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Course Description

Session L3: Design of the Moment Frames

September 24, 2018

This session presents system/connection selection and reviews prequalification issues pertaining to moment frames. The lecture then addresses moment frame analysis, beam, column and connection design. The session concludes with the treatment of base-plates.



Learning Objectives

- Identify the various prequalified connections for use in moment frames.
- List the advantages and disadvantages of the reduced beam section (RBS) connection.
- Describe strong-column / weak-beam proportioning.
- Identify the steps of a moment frame base-plate design.



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Seismic Design in Steel: Concepts and Examples

Session L3: Design of the Moment Frames
September 24, 2018



Rafael Sabelli, SE



Course objectives

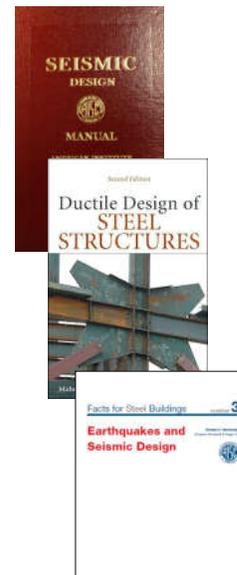
- Understand the principles of seismic design of steel structures.
- Understand the application of those principles to two common systems:
 - Special Moment Frames
 - Buckling-Restrained Braced Frames.
- Understand the application of design requirements for those systems.



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
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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 L3: Design of the Moment Frames



Session topics

- Prequalification issues
- Frame analysis
- Connection design
- Optimization
- Beam strength design
- Column strength design
- Base-plate design
- Completion of design



15

Design overview

- Assume frame is drift governed
 - Postpone strength check until frame optimized
 - Drift
 - Minimization of connection reinforcement
 - Use software to compute drift
 - Model as discussed in Session L2
 - Member selection
 - Prequalification limits
 - Seismic compactness (“highly ductile member”)
 - Strong-column/Weak-beam proportioning



16

Design overview

- RBS selected
 - Good illustration of design procedure
 - Relatively simple analysis
- WUF-W is a good candidate for a small building
 - Too simplified a design procedure
- No field welding for:
 - Bolted flange plate (BFP)
 - Bolted end plate (BEP)
- Connection selection considers
 - Contractor preferences & capabilities
 - Inspection & NDT requirements—shop and field



17

Prequalification issues

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Prequalification issues

- Limits on member sizes
- Limits on degree of beam reduction
- Analysis requirements
- Special design procedure in AISC 358
- Prescribed detailing
- Lateral bracing requirements
- Protected zone



19

RBS Prequalification limits

- Beam
 - I-shaped
 - W36 max.
 - 300 PLF max.
 - $t_f \leq 1\frac{3}{4}$ "
 - Span/depth ≥ 7
 - Seismically compact (in center 2/3 of RBS)
 - Laterally braced at hinge (exception for composite deck)
- Column
 - I-shaped
 - Box
 - Boxed WF
- Connection
 - 25%-50% flange reduction
 - 15%-30% strength reduction
 - Dimensional limits



AISC 358 §5.3

20

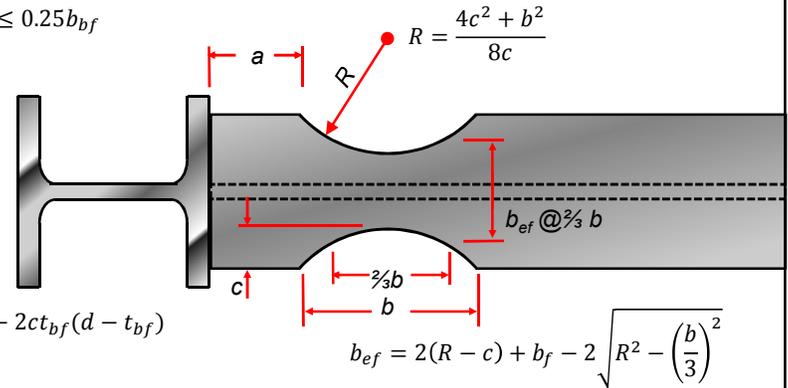


Reduced Beam Section (RBS)

$$0.5b_{bf} \leq a \leq 0.75b_{bf}$$

$$0.65d \leq b \leq 0.85d$$

$$0.1b_{bf} \leq c \leq 0.25b_{bf}$$



$$Z_{RBS} = Z_x - 2ct_{bf}(d - t_{bf})$$

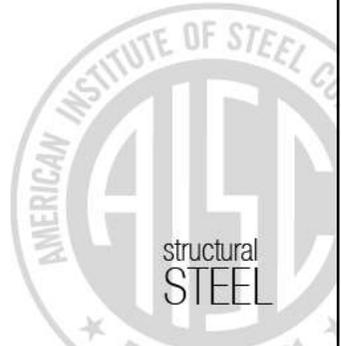
$$b_{ef} = 2(R - c) + b_f - 2\sqrt{R^2 - \left(\frac{b}{3}\right)^2}$$



AISC 358 §5.8

Frame analysis

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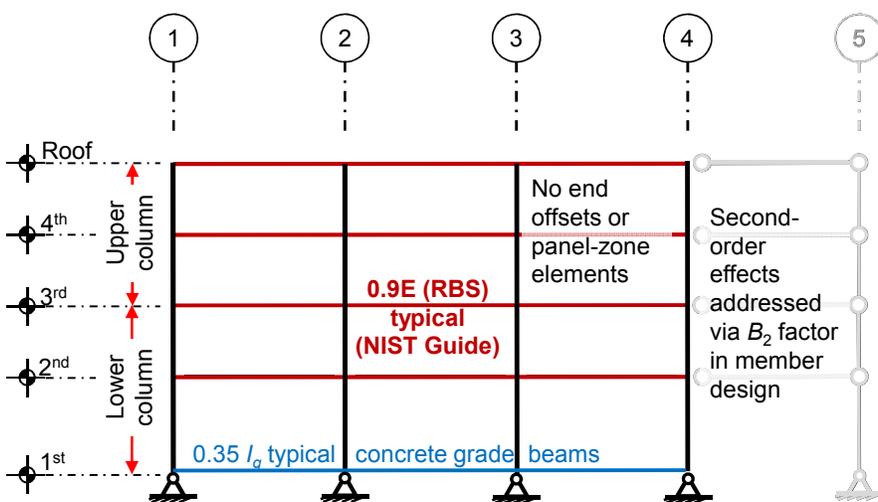
Frame analysis

- Frame model set-up
 - Stiffness modifications
 - Panel-zone modeling
 - End offsets
 - Second-order effects
- Member selection
 - Prequalification
 - Base condition



23

Moment-frame model



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

24

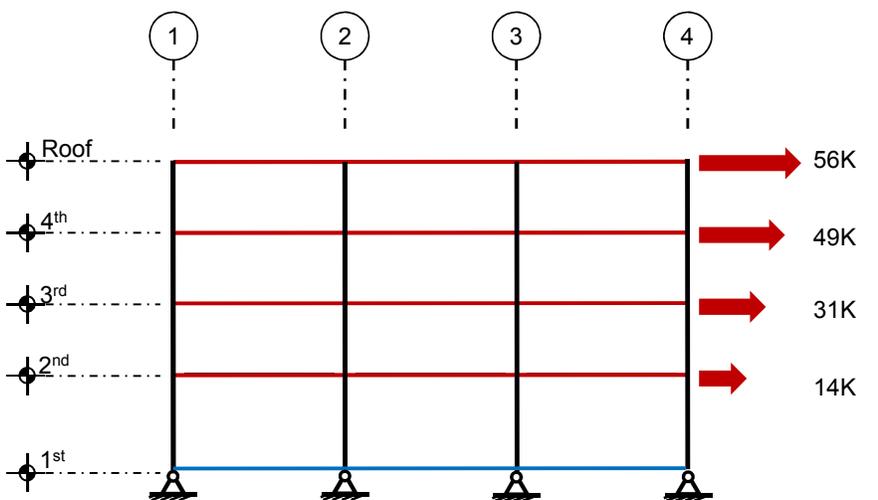


Moment-frame model

- AISC 358 suggests 10% increase in overall drift as a liberal estimate of RBS effect
- NIST Tech Brief on Steel SMF recommends 10% reduction in beam stiffness as a reasonable approximation
 - RBS does not affect column
- Many software programs can automatically include non-prismatic RBS members
 - RBS effect on drift is often 2%-5%

