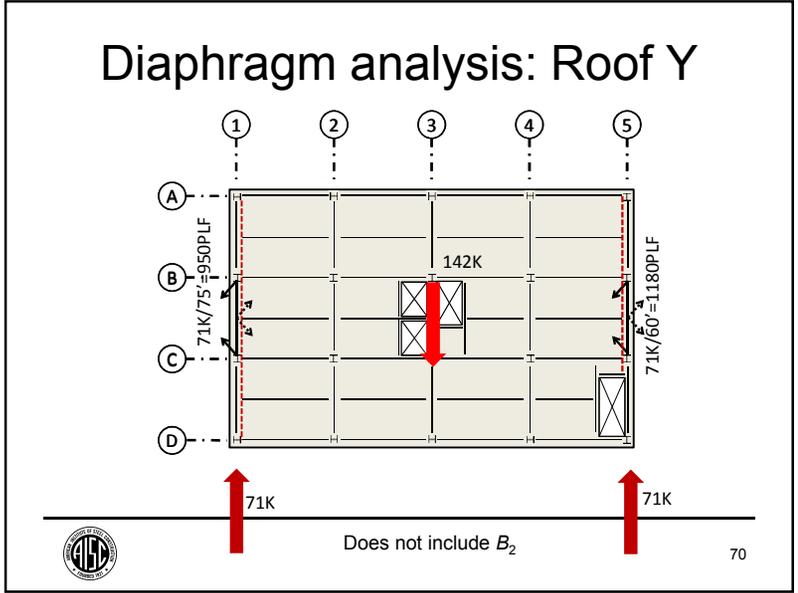
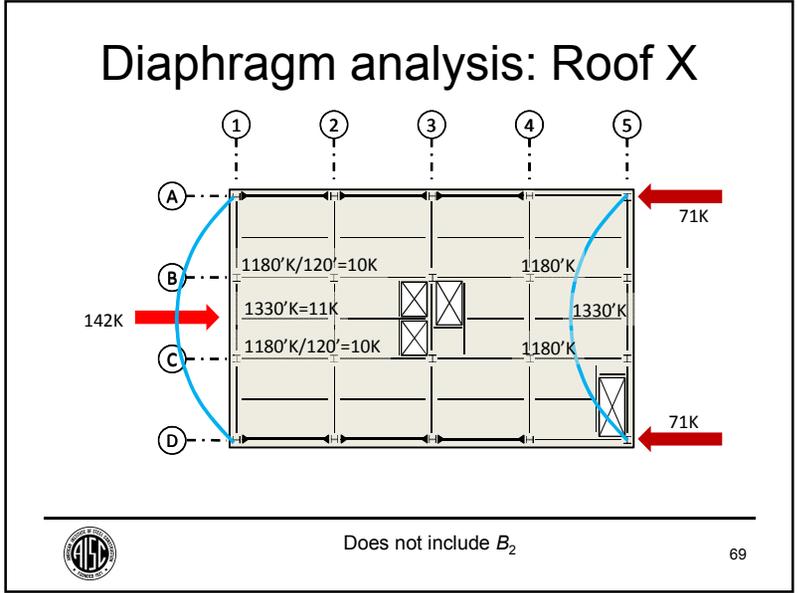
Diaphragm analysis: Roof X



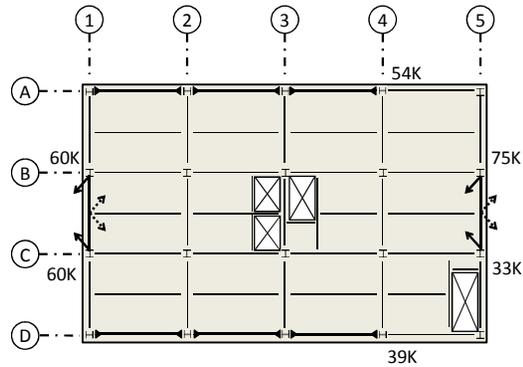
Does not include B_2



68



Chord/collector forces



Does not include B_2

73

Capacity-design forces

There's always a solution in steel.



