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## Course Description

### Session 7: Moment Frames

**April 2, 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.



There's always a solution in steel.

## Seismic Design in Steel: Concepts and Examples

Session 7: Design of the Moment Frames  
April 2, 2018



Rafael Sabelli, SE



## Course objectives

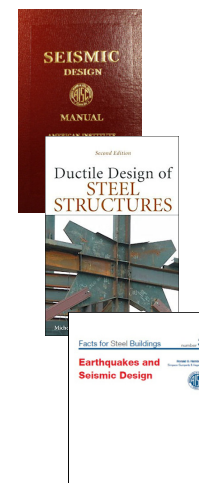
- Understand the principles of seismic design of steel structures.
- Understand the application of those principles to two common systems:
  - Special Moment Frames
  - Buckling-Restrained Braced Frames.
- Understand the application of design requirements for those systems.



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## Resources

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



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## Other resources

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



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## Course outline

### Part I: Concepts

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



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## Course outline

### Part II: Application

- 5. Planning the seismic design
- 6. Building analysis and diaphragm design
- 7. Design of the moment frames**
- 8. Design of the braced frames



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There's always a solution in steel.

## Session 7: 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



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## 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 6
    - Member selection
      - Prequalification limits
      - Seismic compactness ("highly ductile member")
      - Strong-column/Weak-beam proportioning



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## 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



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## Prequalification issues

There's always a solution in steel.



## 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



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## RBS Prequalification limits

- Beam
  - I-shaped
  - W36 max.
  - 300 PLF max.
  - $t_f \leq 1\frac{3}{4}$ "
  - Span/depth  $\geq 7$
  - 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

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### Reduced Beam Section (RBS)

$0.5b_{bf} \leq a \leq 0.75b_{bf}$   
 $0.65d \leq b \leq 0.85d$   
 $0.1b_{bf} \leq c \leq 0.25b_{bf}$

$R = \frac{4c^2 + b^2}{8c}$

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

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

$b_{ef} @ \frac{2}{3}b$

AISC 358 §5.8 21

### Frame analysis

There's always a solution in steel.

### Frame analysis

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

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### Moment-frame model

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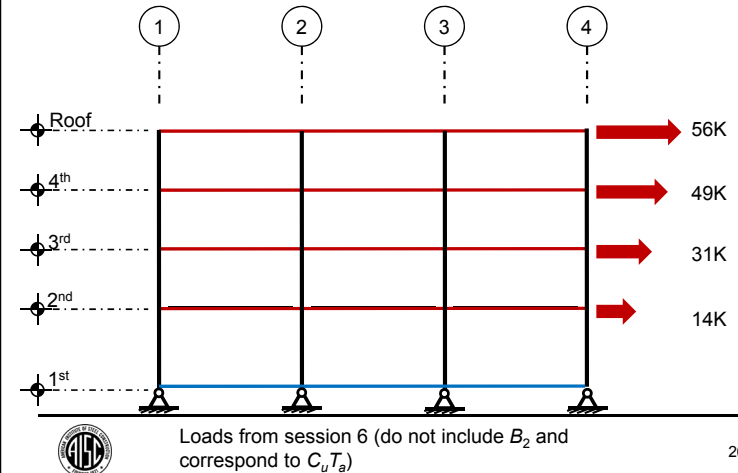
## Moment-frame model

- 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%



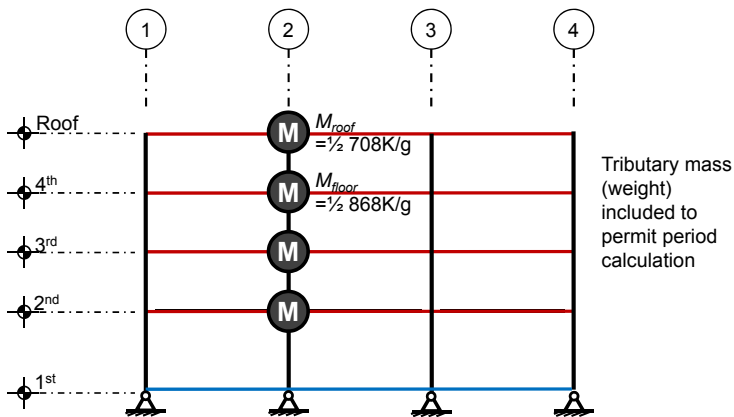
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## Moment-frame model