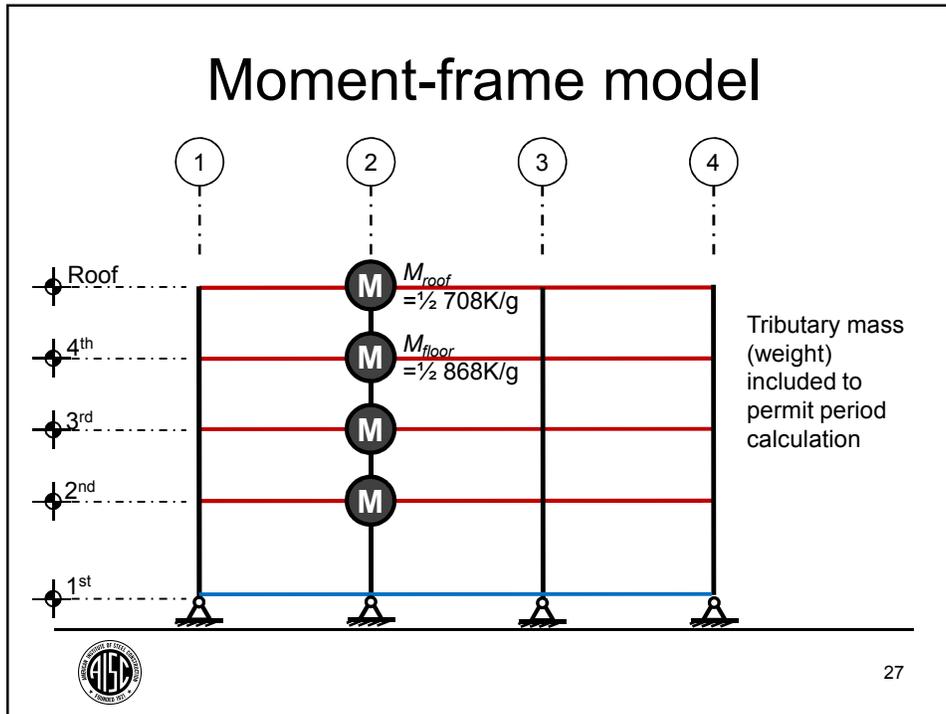


Moment-frame model



Loads from session L2 (do not include B_2 and correspond to $C_u T_a$)





- ### Member selection
- Takes care of some checks
 - Column & beam compactness
 - Highly ductile members per AISC 341 §E3.5
 - Prequalification limits per AISC 358 §5.3
 - Anticipates some requirements
 - Strong-column/weak-beam AISC 341 §E3.4a
 - Potential need for column reinforcement
 - Continuity plates
 - Doublers
- 28



Member selection

- Beam
 - Maximum depth = $(360'' - 14'')/7 = 50''$
 - W36 maximum nominal depth
 - 300 PLF maximum
 - $t_f \leq 1\frac{3}{4}''$
 - Prefer $r_y \geq 2''$
 - For bottom-flange brace spacing $\geq 8'$
 - Maximum $L_b \sim 50r_y$ for $R_y F_y = 55\text{ksi}$
 - AISC 341 §D1.2b



AISC 358 §5.3

29

Member Selection

- Compactness
 - Use SDM Table
 - Note that RBS beam compactness is measured at the $1/6$ point of the cut
 - Additional sections not indicated in the table may work

$$b_{ef} = 2(R - c) + b_f - 2 \sqrt{R^2 - \left(\frac{b}{3}\right)^2}$$



Assume $b_{ef} = 0.8b_f$ for member selection; check later

30

Table 1-3 (continued)
Sections That Satisfy Seismic Width-to-Thickness Requirements
 $F_y = 50 \text{ ksi}$
W-Shapes

Shape	IMF		SMF		STMF	SCCS	OCBF	SCBF		$L_b \text{ max, ft}$	
	Beams and Columns	Beams and Columns	Chord	Chord	Columns	Diagonal Braces	Diagonal Braces	Columns	Beams	λ_{inf}	λ_{inf}
W14x730	•	•	•	•	•	•	•	•	•	19.5	38.5
>257	•	•	•	•	•	•	•	•	•	17.2	33.9
>233	•	•	•	•	•	•	•	•	•	17.0	33.7
>211	•	•	•	•	•	•	•	•	•	16.9	33.4
>193	•	•	•	•	•	•	•	•	•	16.8	33.3
>176	•	•	•	•	•	•	•	•	•	16.7	33.0
>159	•	•	•	•	•	•	•	•	•	16.6	32.9
>145	•	•	•	•	•	•	•	•	•	16.5	32.7
W14x132	•	•	•	•	•	•	•	•	•	15.6	30.9
>120	•	•	•	•	•	•	•	•	•	15.5	30.7
>109	•	•	•	•	•	•	•	•	•	15.5	30.6
W14x82	•	•	•	•	•	•	•	•	•	10.3	20.4
>74	•	•	•	•	•	•	•	•	•	10.3	20.4
>68	•	•	•	•	•	•	•	•	•	10.2	20.2
>61	•	•	•	•	•	•	•	•	•	10.2	20.1
W14x53	•	•	•	•	•	•	•	•	•	7.98	15.8
>48	•	•	•	•	•	•	•	•	•	7.94	15.7
>43	•	•	•	•	•	•	•	•	•	7.86	15.5
W14x38	•	•	•	•	•	•	•	•	•	6.44	12.7
>34	•	•	•	•	•	•	•	•	•	6.36	12.6
>30	•	•	•	•	•	•	•	•	•	6.19	12.2

Member Selection

- Strong-Column/ Weak-Beam (SCWB)

- $\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1$

- Assume $\frac{\sum Z_c}{\sum Z_{RBS}} > 1.5$

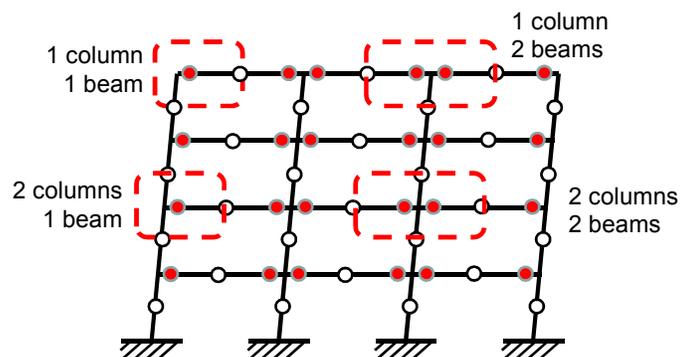
- 3.0 makes column reinforcement less likely
 - Not applicable to deep columns
 - RBS beam reduction
 - 30% assumed
 - SCWB checked later



AISC 341 §E3.4a

31

Beam and column numbers



32

Strong-column/Weak-beam

P19 $=P\$2*\$A\$2/(\$D19*\$B\$2*\$B\$3)$

	A	B	C	D	M	N	O	P	Q	R
1	Columns	Beams	Designation		W14X193	W14X176	W14X159	W14X145	W14X132	W14X120
2	2	2	Z _x		355	320	287	260	234	212
3	Beam %	70%	Designation	Z _x						
34			W27X114	343	1.48	1.33	1.20	1.08	0.97	0.88
35			W27X102	305	1.66	1.50	1.34	1.22	1.10	0.99
36			W27X94	278	1.82	1.64	1.47	1.34	1.20	1.09
37			W27X84	244	2.08	1.87	1.68	1.52	1.37	1.24
32			W24X103	280	1.81	1.63	1.46	1.33	1.19	1.08
33			W24X94	254	2.00	1.80	1.61	1.46	1.32	1.19
34			W24X84	224	2.26	2.04	1.83	1.66	1.49	1.35
35			W24X76	200	2.54	2.29	2.05	1.86	1.67	1.51
36			W24X68	177	2.87	2.58	2.32	2.10	1.89	1.71
37			W24X62	153	3.31	2.99	2.68	2.43	2.18	1.98
38			W24X55	134	3.78	3.41	3.06	2.77	2.49	2.26
70			W21X68	160	3.17	2.86	2.56	2.32	2.09	1.89
71			W21X62	144	3.52	3.17	2.85	2.58	2.32	2.10
72			W21X55	126	4.02	3.63	3.25	2.95	2.65	2.40



33

Preliminary design

- Hand methods (see Naeim, assumptions added)

$$\Delta_i = C_d \frac{V_i h_i^2}{12} \left[\frac{1}{\sum \frac{EI_{ci}}{h_i} + \sum \frac{EI_{bi}}{L}} \right] = 0.02 h_i$$

- Assume typical $I_{ci} = 2I_{bi}$

- End column $I_{ci} = I_{bi}$

$$\Delta_i = \frac{V_i h_i}{12EN} \frac{C_d}{0.02} \left[\frac{h_i}{2} + L \right] \text{ for } N \text{ bays}$$

- Reduces iterations



- But with computers iterations are easy

34

Preliminary design

- Computer methods
 - Model with arbitrary member size
 - Increase or decrease stiffness to achieve proper drift
 - Use seismically compact members
 - Keep an eye on SC/WB



35

Iteration (computer analysis)

- Period changes with member changes
- Base shear changes with period
 - Use model to determine 1st-mode period T_1
 - Typically much larger than $C_u T_a$
 - Reduces forces greatly
- Modify scaling with each iteration
 - Each iteration permits reduction in forces until model period matches period used to derive forces
- Or use a response spectrum in analysis software



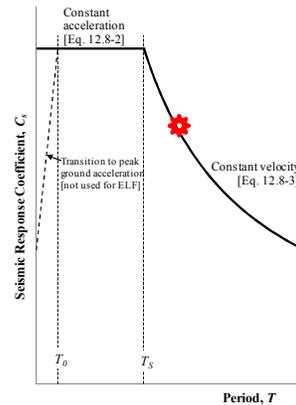
36

Iteration (computer analysis)

$$C_u T_a = 0.92s$$

T	C_s	V_{drift}	%V
0.90	0.083	276	102%
0.95	0.079	262	97%
1.00	0.075	248	92%
1.05	0.071	237	87%
1.10	0.068	226	83%
1.15	0.065	216	80%
1.20	0.063	207	76%
1.25	0.060	199	73%
1.30	0.058	191	71%
1.35	0.056	184	68%
1.40	0.054	177	66%
1.45	0.052	171	63%
1.50	0.050	166	61%