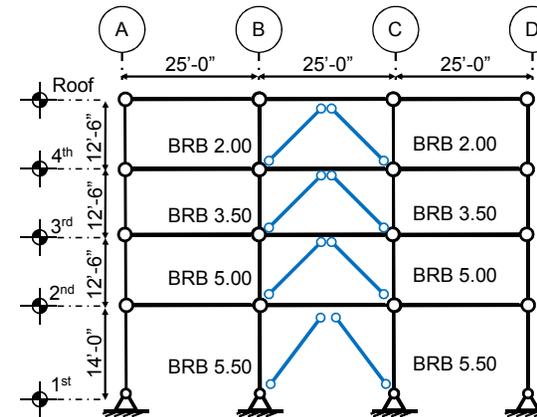
Capacity-design forces

- Per ASCE 7 §12.4.3 the overstrength seismic load, $\Omega_0 E_h$, 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



75

Brace sizes



Design of BRBF presented in Session 8

76

Brace capacity (4th floor)

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



AISC 341 §F4.2a

77

Diaphragm design

There's always a solution in steel.



Deck selection

- Maximum shear
 - 1.18KLF
 - $B_2 * 1.18\text{KLF} = 1.20\text{KLF}$
- 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



79

Deck selection

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



ACI 318 §11.2

80

Shear transfer

- Design for Ω_o & B_2
 - SMF
 - $1.02 \times 3 \times 0.644 \text{KLF} = 1.97 \text{KLF}$
 - BRBF
 - $2.5 \times 1.20 \text{KLF} = 3.00 \text{KLF}$
- Provide $\frac{3}{4}'' \times 4''$ 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)
 - $F_u = 65 \text{ ksi}$

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c}$$

$$Q_n \leq R_g R_p A_{sc} F_u$$

$$A_{sc} = \frac{\pi}{4} \left(\frac{3}{4} \text{ in} \right)^2 = 0.44 \text{ in}^2$$

$$f'_c = 4000 \text{ psi}$$

$$E_c = w_c^{1.5} \sqrt{f'_c}$$

$$E_c = (115 \text{ pcf})^{1.5} \sqrt{4 \text{ ksi}}$$

$$E_c = 2466 \text{ ksi}$$



AISC 360 §18.2a

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"
 - $5.6 \text{ KLF} > 3.0 \text{ KLF}$



AISC 360 §18.2a

83

Collector design

There's always a solution in steel.



Collector design

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



85

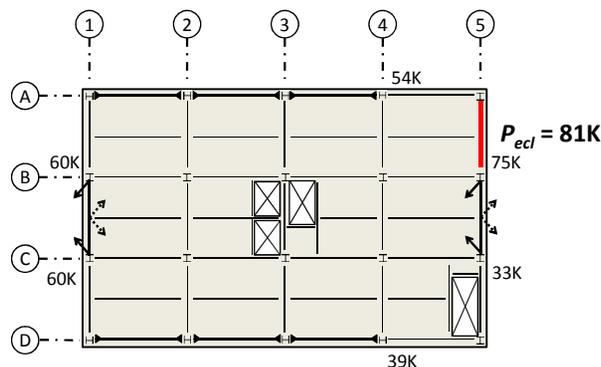
Collector design

- 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



86

Collector design

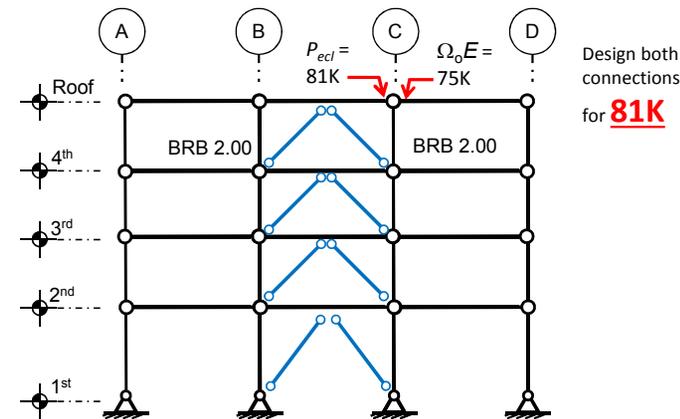


Capacity design: $P_{ecl} = 75K * BRB_Capacity / \Omega_o V_{frame} = 75K * 179K / (2.5 * 66K) = 81K$