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## Moment-frame model



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## 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



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## 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

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## 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

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

W-Shapes

Shape	IMF Beams and Columns	SMF Beams and Columns	STMF Chord Joist	SCCS Columns	OCBF Diagonal Braces	SCBF Diagonal Braces	SCBF Columns	Beams	$\lambda_{max}$	$\lambda_{min}$
W14<731	•	•	•	•	•	•	•	•	19.5	38.5
>257	•	•	•	•	•	•	•	•	17.2	33.9
>233	•	•	•	•	•	•	•	•	17.0	33.7
>211	•	•	•	•	•	•	•	•	16.9	33.4
>193	•	•	•	•	•	•	•	•	16.8	33.2
>176	•	•	•	•	•	•	•	•	16.7	33.0
>159	•	•	•	•	•	•	•	•	16.6	32.9
>145	•	•	•	•	•	•	•	•	16.5	32.7
W14<132	•	•	•	•	•	•	•	•	15.6	30.9
>120	•	•	•	•	•	•	•	•	15.5	30.7
>109	•	•	•	•	•	•	•	•	15.5	30.6
W14<82	•	•	•	•	•	•	•	•	10.3	20.4
>74	•	•	•	•	•	•	•	•	10.3	20.4
>69	•	•	•	•	•	•	•	•	10.2	20.2
>61	•	•	•	•	•	•	•	•	10.2	20.1
W14<53	•	•	•	•	•	•	•	•	7.68	15.8
>48	•	•	•	•	•	•	•	•	7.86	15.7
>43	•	•	•	•	•	•	•	•	8.44	12.7
W14<38	•	•	•	•	•	•	•	•	6.26	12.6
>34	•	•	•	•	•	•	•	•	6.19	12.2
>30	•	•	•	•	•	•	•	•		

## Member Selection

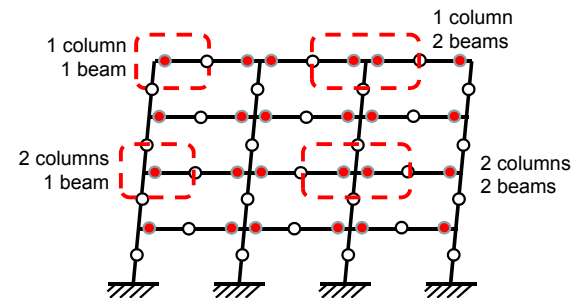
- Strong-Column/ Weak-Beam (SCWB)
  - $\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1$
  - Assume  $\frac{\sum Z_c}{\sum Z_{RBS}} > 1.5$ 
    - 3.0 makes column reinforcement less likely
      - Not applicable to deep columns
    - RBS beam reduction
      - 30% assumed
    - SCWB checked later



AISC 341 §E3.4a

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## Beam and column numbers



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## Strong-column/Weak-beam

P19     $f_x$     =P52\*\$A\$2/(SD19\*\$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



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## 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  $I_{ci} = I_{bi}$

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

- Reduces iterations



- But with computers iterations are easy

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## 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



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## Iteration (computer analysis)

- Period changes with member changes
- Base shear changes with period
  - Use model to determine 1<sup>st</sup>-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

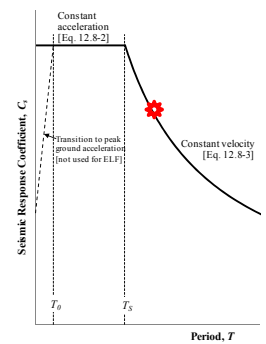


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### Iteration (computer analysis)

$C_u T_a = 0.92s$

T	C <sub>s</sub>	V <sub>drift</sub>	%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%



$T_f = 1.53s$  (final iteration)