$T_1 = 1.53s$ (final iteration)



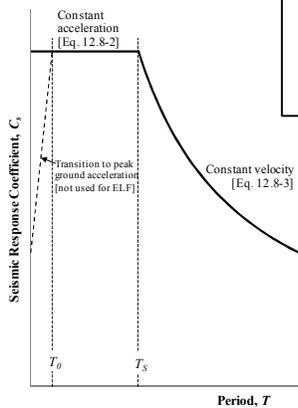
37

Drift-determined period

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

Adapted from Naeim's
Seismic Design Handbook, 3.3.7

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



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

$\frac{2}{3}$ is an approximate correction factor

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

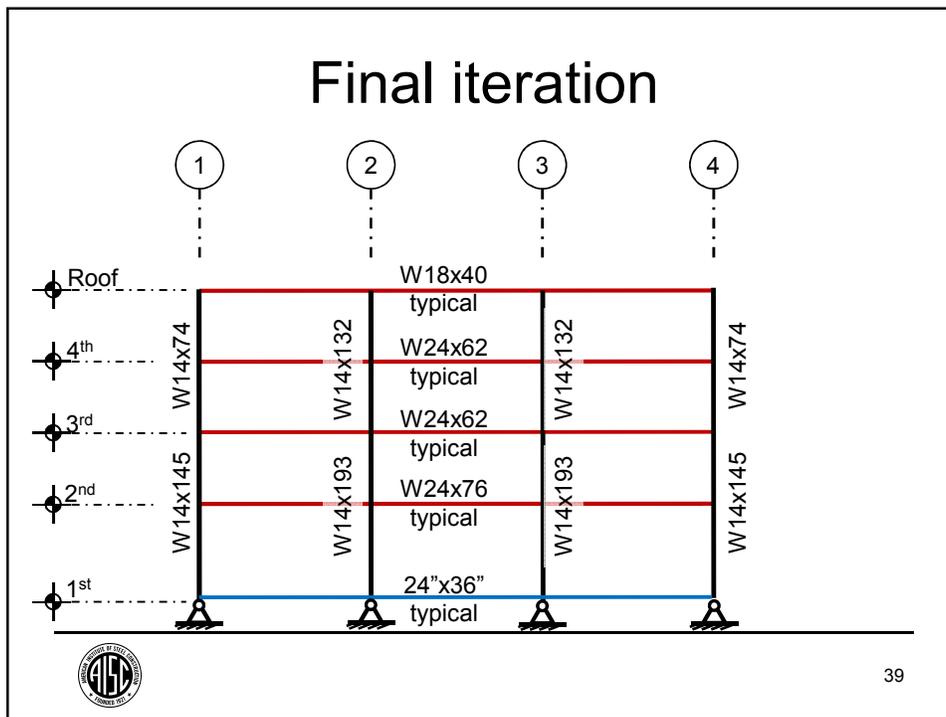
$$T = \frac{2}{3} \frac{0.02h 4\pi^2}{S_{D1}g(sec)} = \frac{2}{3} \frac{0.025h(sec)}{S_{D1}(feet)}$$

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



38





RBS Modeling

- Many software packages facilitate more exact modeling of RBS stiffness
- Other methods available in journals
- Use of $I_{eff} = 0.9I_b$ typically overestimates the reduction significantly

40

Final iteration

- 1st Mode period = 1.53s
- $C_{s(Drift)} = 0.049$
- $V_{(Drift)} = 162K = 60\% V_{(Strength)}$

	Drift ratio	$\Delta = \frac{\delta_i}{h_i} = \frac{C_d \delta_{ei}}{I h_i}$
Story4	0.014	
Story3	0.019	
Story2	0.018	
Story1	0.018	



41

Final iteration

- Stiffness reduced compared to what was assumed in determining B_2
- Recalculate B_2
- Amplify displacements by final B_2



42

Second Order Effects: SMF

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

$$B_2 = \frac{1}{1 - \frac{0.02 P_{Story}}{0.93 V_{drift}}}$$

	B_2
Roof	1.03
4 th	1.04
3 rd	1.05
2 nd	1.06

- Drift-controlled
 - $\Delta = 0.02h$
- Use $P-\Delta$ gravity load
 - $P = 1.0D + 0.5L$
- Assume $P_{mf} = \frac{1}{2} P_{story}$
- $B_2 = 1.06$ (largest)
 - Use ELM
 - $K=1$



AISC 360 Appendix 8

43

Final iteration

- 1st Mode period = 1.53s
- $C_{s(Drift)} = 0.049$
- $V_{(Drift)} = 162K = 60\% V_{(Strength)}$

	B_2 * Drift ratio	
Story4	0.0143	
Story3	0.0197	
Story2	0.0193	≤ 0.02 OK
Story1	0.0193	



$B_2 \leq 1.1$ need not be considered but was for procedural consistency

44

Stability check (ASCE 7 12.8.7)

- $\theta_i = \frac{P_{story}\Delta I_e}{V_i h_i C_d}$
 - $\theta_{max} = \frac{1}{2\beta C_d} \leq 0.25$
 - $\beta = \frac{\text{story shear demand}}{\text{story shear strength at first significant yield}}$
 - Assume $\beta = \phi = 0.9$; $\theta_{max} = \frac{1}{2(0.9)(5.5)} = 0.10$
- $\theta_1 = \frac{3313K(0.0193h)(1.0)}{162K(h)5.5} = 0.07 \text{ OK}$
 - $\frac{1}{1-\theta} = 1.08 \sim B_2$



ASCE 7 §12.8.7

45

Check irregularities

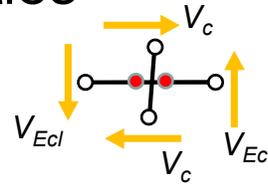
- Vertical soft story
 - Results from model
- | Level | Displacement (in.) | Shear (kips) | Stiffness (kips/in) |
|-----------------|--------------------|--------------|---------------------|
| 4 th | 0.637 | 60 | 95 |
| 3 rd | 0.860 | 113 | 132 |
| 2 nd | 0.840 | 146 | 174 |
| 1 st | 0.929 | 162 | 174 |
- Story stiffness increases with story shear
 - Irregularity not present



46

Check irregularities

- Vertical weak story
 - Beam strength
 - Assume uniform RBS reduction %



Level	Beam	M_{pr} (ft-kip)	V_{Ecl} (kip)	ΣV_c (kip)
Roof	W18x40	303	22.1	638
4 th	W24x62	591	44.0	633
3 rd	W24x62	591	44.0	633
2 nd	W24x76	772	57.5	782

- Irregularity not present



47

What's the score?

- Member selection
 - Members meet compactness requirement
 - Members meet RBS prequalification limits
- Frame meets drift limit
- Frame meets ASCE stability requirement
- ELM with $K=1$ may be used
 - No DAM stiffness reductions



48

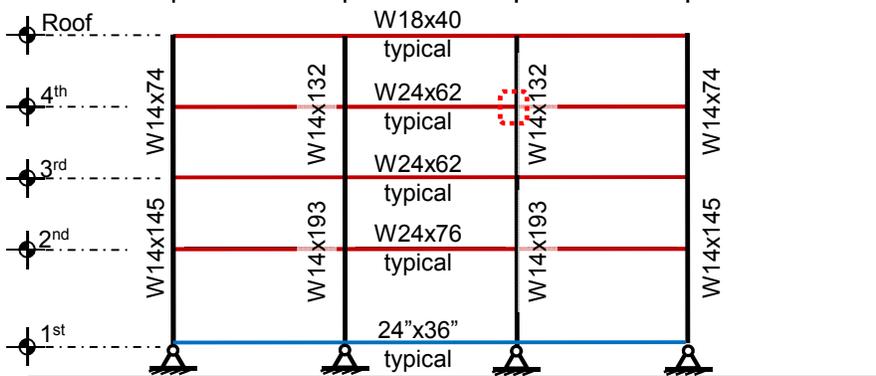
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Connection design



Connection design

	A	d	t _w	b _f	t _f	k	k _t	b/2t _f	h/t _w	I _x	Z _x
W24x62	18.2	23.7	0.430	7.04	0.590	1.63	1 1/16	5.97	50.1	1550	153
W14x132	38.8	14.7	0.645	14.7	1.03	1.09	1 9/16	7.15	17.7	1530	234



50



Connection design

- Compute beam plastic-hinge moment
- Analyze beam at formation of plastic hinges
- Compute forces on connection
- Check connection limit states
 - AISC 341 (SC/WB)
 - AISC 358 (beam checks)
 - AISC 360 (WLY, WC, FLB, PZ)
- Example follows 2010 provisions



2 W24x62 beams
2 W14x132 columns

51

Compute beam plastic-hinge moment

- Select a , b , & c
 - $a = 4''$ $= 0.57b_{bf}$ $0.5b_{bf} \leq a \leq 0.75b_{bf}$
 - $b = 16''$ $= 0.68d$ $0.65d \leq b \leq 0.85d$
 - $c = 1.5''$ $= 0.21b_{bf}$ $0.1b_{bf} \leq c \leq 0.25b_{bf}$

$$Z_{RBS} = Z_x - 2ct_{bf}(d - t_{bf}) = 112in^3 = 73\%Z_x$$

$$M_{pr} = C_{pr}R_yF_yZ_{RBS} = (1.15)(1.1)(50ksi)(112in^3) = 7090kip \cdot in$$