87

Collector forces



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)



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/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



1-20 DIMENSIONS AND PROPERTIES

Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d	Web Thickness, t _w	Flange Thickness, t _f		Distance		Workable Edge
				W ₁	W ₂	X ₁	X ₂	
W21x50	27.2	21.6	0.380	0.820	1.020	10.0	10.0	10.0
W21x45	24.4	21.4	0.375	0.800	1.000	10.0	10.0	10.0
W21x40	21.5	21.2	0.365	0.780	0.980	10.0	10.0	10.0
W21x35	20.0	21.1	0.360	0.770	0.970	10.0	10.0	10.0
W21x30	18.3	21.0	0.355	0.760	0.960	10.0	10.0	10.0
W21x25	16.2	20.8	0.350	0.750	0.950	10.0	10.0	10.0
W21x20	14.1	20.6	0.345	0.740	0.940	10.0	10.0	10.0
W21x15	12.1	20.5	0.340	0.730	0.930	10.0	10.0	10.0
W21x10	10.3	20.4	0.335	0.720	0.920	10.0	10.0	10.0
W21x6	8.8	20.3	0.330	0.710	0.910	10.0	10.0	10.0
W21x3	7.5	20.2	0.325	0.700	0.900	10.0	10.0	10.0

1-21 DIMENSIONS AND PROPERTIES

Table 1-1 (continued)
W-Shapes
Properties

Shape	Compact Section Coefficients		Area S-X			Area S-Y			Torsional Properties					
	λ _p	λ _r	S _x	S _y	Z _x	Z _y	J	C _w	I _p	r _p				
W21x50	0.31	0.50	2070	102	870	221	194	347	234	207	6.03	9840		
W21x45	0.31	0.50	1800	171	860	196	814	195	183	305	2.21	2016	434	8630
W21x40	0.31	0.50	1600	150	848	172	708	173	181	268	2.30	2025	332	7160
W21x35	0.31	0.50	1400	140	840	160	647	157	180	244	2.17	2014	245	6760
W21x30	0.31	0.50	1200	127	834	144	575	143	177	217	2.15	2014	183	5960
W21x25	0.31	0.50	1000	110	830	128	484	118	173	184	2.11	2003	134	4980
W21x20	0.31	0.50	800	93	828	107	387	93	168	149	2.09	2022	93	3950
W21x15	0.31	0.50	600	78	826	89	292	83	165	148	1.99	2015	77	3160
W21x10	0.31	0.50	400	64	824	74	204	70	164	130	1.92	1943	54	2570
W21x6	0.31	0.50	200	51	822	60	138	58	162	102	1.80	1923	37	2110
W21x3	0.31	0.50	100	41	820	48	87	47	160	78	1.70	1903	26	1710

* Shapes in asterisk for compression with $F_y = 50$ ksi.
 † Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
 ‡ The actual size, composition and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
 § Flange thickness greater than 2 in. Special requirements may apply per AISI Spectral Section A3.1.

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90

Collector design

- Compressive Strength
 - Major axis
 - $(KL)_x = 25'-0"$
 - Minor axis
 - $(KL)_y = 0'-0"$
 - Constrained-axis flexural-torsional buckling
 - $(KL)_{CAFT} = 12'-6"$

92

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23

Collector design

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



93

Collector design

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



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

94

Collector design

Table 1. (continued)
Torsional Buckling Design Strength in Axial Compression
 ϕP_n , kip

$F_y = 50$ ksi

Shape	W18x							W16x						
	50	46	40	35	100	89	77							
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	$P_u / \phi P_n = 81\text{K}/386\text{K} = 0.21$									
15	428	335	357	235										760
16	414	311	344	216	276	170	217	131	1090	969	948	843	796	734
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



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

95

Collector Design

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



AISC 360 Appendix 8 §8.2.1

96

Collector Design

- Flexural strength
 - AISC Manual Table 3-19
 - $Y_2=3.5$ in
 - Use $\Sigma Q_n=184$ K
 - $184\text{K}/11.2\text{K}/_{\text{stud}}=16.4$ studs (each side of midpoint)
 - 32.8 studs
 - Collector studs
 - $81\text{K}/11.2\text{K}/_{\text{stud}}=7.2$ studs
 - Not additive to flexure studs
- $\phi M_n = 516$ 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.