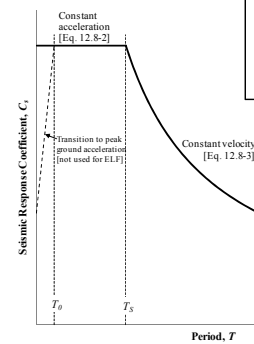


### Drift-determined period

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

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

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



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

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

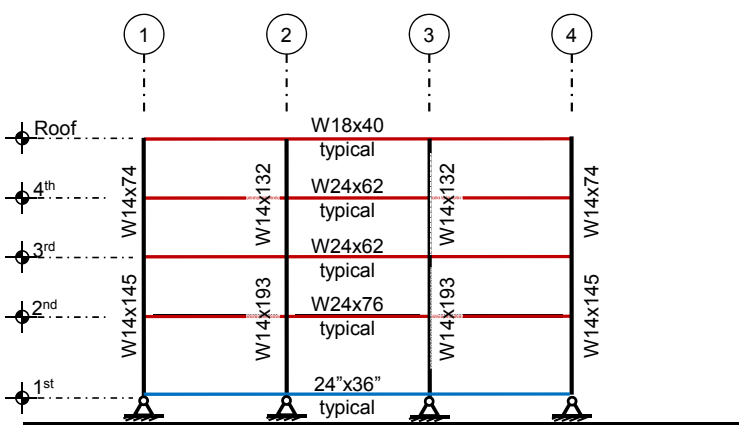
$$T = \frac{2}{3} \frac{0.025(51.5')(sec)}{0.6(feet)} = 1.4sec$$

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

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



### Final iteration



### 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




### Final iteration

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

$$\Delta = \frac{\delta_i}{h_i} = \frac{C_d \delta_{ei}}{1h_i}$$

Story4	0.014
Story3	0.019
Story2	0.018
Story1	0.018


---


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### Final iteration

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

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### 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 <sup>th</sup>	1.04
3 <sup>rd</sup>	1.05
2 <sup>nd</sup>	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
  - $K = 1$

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AISC 360 Appendix 8
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### Final iteration

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

$$B_2 * \text{Drift ratio}$$

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
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## 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

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## Check irregularities

- Vertical soft story

- Results from model

Level	Displacement (in.)	Shear (kips)	Stiffness (kips/in)
4 <sup>th</sup>	0.637	60	95
3 <sup>rd</sup>	0.860	113	132
2 <sup>nd</sup>	0.840	146	174
1 <sup>st</sup>	0.929	162	174

- Story stiffness increases with story shear
- Irregularity not present



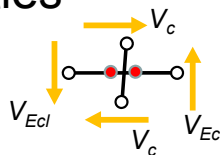
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## Check irregularities

- Vertical weak story

- Beam strength

- Assume uniform RBS reduction %



Level	Beam	M <sub>pr</sub> (ft-kip)	V <sub>Ec1</sub> (kip)	ΣV <sub>c</sub> (kip)
Roof	W18x40	303	22.1	638
4 <sup>th</sup>	W24x62	591	44.0	633
3 <sup>rd</sup>	W24x62	591	44.0	633
2 <sup>nd</sup>	W24x76	772	57.5	782

- Irregularity not present



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## 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



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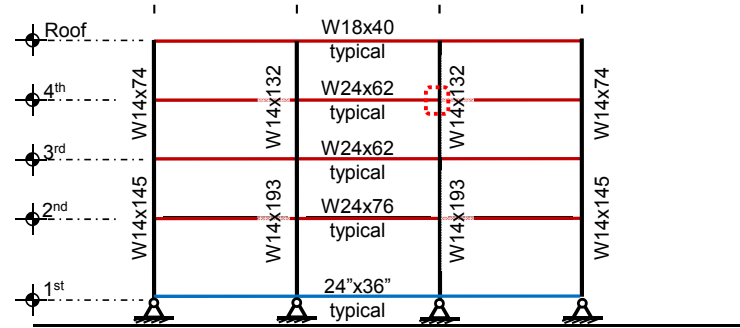
There's always a solution in steel.