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \quad C_{pr} = \frac{(50ksi) + (65ksi)}{2(50ksi)} = 1.15$$

AISC 358 §5.8



2 W24x62 beams
2 W14x132 columns

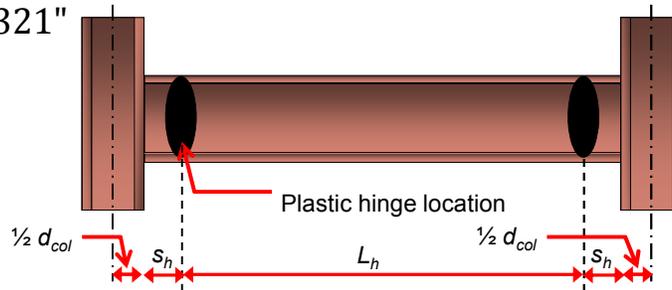
52



Beam analysis

- Geometry:

- $s_h = a + b/2 = 12"$
- $L_h = L - d_{col} - 2s_h = 360" - 14.7" - 2(12")$
- $L_h = 321"$



2 W24x62 beams
2 W14x132 columns

53

Approximate RBS beam stiffness

- 10% reduction in beam stiffness may not be valid for some cases
 - Low span-to-depth ratios
 - As low as 5 permitted for IMF
 - Thick-flange/thin-web beams
 - Built up



2 W24x62 beams
2 W14x132 columns

54

Approximate RBS beam stiffness

- 10% reduction in beam stiffness very conservative for most cases
 - High span-to-depth ratios
 - ≥ 7 required for SMF
 - Typical flange/web thickness ratios
 - Rolled WF beams
- Sabelli approximate equation:

$$\frac{I_{ef}}{I} = \frac{1}{1 + \frac{L_h}{(L_h + 2s_h)^2} \left[\frac{b}{1 - (c t_f / I)(d - t_f)^2} \right]} = 0.93$$



2 W24x62 beams
 2 W14x132 columns

55

Beam analysis

- Gravity distributed load
 - $w_u = 1.2D + 0.5L$
 - $w_u = 1.2(0.840\text{klf})D + 0.5(0.600\text{klf}) = 1.31\text{klf}$



2 W24x62 beams
 2 W14x132 columns

56

Beam analysis

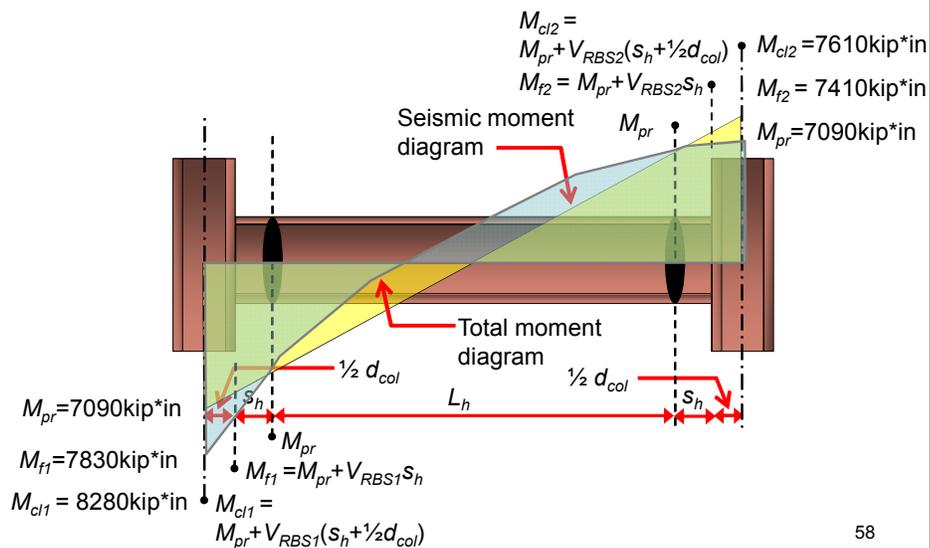
- Gravity shear
 - $V_g = \frac{1}{2} w_u L_h = \frac{1}{2}(1.31\text{klf})(321\text{in})(1\text{ft}/12\text{in})$
 - $V_g = 17.5\text{K}$
- Seismic shear
 - $V_{ecl} = 2M_{pr}/L_h = 2(7090\text{kip}\cdot\text{in})/(321\text{in})=44.1\text{K}$
- Total shear
 - $V_{RBS1} = 17.5\text{K}+44.1\text{K}=61.6\text{K}$
 - $V_{RBS2} = 17.5\text{K}-44.1\text{K}=-26.6\text{K}$ (i.e., up)



2 W24x62 beams
2 W14x132 columns

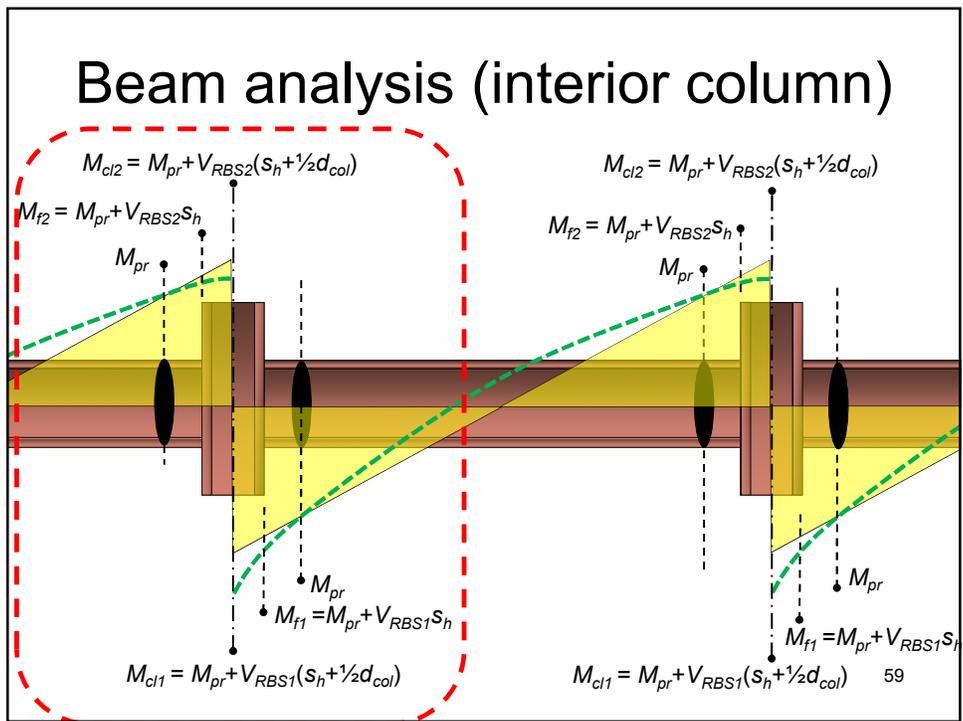
57

Beam analysis



58





Check section at column face

- Moment
 - $\phi M_n = 1.0Z_x R_y F_y$
 - $\phi M_n = 8415 \text{kip} \cdot \text{in} > 7830 \text{kip} \cdot \text{in}$ OK
- Shear
 - $\phi V_n = 306K$ Table 3-6
 - $\phi V_n = 306K > 61.6K$ OK



2 W24x62 beams
2 W14x132 columns

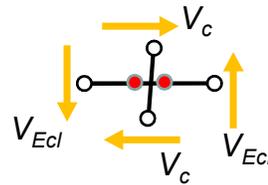
60

Strong-Column/Weak-Beam

- $\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1$

AISC 341 §E3.4a

- $M_{pc}^* = Z_c (F_y - P_u/A) + V_c d_b/2$
 - $P_u = 1.4D + 0.5L + 3.0E_h = 249K$
 - $V_c = \frac{2V_{ecl}L/2}{1/2h_i + 1/2h_{i-1}} = 106K$
 - $M_{pc}^* = 11,500 \text{ kip}\cdot\text{in}$
 - $\sum M_{pc}^* = 22,900 \text{ kip}\cdot\text{in}$



2 W24x62 beams
2 W14x132 columns

61

Strong-Column/Weak-Beam

- $\sum M_{pb}^* = M_{cl1} + M_{cl2} = 15,900 \text{ kip}\cdot\text{in}$
- $\frac{\sum M_{pc}^*}{\sum M_{pb}^*} = \frac{22,900 \text{ kip}\cdot\text{in}}{15,900 \text{ kip}\cdot\text{in}} = 1.44 \text{ OK}$

$$\frac{\sum Z_c}{\sum Z_{RBS}} = 2.1 \quad (\text{just to gage the effectiveness of this as a predictor of SCWB ratio})$$

AISC 341 §E3.4a



2 W24x62 beams
2 W14x132 columns

62

Forces at column face

- $P_f = \frac{0.85M_f}{d_b - t_{bf}} = 288\text{K}$ AISC 341-16 E3.6f
- Web Local yielding
 - $\phi R_n = \phi(5k_c + N)F_y t_{cw}$
 - $\phi R_n = 1.0(5k_c + t_{bf})F_y t_{cw} = 282\text{K}$ NG
 - Continuity plates required for $288\text{K} - 282\text{K} = 6\text{K}$