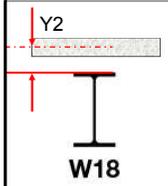
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97

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

$F_y = 50$ ksi



Shape	M_p/Ω_b		PNA ^c	Y_1^a in.	ΣQ_n kip	Y_2^b , in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18x50	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									

$\Omega_b = 1.67$ $\phi_p = 0.90$

^a Y_1 = distance from top of the steel beam to plastic neutral axis
^b Y_2 = 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
 - $\phi M_n = 379$ kip-ft
- $P_u / \phi P_n = 81\text{K}/386\text{K} = 0.21 > 0.2$
- $P_u / \phi P_n + 8/9 M_u / \phi M_n$ (H1-1b)
- $= (0.21) + 8/9 (142\text{K}) / (379\text{K}) = 0.55$ OK
 - Provide studs @ 24" (12 studs)



99

Collector connections

There's always a solution in steel.



Collector connection

- Single-plate connection
 - (4) 7/8" Ø A325N bolts
 - 3" spacing
 - 1.5" edge distance top & bottom
 - 2.5" side edge distance
 - 3/8" A36 plate
 - 1/4" double-sided fillets

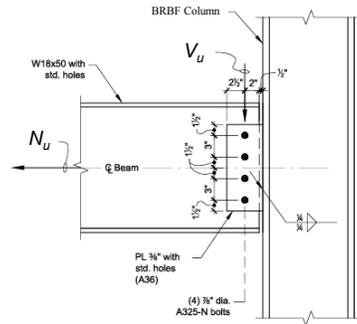
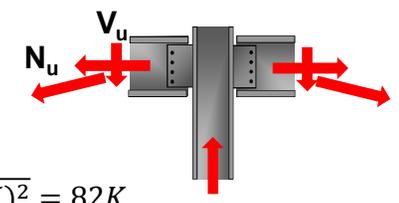


Fig. 8-6. Collector connection investigated in Example 8.4.2.



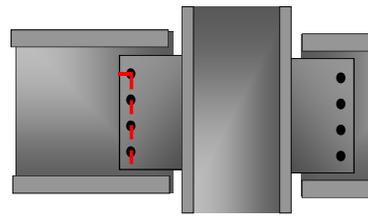
Required strength

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



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
 - $\phi V_n = 78.3K$



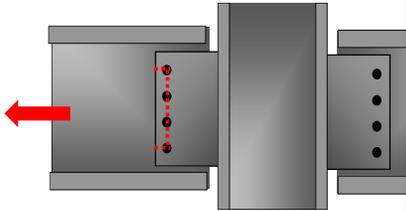
Table 10-10a (continued)
Single-Plate Connections
Plate $F_y = 36$ ksi
Bolt, Weld and Single-Plate Available Strengths, kips
7/8-in.-diameter bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
4 (L = 12)	Group A	N	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
		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	—	—
			SSLT	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	71.2	107

$\phi V_n = 13K$
 $\phi V_n = 78.3K$
 $\frac{13K}{78.3K} = 0.176$
 $\sqrt{\left(\frac{V_u}{\phi V_n}\right)^2 + \left(\frac{T_u}{\phi T_n}\right)^2} \leq 1$
 $\frac{T_u}{\phi T_n} \leq \sqrt{1 - \left(\frac{V_u}{\phi V_n}\right)^2} = 0.98$
 $\phi T_n \geq T_u / 0.98$
 $\phi T_n \geq 81K / 0.98 = 82K$