## Connection design



## Connection design

	A	d	t <sub>w</sub>	b <sub>f</sub>	t <sub>f</sub>	k	k <sub>t</sub>	b/2t <sub>f</sub>	h/t <sub>w</sub>	I <sub>x</sub>	Z <sub>x</sub>
W24x62	18.2	23.7	0.430	7.04	0.590	1.63	1 <sup>1</sup> / <sub>16</sub>	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




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## 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

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2 W24x62 beams  
2 W14x132 columns
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## 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

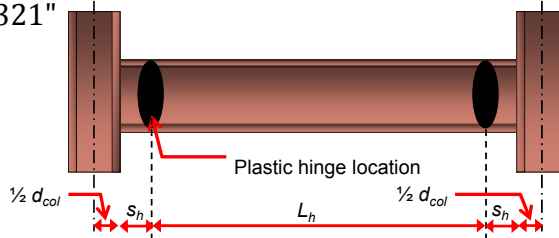
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2 W24x62 beams  
2 W14x132 columns
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## Beam analysis

- 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

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## 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

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## Approximate RBS beam stiffness

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

$$\frac{I_{ef}}{I} = \frac{1}{1 + \frac{L_h}{(L_h + 2s_h)^2} \left[ \frac{b}{1 - (ct_f/I)(d - t_f)^2} \right]} = 0.95$$



2 W24x62 beams  
 2 W14x132 columns

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## Beam analysis

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

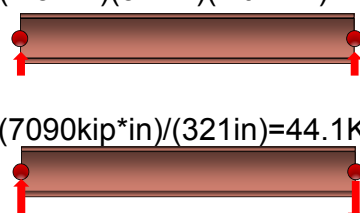



2 W24x62 beams  
 2 W14x132 columns

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### 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)

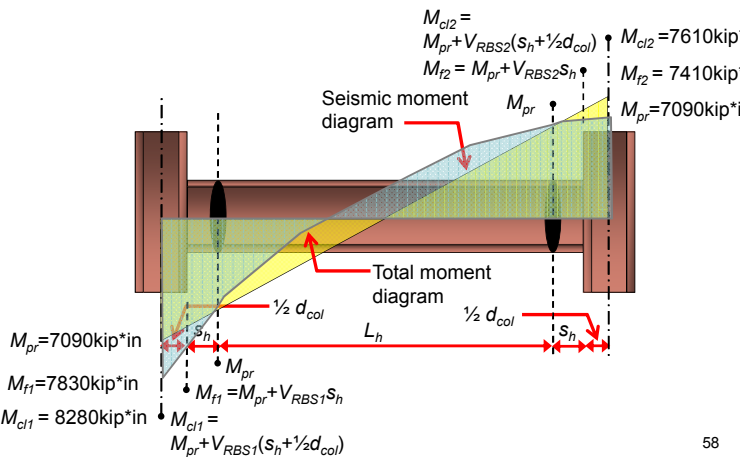





2 W24x62 beams  
 2 W14x132 columns

57

### Beam analysis

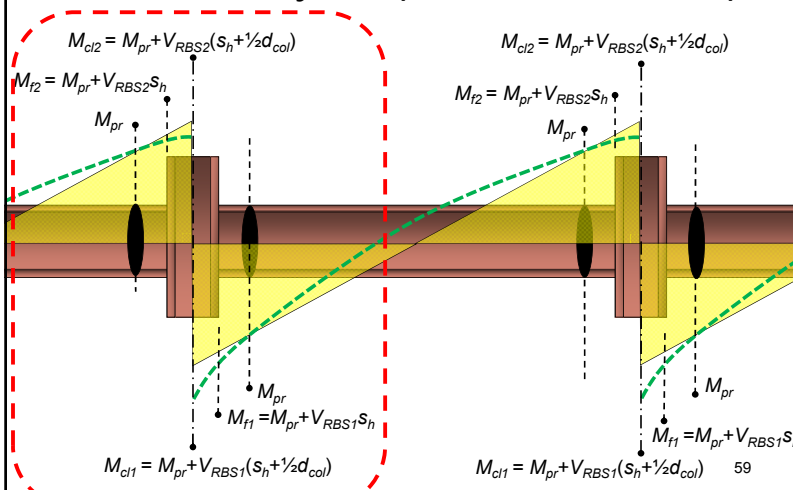





2 W24x62 beams  
 2 W14x132 columns

58

### Beam analysis (interior column)






2 W24x62 beams  
 2 W14x132 columns

59

### Check section at column face

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



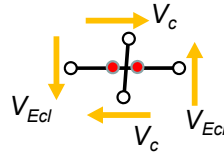
2 W24x62 beams  
 2 W14x132 columns

60



## Strong-Column/Weak-Beam

- $\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1$  AISC 341 §E3.4a
- $M_{pc}^* = Z_c(F_y - P_u/A) + V_c d_b/2$ 
  - $P_u = 1.4D + 0.5L + 3.0E_h = 249K$
  - $V_c = \frac{2V_{Ecl}L/2}{1/2h_i + 1/2h_{i-1}} = 106K$
  - $M_{pc}^* = 11,500kip\cdot in$
  - $\sum M_{pc}^* = 22,900kip\cdot in$