AISC 360 §J.10



2 W24x62 beams
2 W14x132 columns

63

Forces at column face

- Web crippling
 - $\phi R_n = \phi 0.80 t_{cw}^2 \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{E F_y w t_{cf}}{t_{cw}}}$
 - $\phi R_n = 0.75 * 0.80 t_{cw}^2 \left[1 + 3 \left(\frac{t_{bf}}{d_c} \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{E F_y w t_{cf}}{t_{cw}}}$
 - $\phi R_n = 402\text{K}$, OK

AISC 360 §J.10



2 W24x62 beams
2 W14x132 columns

64

Forces at column face

- Flange local bending
 - $\phi R_n = 0.9 * 6.25 t_{cf}^2 F_{yc} = 298K$ OK
 - More stringent requirement in AISC 341-10 §E3.6f
 - $t_{cf} \geq 0.4 \sqrt{1.8 b_{bf} t_{bf} \left(\frac{F_{yb} R_{yb}}{F_{yc} R_{yc}} \right)}$ equivalent to
 - $6.25 t_{cf}^2 \geq 1.8 b_{bf} t_{bf} \left(\frac{F_{yb} R_{yb}}{F_{yc} R_{yc}} \right)$
 - $0.9 * 6.25 F_{yc} t_{cf}^2 \geq 0.9 * 1.8 / R_{yc} b_{bf} t_{bf} F_{yb} R_{yb}$
 - $R_u \sim 1.5 b_{bf} t_{bf} R_y F_{yb} = 343K$
 - Continuity plates would be required

AISC 360 §J.10



2 W24x62 beams
2 W14x132 columns

65

Additional AISC 341 check

- $t_{cf} \geq b_{bf} / 6$
- $t_{cf} = 1.03in$
- $b_{bf} / 6 = 7.04in / 6 = 1.17in > 1.03in$, NG
 - Continuity plates required

AISC 341 §E3.6f



2 W24x62 beams
2 W14x132 columns

66

Panel Zone

- $V_u = \frac{M_{f1} + M_{f2}}{d_b} - V_c = 643K - 106K = 537K$
- $\phi V_n = 1.0F_y \left[0.6t_{cw}d_c + \frac{1.8b_{cf}t_{cf}^2}{d_b} \right]$ AISC 341 E3.6e(1)
- $\phi V_n = 285K + 59K = 344k$
- Doubler required
 - $537K - 344K = 193K$
 - $193K \frac{t_{cw}}{285k} = 0.44"$, 1/2" doubler required



2 W24x62 beams
2 W14x132 columns

67

Optimization

There's always a solution in steel.



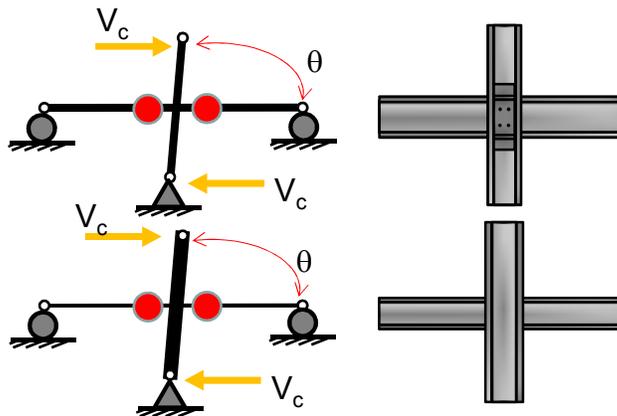
Optimization

- Current design works
 - Meets drift limit
 - Connection can meet all limit states
 - Requires continuity plates
 - Requires doubler
- Investigate design of equivalent stiffness
 - Heavier column
 - Lighter beam
 - Eliminate reinforcement



69

Elimination of doublers and continuity plates



For RBS:
$$I_{B2} \geq \frac{1}{0.9 \frac{N_c}{N_b} \frac{h}{L} (\Delta_{all}/\Delta I_{C1} - 1/I_{C2}) + \Delta_{all}/\Delta I_{B1}}$$

Elimination of doublers and continuity plates

<ul style="list-style-type: none"> • Try W14x145 <ul style="list-style-type: none"> ○ $I_{b2} \geq 1460\text{in}^4$ ○ W24x62 <ul style="list-style-type: none"> • $I_b = 1550\text{in}^4$ ○ Continuity plates ○ 5/16" doubler 	<ul style="list-style-type: none"> • Try W14x159 <ul style="list-style-type: none"> ○ $I_{b2} \geq 1420\text{in}^4$ ○ W24x62 <ul style="list-style-type: none"> • $I_b = 1550\text{in}^4$ ○ No continuity plates ○ 3/16" doubler 	<ul style="list-style-type: none"> • Try W14x176 <ul style="list-style-type: none"> ○ $I_{b2} \geq 1360\text{in}^4$ ○ W24x55 <ul style="list-style-type: none"> • $I_b = 1350\text{in}^4$ • Check drift! ○ No continuity plates ○ No doubler
-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------	--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------	-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------

<u>Frame:</u>	<u>Original design:</u>
$N_c = N_b = 2$	W24x62: $I_{b1} = 1550\text{in}^4$
$h = 12.5'$	W14x132: $I_{c1} = 1530\text{in}^4$
$L = 30'$	$\Delta_{all}/\Delta = 1.02$

For RBS: $I_{B2} \geq \frac{1}{0.9 \frac{N_c h}{N_b L} (\Delta_{all}/\Delta I_{C1} - 1/I_{C2}) + \Delta_{all}/\Delta I_{B1}}$

Optimization

<ul style="list-style-type: none"> • Original design <ul style="list-style-type: none"> ○ W24x62 beams $\frac{Z_c}{Z_{RBS}} = 2.1$ ○ W14x132 columns ○ 1.9% drift ○ 4th floor <ul style="list-style-type: none"> • Continuity plates • Doubler plate ○ Roof (not shown) <ul style="list-style-type: none"> • Continuity plates • No doubler plate 	<ul style="list-style-type: none"> • Redesign (3 bays) $\frac{Z_c}{Z_{RBS}} = 3.2$ <ul style="list-style-type: none"> ○ W24x55 beams ○ W14x176 columns ○ 2.0% drift ○ 1230# more steel <ul style="list-style-type: none"> ○ +\$1230 ○ No continuity plates <ul style="list-style-type: none"> • 32 eliminated: <ul style="list-style-type: none"> ○ -\$3200 ○ No doubler plates <ul style="list-style-type: none"> • 2 eliminated: <ul style="list-style-type: none"> ○ -\$600
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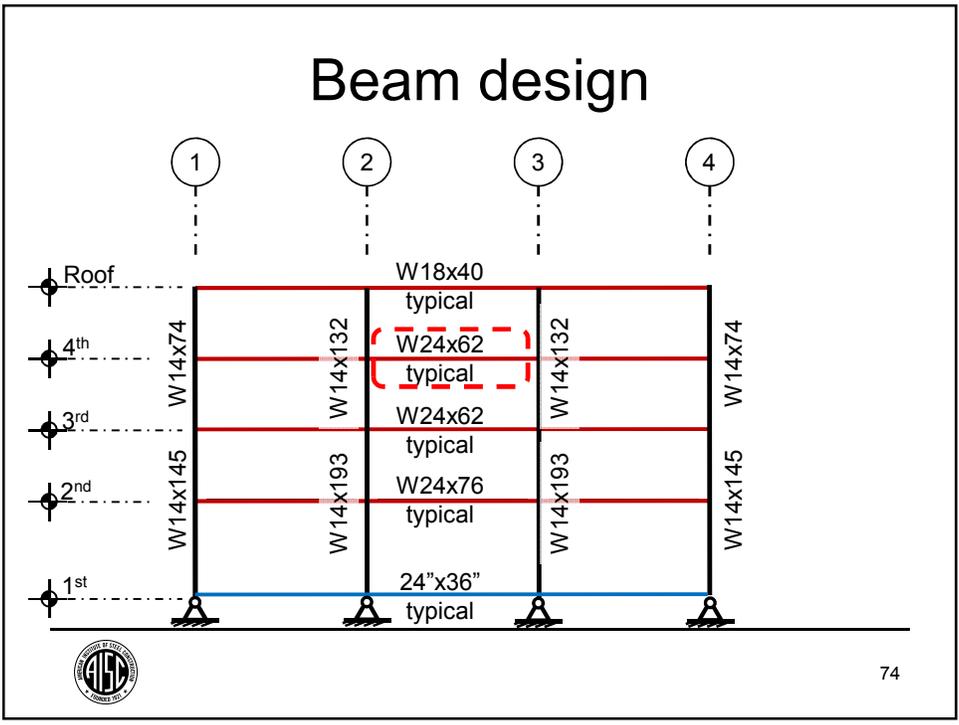
\$2570 savings

72



There's always a solution in steel.