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
 - $t_w(65\text{ksi}/58\text{ksi}) =$
 - $0.355''(65/58) = 0.39'' > \frac{3}{8}''$
 - $t_w(50\text{ksi}/36\text{ksi}) =$
 - $0.355''(50/36) = 0.49'' > \frac{3}{8}''$
 - Plate governs


105

Collector connection

- Plate limit states
 - Yield: $\phi F_y A$
 - $= 0.9(36\text{ksi})(\frac{3}{8}'')(12'')$
 - $= 146\text{K}$
 - Rupture: $\phi F_u A_e = F_u A_n$
 - $= 0.75(58\text{ksi})(\frac{3}{8}'')(12'' - 4'')$
 - $= 130\text{K}$
- Block shear $\phi R_n \leq$
 - $\phi(0.6F_u A_{nv} + U_{BS}F_u A_{nt})$
 - $0.75(\frac{3}{8}'')(0.6 \cdot 58\text{ksi} \cdot 2 \cdot 2'' + 1.0 \cdot 58\text{ksi} \cdot 6'')$ = 137K
 - $\phi(0.6F_y A_{gv} + U_{BS}F_u A_{nt})$
 - $0.75(\frac{3}{8}'')(0.6 \cdot 50\text{ksi} \cdot 2 \cdot 2.5'' + 1.0 \cdot 58\text{ksi} \cdot 6'')$ = 140K


AISC 360 §J4
106

Collector connection

- Bolt limit states
 - Shear: Table 7-1
 - $4 \cdot 24.3\text{K} = 97.2\text{K}$
 - Bearing (spacing)
 - Table 7-4
 - $4(\frac{3}{8}'' \cdot 91.4\text{K}/\text{in}) = 137\text{K}$
 - Bearing (edge distance)
 - Table 7-5
 - $4(\frac{3}{8}'' \cdot 79.9/\text{in}) = 120\text{K}$
- Governing strength:
 - $\phi R_n = 97.2\text{K}$
 - $R_u / \phi R_n = 81\text{K} / 97\text{K} = 0.84$

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


AISC 360 §J3
107

Diaphragm openings

There's always a solution in steel.