2 W24x62 beams  
2 W14x132 columns

61

## Strong-Column/Weak-Beam

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

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

AISC 360 §J.10



2 W24x62 beams  
2 W14x132 columns

63

## Forces at column face

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

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## 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

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## Panel Zone

- $V_u = \frac{M_{f1} + M_{f2}}{d_b} - V_c = 643K - 106K = 537K$
- $\phi V_n = 1.0 F_y \left[ 0.6 t_{cw} d_c + \frac{1.8 b_{cf} t_{cf}^2}{d_b} \right]$  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

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## Optimization

There's always a solution in steel.



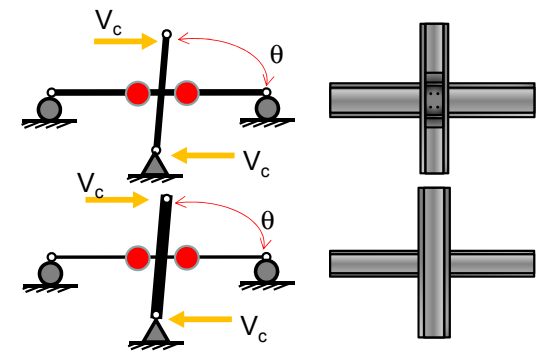
### Optimization

- 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



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### Elimination of doublers and continuity plates



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

### Elimination of doublers and continuity plates

- |                          |                          |                          |
|--------------------------|--------------------------|--------------------------|
| • Try W14x145            | • Try W14x159            | • Try W14x176            |
| ○ $I_{b2} \geq 1460in^4$ | ○ $I_{b2} \geq 1420in^4$ | ○ $I_{b2} \geq 1360in^4$ |
| ○ W24x62                 | ○ W24x62                 | ○ W24x55                 |
| • $I_b = 1550in^4$       | • $I_b = 1550in^4$       | • $I_b = 1350in^4$       |
| ○ Continuity plates      | ○ No continuity plates   | • Check drift!           |
| ○ 5/16" doubler          | ○ 3/16" doubler          | ○ No continuity plates   |
|                          |                          | ○ No doubler             |

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



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

- |  |  |
|--|--|
| • Original design                          | • Redesign (3 bays)                        |
| ○ W24x62 beams $\frac{Z_c}{Z_{RBS}} = 2.1$ | ○ W24x55 beams $\frac{Z_c}{Z_{RBS}} = 3.2$ |
| ○ W14x132 columns                          | ○ W14x176 columns                          |
| ○ 1.9% drift                               | ○ 2.0% drift                               |
| ○ 4 <sup>th</sup> floor                    | ○ 1230# more steel                         |
| • Continuity plates                        | ○ +\$1230                                  |
| • Doubler plate                            | ○ No continuity plates                     |
| ○ Roof (not shown)                         | • 32 eliminated:                           |
| • Continuity plates                        | ○ -\$3200                                  |
| • No doubler plate                         | ○ No doubler plates                        |
|  | • 2 eliminated:                            |
|  | ○ -\$600                                   |