Beam strength design



Beam design

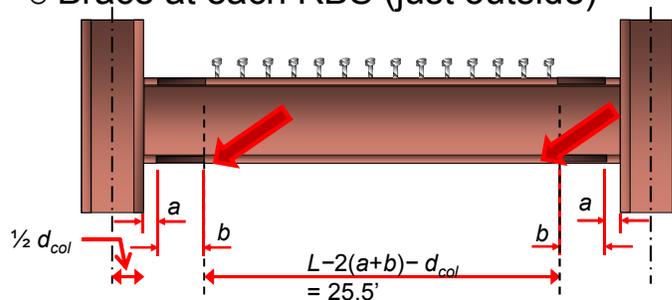
- 4th floor beam
- W24x62 (original design)
- V_u superseded by capacity design
- $P_u = 6K$ (can be neglected)



75

Beam design

- Lateral bracing
 - Top flange continuously braced by deck
 - Brace at each RBS (just outside)



AISC 341 §E3.4b

76

Beam design

- Lateral bracing for *highly ductile members*
 - $L_b \leq 0.086 \frac{r_y E}{F_y} = 0.095 \frac{r_y E}{R_y F_y} = 50r_y = 50(1.38") = 5.76'$
 - Brace @ $25.5'/5=5.1'$
- Alternative: use W21x73
 - $I=1600\text{in}^4$ (compared to 1550in^4 for W24x62)
 - $50r_y = 50(1.81") = 7.5'$
 - Brace @ $25.5'/4=6.4'$
 - Redesign connection

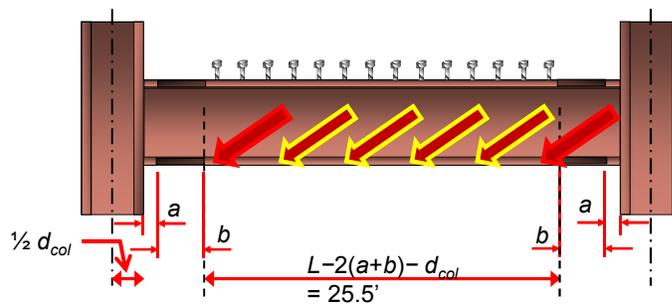


AISC 341 §D1.2b

77

Beam design

- Lateral bracing



78

Beam design

- 4th floor beam
- W24x62 (original design)
- $M_u = 1.4D + 0.5L + 1.0E_h$
 - $M_u = 1.4D + 0.5L + 1.0(B_2 162'K)$
 - $w_u = 1.4(0.840\text{klf}) + 0.5(0.600\text{klf}) = 1.48\text{klf}$
 - At column centerline
 - $M_u = w_u (30\text{ft})^2/12 + 1.0([1.05]162'K) = 281'K$
 - At RBS: $\phi M_n = 0.9 F_y Z_{RBS} = 5040''K = 420'K$ OK



79

Beam design

- At $L_b > L_p$ (1st segment outside RBS)
 - $M_u \approx 281'K \frac{180'' - d_c/2 - s_h}{180''} = 0.89M_u = 250'K$
 - $C_b = 1.2$ (see SDM Example 4.3.3 for method)
 - $\phi M_n = 570'K$
 - for $L_b = 5.5'$ and $C = 1.0$ (Table 3-10)
 - For $C_b = 1.2$, $1.2 * 570'K = 684'K$
 - $\phi M_n = 0.9 F_y Z_x = 574'K$ OK



80

Beam design

C_b

M_1/M_2

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C}$$

For straight-line moment diagram (no distributed load)

$M_1/M_2 \sim 1 - 2L_b/L_h$

AISC 360 §F.1

81

There's always a solution in steel.

Column strength design

Column design

	A	d	t _w	b _f	t _f	k _r	W	b/2t _f	h/t _w	I _x	Z _x
W14x145	42.7	14.8	0.680	15.5	1.09	1 9/16	145	7.11	16.8	1710	260

83

Column design

- Case 1
 - Combo M-BLC-1
 - $1.4D + 0.5L + 1.0E_h$
 - By inspection compression controls
 - Axial and flexure considered
 - B_1 and B_2 apply

- Case 2
 - Combo M-OLC-1
 - $1.4D + 0.5L + 3.0E_h$
 - By inspection compression controls
 - Axial in the absence of flexure considered
 - B_2 applies

AISC 341 §D1.4a
84



Column design

- Shear
 - $V_D = 3.2K$
 - $V_L = 1.5K$
 - $V_E = B_2 25.9K$
 $= 1.07(25.9K)$
 $= 27.7K$
 - Shear strength OK by inspection
- Axial
 - $P_D = 151.8K$
 - $P_L = 60.8K$
 - $P_E = B_2 51.6K$
 $= 55.2K$
- Moment
 - $M_D = 29.7'K$
 - $M_L = 13.6'K$
 - $M_E = B_2 200'K$
 $= 214'K$



85

Column design (case 1)

- Combo M-BLC-1
 - $1.4D + 0.5L + 1.0E_h$
- Axial
 - $P_u = 1.4(151.8K) + 0.5(60.8K) + (55.2K) = 298K$
- Moment
 - $M_u = 1.4(29.7'K) + 0.5(13.6'K) + (214'K) = 262'K$

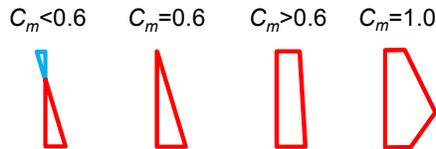


86

Column design (case 1)

- $B_1 = \frac{C_m}{1 - P_u/P_{E1}} \geq 1$

- Assume $C_m=0.6$ (for triangular moment diagram)



- $P_{E1} = \frac{\pi^2 EI}{L^2} = 17,340K$

If $C_m=0.6$

- $B_1 = 1$

$B_1 = 1$ for $P_u \leq 0.4P_{E1}$



AISC 360 Appendix 8

87

Column design (case 1)

- $\phi P_n = 1690K$ (Table 4-1)
 - $P_u/\phi P_n = 0.18$
- $\phi M_n = 975'K$ ($L_b < L_p$, Table 3-2)
- $\frac{1}{2}P_u/\phi P_n + M_u/\phi M_n = 0.36$ OK



88

Column design (case 2)

- Combo M-OLC-1
 - $1.4D + 0.5L + 3.0E_h$
- No moment
- Axial
 - $P_u = 1.4(151.8K) + 0.5(60.8K) + 3.0(55.2K)$
= 408K
 - $P_u / \phi P_n = 0.24$ OK



89

There's always a solution in steel.

Base-plate design



Base-plate design

	A	d	t_w	b_f	t_f	k_f	W	$b/2t_f$	h/t_w	I_x	Z_x
W14x145	42.7	14.8	0.680	15.5	1.09	1 9/16	145	7.11	16.8	1710	260

Base-plate design

- Overstrength forces
- Additional ductility still helpful
- Consider tension case
- Follow Design Guide 1 method
 - Example 4.4.3 in SDM
 - “Base plate with large moment” method

92

Base-plate design

- Shear
 - $V_D = 3.2K$
 - $V_L = 1.5K$
 - $V_E = B_2 25.9K$
 $= 1.07(25.9K)$
 $= 27.7K$
- Axial
 - $P_D = 151.8K$
 - $P_L = 60.8K$
 - $P_E = B_2 51.6K$
 $= 55.2K$
- Moment
 - $M_D = 12.8'K$
 - $M_L = 5.8'K$
 - $M_E = B_2 196'K$
 $= 209'K$



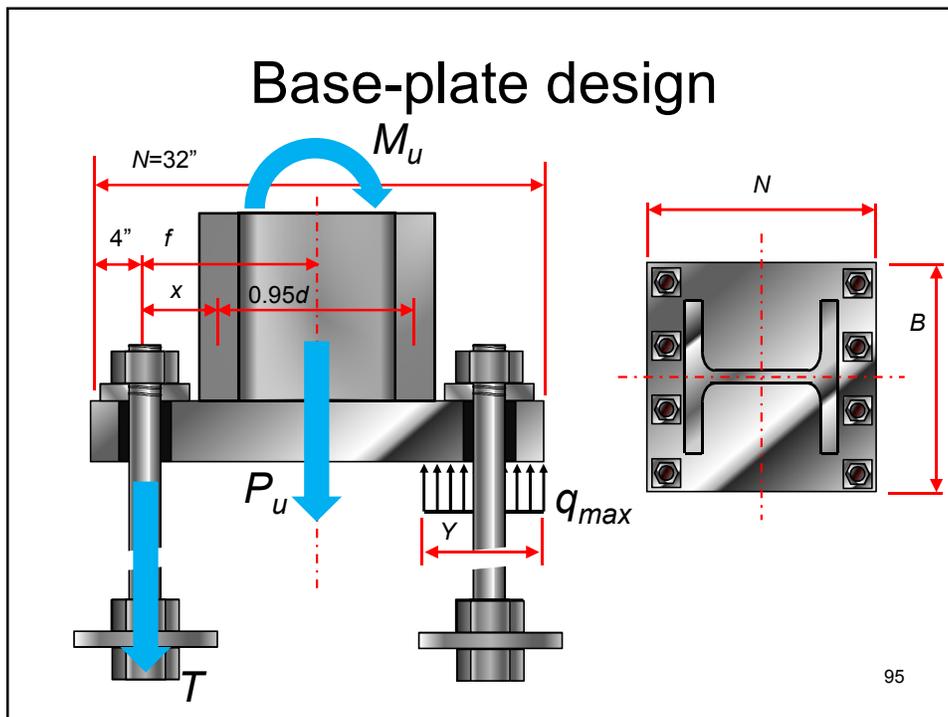
93

Base-plate design

- $R_u = 0.7D + 3.0E_h$
 - combo M-OLC-2 (session L1)
- $V_u = 85.3K$
- $P_u = 59.3K$ Tension
- $M_u = 635'K$
 - Column expected strength:
 - $1.1R_y F_y Z_x = 1310'K$ D2.6c(b)