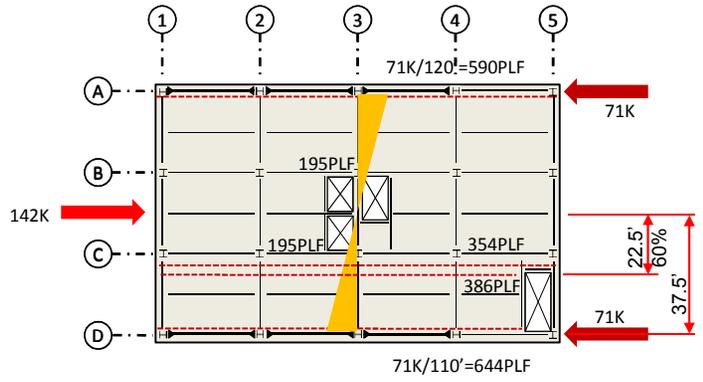


Diaphragm openings

- Local shears
- Local collector forces
- Local chord forces

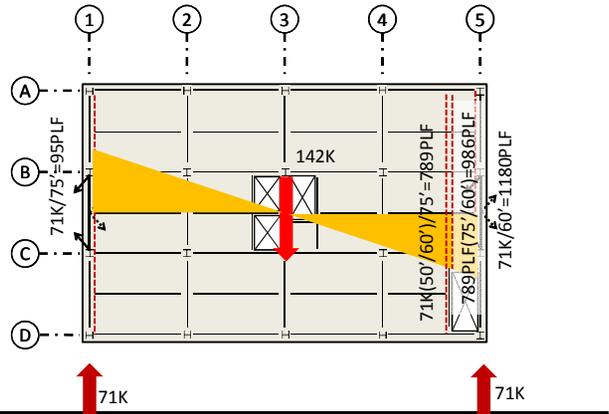


Diaphragm analysis: Roof X



Does not include B_2

Diaphragm analysis: Roof Y

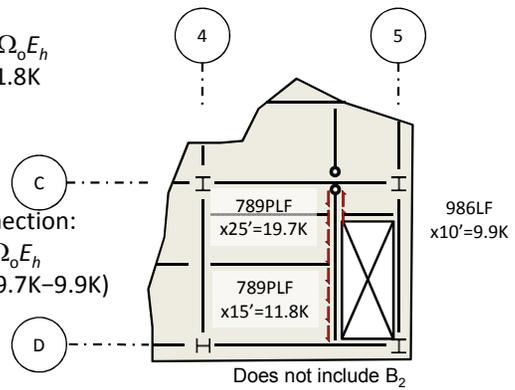


Does not include B_2

Local collector forces: Y

Collector:
 Design for $B_2\Omega_0E_h$
 $= 1.02 * 2.5 * 11.8K$
 $= 30.2K$

Collector connection:
 Design for $B_2\Omega_0E_h$
 $= 1.02 * 2.5 * (19.7K - 9.9K)$
 $= 25.0K$



Does not include B_2



Local chord forces: Y

$$\frac{1.18klf + 0.986klf}{2} 10' = 10.8K$$

Chord and connection:
Design for B_2E_h
= $1.02 \cdot 10.8K$
= 11.0K

Does not include B_2

113

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Summary

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

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End of session L2

Next:
Design of the Moment frames

Additional resources

Guide to the Design of Diaphragms, Chords and Collectors
Based on the 2005 IBC and ASCE/SEI 7-05

NEHRP Seismic Design Technical Brief No. 5
Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms
A Guide for Practicing Engineers

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

MEMBER	NON-MEMBER
FREE	\$110.00

ADD TO CART

Liu, D.; Davis, B.; Arban, L.; Sabelli, R. (2013). "Torsional and Constrained-Axis Flexural-Torsional Buckling Tables for Steel W-Shapes in Compression." Engineering Journal, American Institute of Steel Construction, Vol. 50, pp. 255-247.

Torsional buckling (TB), an applicable limit state for W-shape members subject to axial compression, often controls when the torsional effective unbraced length exceeds the major-axis flexural buckling effective unbraced length. Constrained-axis flexural-torsional buckling (CAFTB) is a potential limit state for W-shape members that are constrained to buckle with the center of twist at a location other than the centroidal axis, as is the case for a typical beam with one flange braced by a diaphragm and the other unbraced. Manual calculation of the TB or CAFTB available compressive strength is a somewhat lengthy process, especially when the section is slender for axial compression, and no design aid currently exists in the AISC Manual. This paper provides tables that facilitate the determination of TB and CAFTB available compressive strengths. Several example calculations are also provided.

Published: 2013, Quarter 4.

AUTHOR(S):
Dr. Liu: Brad Davis, Leigh Arban, and Rafael Sabelli

117

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Question time

structural STEEL

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days...

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

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 Package Registrants CEU/PDH Certificates

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



8-Session Package Registrants CEU/PDH Certificates

Videos and Quizzes

- 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 CEU/PDH Certificates

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.



8-Session Package Registrants Course Resources

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



8-Session Package Registrants Course Resources

Go to www.aisc.org and sign in.

The screenshot shows the AISC website's login page. At the top, there is a navigation menu with links for EDUCATION, PUBLICATIONS, STEEL SOLUTIONS CENTER, AWARDS AND COMPETITIONS, and TECHNICAL RESOURCES. Below the menu is a large banner image of a modern building with a glass facade. The AISC logo is prominently displayed in the center of the banner. Below the banner, there is a login form with fields for USERNAME and PASSWORD, a 'Remember Me' checkbox, and a 'LOGIN' button. To the right of the login form, there is a 'DON'T HAVE AN ACCOUNT?' section with a brief description of the MyAISC account and a 'REGISTER NOW' button. At the bottom of the login form, there are links for 'Forgot Username?' and 'Forgot Password?'.

8-Session Package Registrants Course Resources

Go to www.aisc.org and sign in.