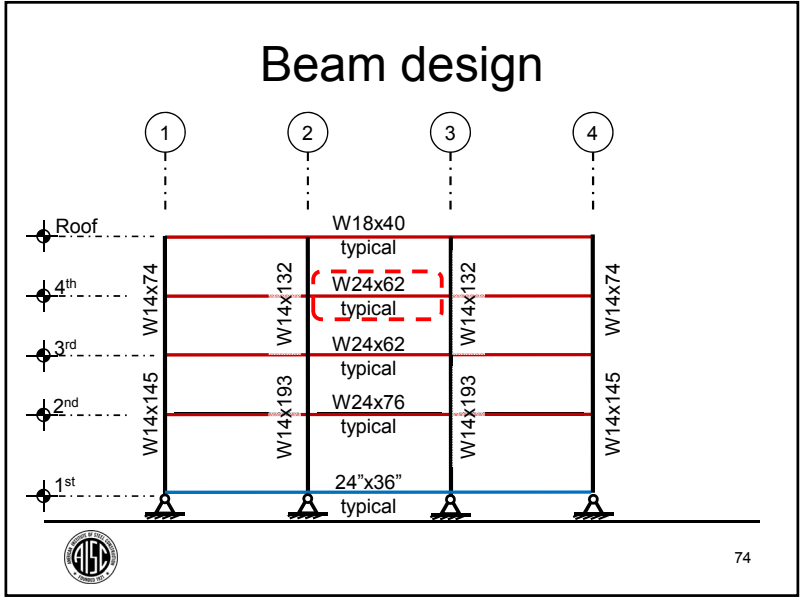
\$2570 savings

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There's always a solution in steel.

## Beam strength design

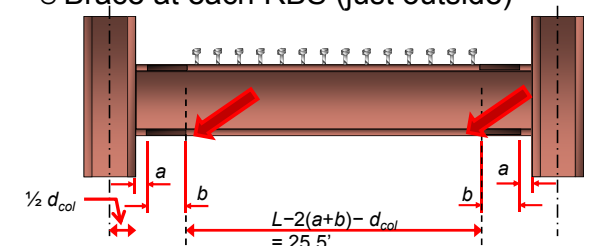
### Beam design

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




### Beam design

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



$L - 2(a+b) - d_{col} = 25.5'$

AISC 341 §E3.4b




## 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  $1550 \text{ in}^4$  for W24x62)
  - $50r_y = 50(1.81") = 7.5'$
  - Brace @ 25.5'/4=6.4'
  - Redesign connection

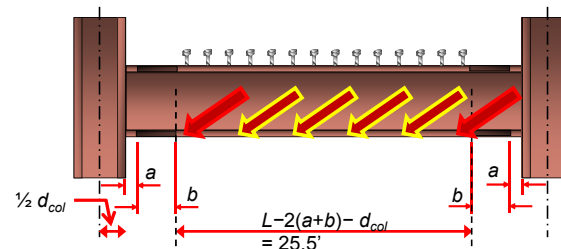


AISC 341 §D1.2b

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## Beam design

- Lateral bracing



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## Beam design

- 4<sup>th</sup> 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



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## Beam design

- At  $L_b > L_p$  (1<sup>st</sup> segment outside RBS)
  - $M_u \approx 281'K \frac{180'' - d_c/2 - s_h}{180''} = 0.89 M_u = 250'K$
  - $C_b = 1.2$  (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



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### Beam design

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

For straight-line moment diagram (no distributed load)  
 $M_1/M_2 \sim 1 - 2L_b/L_h$

AISC 360 §F.1

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### Column strength design

There's always a solution in steel.

### Column design

	A	d	t <sub>w</sub>	b <sub>f</sub>	t <sub>f</sub>	k <sub>1</sub>	W	b/2t <sub>f</sub>	h/t <sub>w</sub>	I <sub>x</sub>	Z <sub>x</sub>
<b>W14x145</b>	42.7	14.8	0.680	15.5	1.09	1 9/16	145	7.11	16.8	1710	260

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### Column design

- Case 1
  - Combo M-BLC-1
    - 1.4D + 0.5L + 1.0E<sub>h</sub>
    - By inspection compression controls
  - Axial and flexure considered
    - B<sub>1</sub> and B<sub>2</sub> apply

- Case 2
  - Combo M-OLC-1
    - 1.4D + 0.5L + 3.0E<sub>h</sub>
    - By inspection compression controls
  - Axial in the absence of flexure considered
    - B<sub>2</sub> applies

AISC 341 §D1.4a

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### 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
  - $M_D = 29.7'K$
  - $M_L = 13.6'K$
  - $M_E = B_2 200'K = 214'K$



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### 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
  - $M_u = 1.4(29.7'K) + 0.5(13.6'K) + (214'K) = 262'K$



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### 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)
- $C_m < 0.6$

$C_m = 0.6$

$C_m > 0.6$

$C_m = 1.0$
- $P_{E1} = \frac{\pi^2 EI}{L^2} = 17,340K$
  - $B_1 = 1$
- If  $C_m = 0.6$   
 $B_1 = 1$  for  $P_u \leq 0.4P_{E1}$