94



Base-plate design

- $f_{pmax} = \phi 0.85 f'_c \sqrt{\frac{A_2}{A_1}} =$
 $(0.65)0.85(4ksi)2 = 4.42ksi$
- $q_{max} = f_{pmax}B = 4.42ksi(32") = 141K/in$
- $e = \frac{M_u}{P_u} = \frac{635'K}{-59.3K} = -128"$



AISC Design Guide 1 §3.4

96

Base-plate design

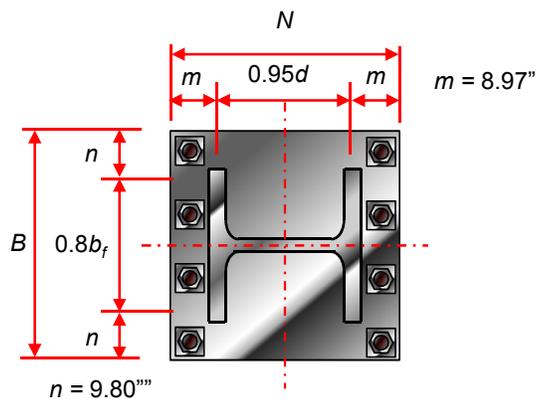
- $Y = \left[f + \frac{N}{2} \right] \pm \sqrt{\left(f + \frac{N}{2} \right)^2 - \frac{2P_u(e+f)}{q_{max}}}$
- $Y = \left[12" + \frac{32"}{2} \right] \pm \sqrt{\left(12" + \frac{32"}{2} \right)^2 - \frac{2(-59.3K)(-128"+12")}{141K/in}}$
- $Y = 54.2", \underline{1.81"}$
- $T = q_{max} Y + P_u = 141K/in * 1.81" + 59.3K = 314K$



78.6K/rod AISC Design Guide 1 §3.4

97

Base-plate design



98

Base-plate design

- Flexure at the bearing side

$$t_p \geq 2.11 \sqrt{\frac{f_{pmax} Y [\max(m,n) - Y/2]}{F_y}}$$

$$t_p \geq 2.11 \sqrt{\frac{4.42 \text{ksi} (1.81") [9.80" - 1.81"/2]}{50 \text{ksi}}} = 2.51"$$

- Use 2³/₄" plate



AISC Design Guide 1 §3.4

99

Base-plate design

- Flexure at the tension side

- $x = f - d/2 + t_f/2 = 5.15"$

- $t_p \geq 2.11 \sqrt{\frac{T x}{B F_y}} = 2.1"$ Use 2³/₄" plate

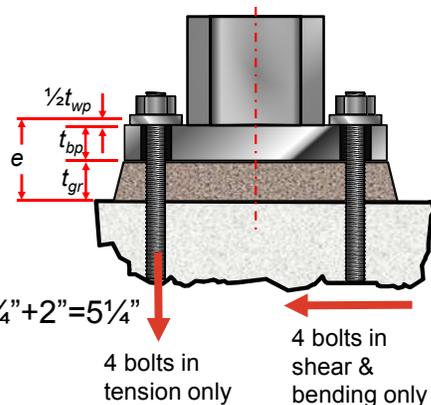


AISC Design Guide 1 §3.4

100

Anchor-rod design

- Shear
 - $V_u = 85.3\text{K}/4\text{bolts}$
 $= 21.4\text{K}/\text{bolt}$
- Moment
 - $e = \frac{1}{2} t_{wp} + t_{bp} + t_{gr}$
 - $e = \frac{1}{2}(1\text{''})[\text{assumed}] + 2\frac{3}{4}\text{''} + 2\text{''} = 5\frac{1}{4}\text{''}$
 - $M_u = \frac{1}{2} V_u e$
 $= 56.2\text{''K}$



101

Anchor-rod design (tension)

- Check anchor rod
 - Loads
 - $M_u = 0\text{''K}$
 - $T_u = 78.6\text{K}$
 - $V_u = 0\text{K}$
 - Properties
 - $d = 1\frac{1}{2}\text{''}$
 - $A = 1.77\text{in}^2$
 - $A_{net} = 1.41\text{in}^2 = 0.795A$
 - Table 7-17
 - $Z = 0.563\text{in}^3$
- Stresses
 - $F_{nt} = 0.795F_u = 59.7\text{ksi}$
 - $F_{nv} = 0.45F_u = 33.8\text{ksi}$
 - $f_v = \frac{V_u}{A} = 0\text{ksi}$
 - $f_a = \frac{T_u}{A} = 44.5\text{ksi}$
 - $f_b = \frac{M_u}{Z} = 0\text{ksi}$
 - $f_t = f_a + f_b = 44.5\text{ksi}$
 - $F'_{nt} = \left[1.3 - \frac{f_v}{\phi F_{nv}}\right] F_{nt} \leq F_{nt}$
 - $\phi F'_{nt} = (0.75)59.7\text{ksi} = 44.7\text{ksi}$
 - $f_t \leq \phi F'_{nt}$ OK



Note: $F_{nt} = 0.75 F_u$ accounts for effective net area approximately. This method does so explicitly.

AISC 341 §J3.7 102

Anchor-rod design (shear)

- Check anchor rod
 - Loads
 - $M_u = 56.2\text{K}$
 - $T_u = 0\text{K}$
 - $V_u = 21.4\text{K}$
 - Properties
 - $d = 2\text{''}$
 - $A = \pi\text{in}^2$
 - $A_{net} = 2.50\text{in}^2 = 0.795A$
 - Table 7-17
 - $Z = 1.33\text{in}^3$
- Stresses
 - $F_{nt} = 0.795F_u = 59.7\text{ksi}$
 - $F_{nv} = 0.45F_u = 33.8\text{ksi}$
 - $f_v = \frac{V_u}{A} = 3.4\text{ksi}$
 - $f_a = \frac{T_u}{A} = 0\text{ksi}$
 - $f_b = \frac{M_u}{Z} = 42.1\text{ksi}$
 - $f_t = f_a + f_b = 42.1\text{ksi}$
 - $F'_{nt} = \left[1.3 - \frac{f_v}{\phi F_{nv}}\right] F_{nt} \leq F_{nt}$
 - $\phi F'_{nt} = (0.75)59.7\text{ksi} = 44.7\text{ksi}$
 - $f_t \leq \phi F'_{nt}$ OK



AISC 341 §J3.7

103

Anchor-rod design

- Rods designed for tension only:
 - 1½" diameter
 - Utilizes calculated net section
- Rods designed for tension on one side, shear and bending on the other
 - 2" diameter
 - Rods are not a good bending element
- Rods designed for tension, shear, and bending
 - 2¼" diameter



104

Anchor-rod design

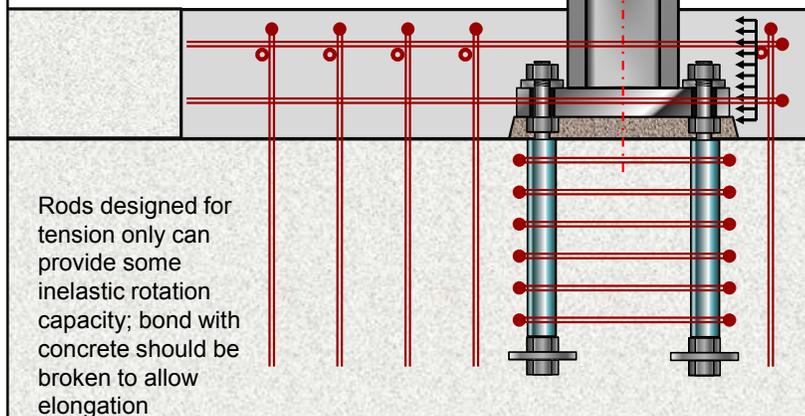
- Shear alternatives (no moment in rods)
 - Bearing of base plate and column
 - OK for base plates embedded in slabs
 - Concrete reinforcement to complete load path
 - $f_p = \phi 0.85 f'_c = (0.65)0.85(4ksi) = 2.2ksi$
 - $f_p B t_p = 2.2ksi * 32" * 3" = 211K$
 - Friction
 - $C_\mu = (141K/in * 1.81")(0.4) = 102K$ ACI 349



105

Anchor-rod design

Shear alternatives



106



Anchor-rod design

Table 14-2
Recommended Maximum Sizes for
Anchor-Rod Holes in Base Plates Increase
for grade
55

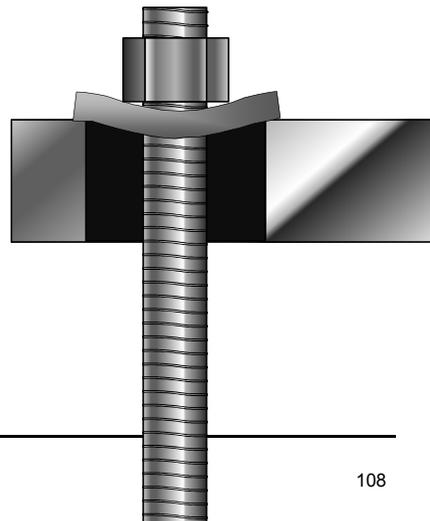
- Use (8) 1½" Grade 55 anchor rod F1554, S1

Anchor Rod Diameter, in.	Hole Diameter, in.	Min. Washer Size, in.	Min. Washer Thickness
1½	2 ⁵ / ₁₆	3½	5/8
1¾	2 ³ / ₄	4	7/8
2	3¼	5	¾
2½	3¾	5½	7/8

- Notes:
1. Circular or square washers meeting the washer size are acceptable.
 2. Clearance must be considered when choosing an appropriate anchor rod hole location, noting effects such as the position of the rod in the hole with respect to the column, weld size and other interferences.
 3. When base plates are less than 1¼ in. thick, punching of holes may be an economical option. In this case, ¾-in. anchor rods and 1½-in.-diameter punched holes may be used with ASTM F844 (USS Standard) washers in place of fabricated plate washers.