The screenshot shows the MyAISC user profile page. On the left side, there is a 'MyAISC' sidebar with links for 'Edit Profile', 'My Downloads', 'My Pending Quizzes', 'My Events', 'Order History', and 'Course History'. The 'Course Resources' link is circled in red. On the right side, there are sections for 'MY PROFILE', 'MY PURCHASED DOWNLOADS', and 'MY COURSE RESOURCES'. The 'MY COURSE RESOURCES' section is also circled in red and contains the text 'View online resources for Night School and Live Webinar package registrations.' and a 'VIEW RESOURCES' button.

8-Session Package Registrants Course Resources

The screenshot shows the AISC website's course resources page. At the top, there is a navigation menu with links for EDUCATION, PUBLICATIONS, STEEL SOLUTIONS CENTER, AWARDS AND COMPETITIONS, and TECHNICAL RESOURCES. Below the menu is a large banner image of a modern building with a glass facade. The AISC logo is prominently displayed in the center of the banner. Below the banner, there is a breadcrumb trail: 'HOME > MY ACCOUNT > COURSE RESOURCES'. Below the breadcrumb trail, there is a 'Course Resources' section with a table of course events.

Event	Start Date
Steel Course	1/1/1901 12:00:00 AM
8-Session Package Registrants Session L1 - Fundamentals of Construction Design	09/19/2018 10:00:00 PM
8-Session Package Registrants Session L2 - Seismic Design in Steel	09/19/2018 10:00:00 PM
8-Session Package Registrants Session L3 - Design of Tension Members	11/08/2018 10:00:00 PM
8-Session Package Registrants Session L4 - Compression Members	11/08/2018 10:00:00 PM

8-Session Package Registrants Course Resources

The screenshot shows the AISC website's course resources page. At the top, there is a navigation menu with links for EDUCATION, PUBLICATIONS, STEEL SOLUTIONS CENTER, AWARDS AND COMPETITIONS, and TECHNICAL RESOURCES. Below the menu is a large banner image of a modern building with a glass facade. The AISC logo is prominently displayed in the center of the banner. Below the banner, there is a breadcrumb trail: 'HOME > MY ACCOUNT > COURSE RESOURCES > SEISMIC DESIGN IN STEEL'. Below the breadcrumb trail, there is a 'Seismic Design in Steel' section with a sub-section for '8-SESSION PACKAGE RESOURCES'. Below this sub-section, there is a table of course events.

Event	Date	Platform	Video	Quiz	Attendance
R1 Introduction to Effective Seismic Design	N/A	OnDemand	Video	Pass	N/A
R2 Seismic Design Concepts - Moment Frames	N/A	OnDemand	Video	Pass	N/A
R3 Seismic Design Concepts - Braced Frames	N/A	OnDemand	Video	Pass	N/A
R4 Seismic Design Concepts - Design	N/A	OnDemand	Video	Pass	N/A
L1 Application - Planning the Seismic Design	See 10/22/2018 1:00PM EDT	OnDemand	Available 09/22/2018 5:00PM EDT	Available 09/12/2018 5:00PM EDT	Pending
L2 Application - Building Analysis/Designs	See 17/2018 1:00PM EDT	OnDemand	Available 09/20/2018 5:00PM EDT	Available 09/13/2018 5:00PM EDT	Pending
L3 Application - Moment Frames	See 04/2018 1:00PM EDT	OnDemand	Available 09/20/2018 5:00PM EDT	Available 09/13/2018 5:00PM EDT	Pending
L4 Design of the Braced Frames	See 12/2018 1:00PM EDT	OnDemand	Available 09/20/2018 5:00PM EDT	Available 09/13/2018 5:00PM EDT	Pending
Seismic Design in Steel - Final Exam	Oct 3, 2018 12:00AM EDT	OnDemand	Available 09/20/2018 5:00PM EDT	Available 09/13/2018 5:00PM EDT	Pending

There's always a solution in steel.

Thank You

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

A large, faint watermark of the American Institute of Steel Construction (AISC) logo is visible in the bottom right corner of the slide. The logo consists of a circular emblem with the letters 'AISC' in the center, surrounded by the text 'AMERICAN INSTITUTE OF STEEL CONSTRUCTION' and 'FOUNDED 1921'. Below the emblem, the words 'structural STEEL' are written in a stylized font.