AISC 360 Appendix 8

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### 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



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## 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

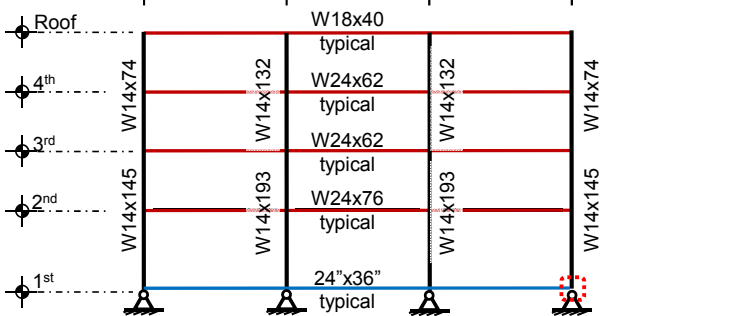
There's always a solution in steel.

## Base-plate design



## Base-plate design

W14x145	A	d	t <sub>w</sub>	b <sub>f</sub>	t <sub>f</sub>	k <sub>1</sub>	W	b/2t <sub>f</sub>	h/t <sub>w</sub>	I <sub>x</sub>	Z <sub>x</sub>
	42.7	14.8	0.680	15.5	1.09	1 9/16	145	7.11	16.8	1710	260



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## 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



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### 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
  - $M_D = 12.8'K$
  - $M_L = 5.8'K$
  - $M_E = B_2 196'K$   
 $= 209'K$



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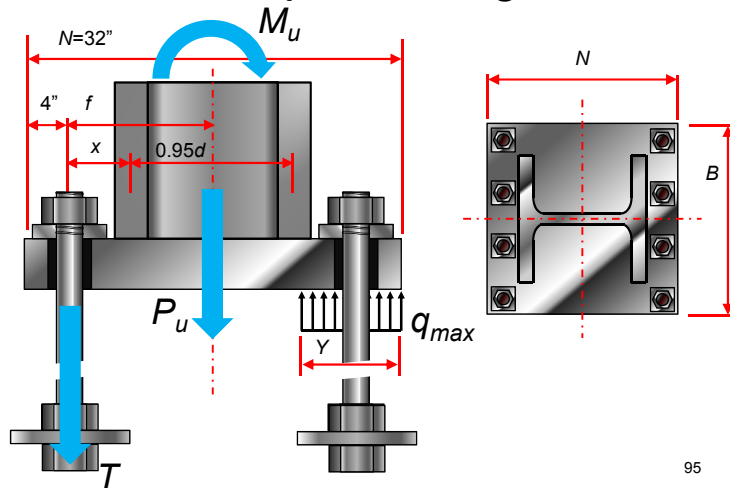
### Base-plate design

- $R_u = 0.7D + 3.0E_h$ 
  - combo M-OLC-2 (session 5)
- $V_u = 85.3K$
- $P_u = 59.3K$  Tension
- $M_u = 635'K$ 
  - Column expected strength:  
 $\bullet 1.1R_y F_y Z_x = 1310'K$  D2.6c(b)



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### Base-plate design



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### Base-plate design

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



AISC Design Guide 1 §3.4

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### Base-plate design

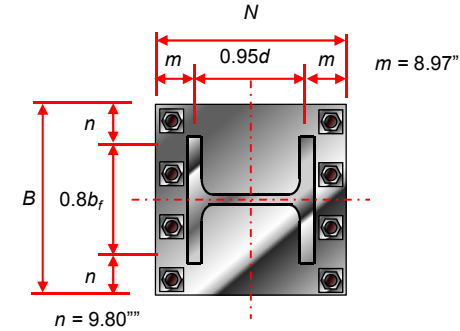
- $Y = \left[ f + \frac{N}{2} \right] \pm \sqrt{\left( f + \frac{N}{2} \right)^2 - \frac{2P_u(e+f)}{q_{max}}}$
- $Y = \left[ 12" + \frac{32"}{2} \right] \pm \sqrt{\left( 12" + \frac{32"}{2} \right)^2 - \frac{2(-59.3K)(-128"+12")}{141K/in}}$
- $Y = 54.2", 1.81"$
- $T = q_{max} Y + P_u = 141K/in * 1.81"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