Anchor-rod design

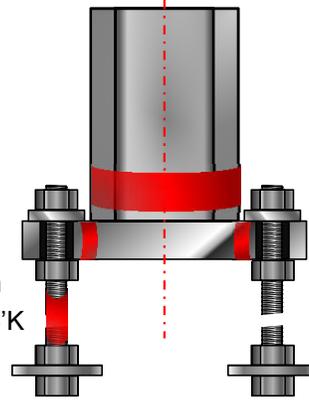
- Unpublished recommendations to prevent plate dishing:
 - Grade 36
 - Use Table 14-2
 - Grade 55
 - Use $t=0.4d_{rod}$
 - Grade 105
 - Use $t=0.69d_{rod}$



108

Base connection design

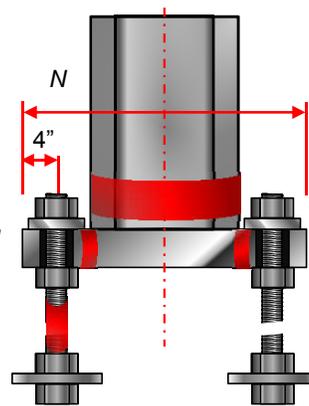
- Potential sources of base inelastic rotation
 - Column inelastic rotation
 - Significant overstrength
 - Affects column
 - Anchor-rod elongation
 - Compression increases overstrength
 - $M_c = P_u(N - 2 \cdot 4'') / 2 = 298K(2') / 2 = 298'K$
 - Base plate flexure
 - Compression increases overstrength
 - Foundation rotation



109

Base connection design

- Column strength
 - $R_y F_y Z_x = 1191'K$
- Anchor rods
 - $M_y \sim \sum A d F_y \sim \sum A (N - 2 \cdot 4'') F_y + M_c$
 - $M_y \sim 1060'K$ (1½" rods); 1340'K @ F_u
 - $M_y \sim 1660'K$ (2" rods); 2170'K @ F_u
- Base plate
 - $M_y \sim [M_{p,plate} / 4''] (N - 2 \cdot 4'') + M_c$
 - $M_{p,plate} = F_y B t_p^2 / 4 = 3025''K$
 - $M_y \sim 1810'K$



110

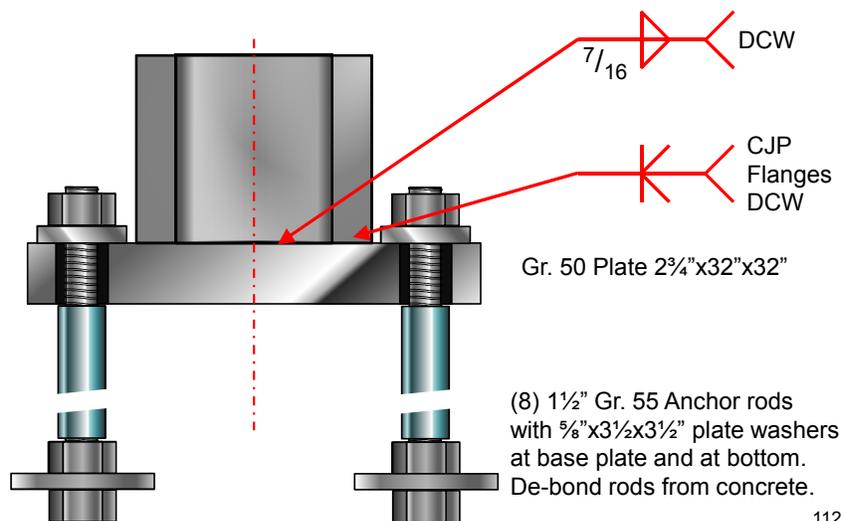
Base connection design

- M-OLC-2
 - $M_u = 0.9D + E_v + \Omega_o E_h = 652'K$
- Use 1½" anchor rods
- Design column connection to develop column strength
 - CJP flanges
 - Web
 - Use 7/16" weld = 0.64t_w



111

Base connection design



112

There's always a solution in steel.

Completion of design



Completion of design

- Connection
 - Indicate DCW
 - Indicate weld tab removal
 - Indicate steel backing requirements
- Beam
 - Indicate protected zone
 - Design lateral braces
- Column
 - Design column splice



114

Completion of design

- Base plate
 - Provide load path for shear
 - Design concrete anchorage/force transfer to foundation
 - Design grade beam for ductile rotation
- Specifications
 - Specify weld toughness requirements
 - Specify Quality Assurance Plan



115

Summary

There's always a solution in steel.



Summary

- SMF tend to be governed by drift
 - Design for drift first
 - Conservative assumptions in strength design do not generally affect design
 - $C_b = 1$
 - $C_m = 1$
 - Combine maxima in lieu of performing multiple calculations
- Selecting heavier columns initially reduces reinforcement



117

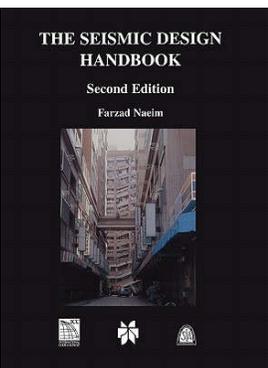
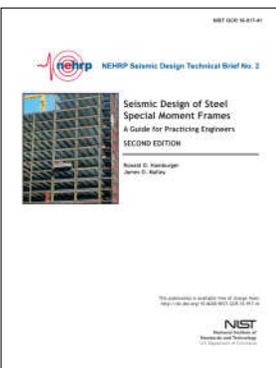
End of Session L3

Next:
**Design of the
Braced frames**

There's always a solution in steel.

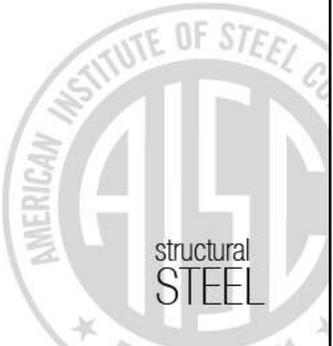


Additional resources



Question time

There's always a solution in 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.

8-Session Package Registrants Course Resources

Go to www.aisc.org and sign in.



8-Session Package Registrants Course Resources



The screenshot shows the AISC website navigation menu with links for EDUCATION, PUBLICATIONS, STEEL SOLUTIONS CENTER, AWARDS AND COMPETITIONS, and TECHNICAL RESOURCES. The breadcrumb trail reads: AISC > MY ACCOUNT > COURSE RESOURCES.

Course Resources

Event	Start Date
Test Course	8/1/1900 12:00:00 AM
16.15 8-Session Package Night School 15 - Fundamentals of Connection Design	20/18/2017 7:00:00 PM
16.16 8-Session Package Night School 16 - Seismic Design in Steel	2/9/2018 7:00:00 PM
16.17 8-Session Package Night School 17 - Design of Exposed Attachments	7/16/2018 7:00:00 PM
8-Session Package - Seismic Design in Steel - Contents & Exams	7/16/2018 1:30:00 PM

8-Session Package Registrants Course Resources



The screenshot shows the AISC website navigation menu with links for EDUCATION, PUBLICATIONS, STEEL SOLUTIONS CENTER, AWARDS AND COMPETITIONS, and TECHNICAL RESOURCES. The breadcrumb trail reads: AISC > MY ACCOUNT > COURSE RESOURCES > SEISMIC DESIGN IN STEEL.

Seismic Design in Steel

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Videos	Quiz	Attendance
R1 Introduction To Effective Seismic Design	N/A	Handouts	Video Pascode 18H0F31	Pass Score 100	N/A
R2 Seismic Design Concepts - Moment Frames	N/A	Handouts	Video Pascode 18Q21218	Pass Score 100	N/A
R3 Seismic Design Concepts - Braced Frames	N/A	Handouts	Video Pascode 38103005	Pass Score 100	N/A
R4 Seismic Design Concepts - Design	N/A	Handouts	Video Pascode 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
Seismic Design in Steel - Final Exam	Oct 3 2018 12:00AM EDT	Handouts	Available 10/03/2018 5:00PM EDT	Available 10/03/2018 5:00PM EDT	



There's always a solution in steel.

Thank You

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