78.6K/rod AISC Design Guide 1 §3.4

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### Base-plate design



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### Base-plate design

- Flexure at the bearing side
- $t_p \geq 2.11 \sqrt{\frac{f_{pmax} Y [\max(m,n) - Y/2]}{F_y}}$
- $t_p \geq 2.11 \sqrt{\frac{4.42ksi(1.81") [9.80" - 1.81"/2]}{50ksi}} = 2.51"$
- Use  $2\frac{3}{4}"$  plate



AISC Design Guide 1 §3.4

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### Base-plate design

- Flexure at the tension side
  - $x = f - d/2 + t_f/2 = 5.15"$
  - $t_p \geq 2.11 \sqrt{\frac{T_x}{BF_y}} = 2.1"$  Use  $2\frac{3}{4}"$  plate

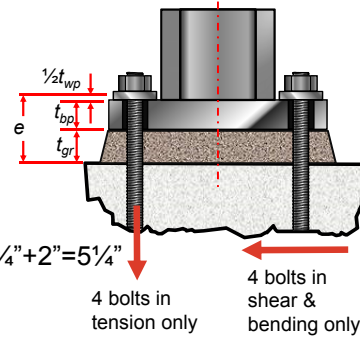


AISC Design Guide 1 §3.4

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## Anchor-rod design

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



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## Anchor-rod design (tension)

- Check anchor rod
  - Loads
    - $M_u = 0"K$
    - $T_u = 78.6\text{K}$
    - $V_u = 0\text{K}$
  - Properties
    - $d = 1\frac{1}{2}"$
    - $A = 1.77\text{in}^2$
    - $A_{net} = 1.41\text{in}^2 = 0.795A$ 
      - 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
    - $M_u = 56.2"K$
    - $T_u = 0\text{K}$
    - $V_u = 21.4\text{K}$
  - Properties
    - $d = 2"$
    - $A = \pi\text{in}^2$
    - $A_{net} = 2.50\text{in}^2 = 0.795A$ 
      - 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

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## Anchor-rod design

- Rods designed for tension only:
  - $1\frac{1}{2}"$  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
  - $2\frac{1}{4}"$  diameter



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## Anchor-rod design

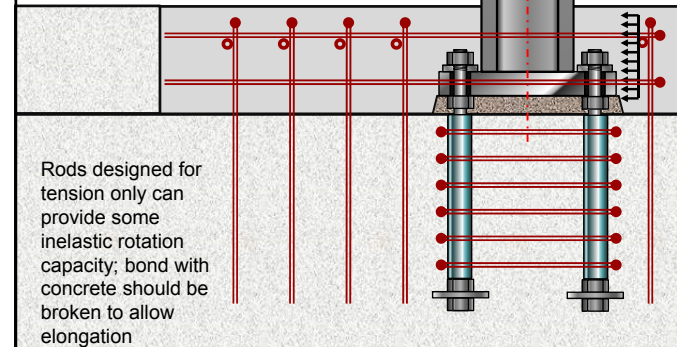
- Shear alternatives (no moment in rods)
  - Bearing of base plate and column
    - OK for base plates embedded in slabs
    - Concrete reinforcement to complete load path
    - $f_p = \phi 0.85 f'_c = (0.65)0.85(4ksi) = 2.2ksi$
    - $f_p B t_p = 2.2ksi * 32" * 3" = 211K$
  - Friction
    - $C_\mu =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



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## Anchor-rod design

Shear alternatives



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## Anchor-rod design

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

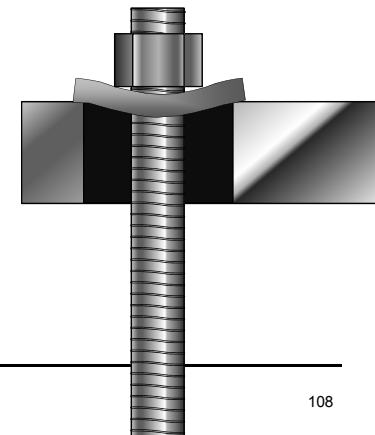
- Use (8) 1½" Grade 55 anchor rod F1554, S1

Anchor Rod Diameter, in.	Hole Diameter, in.	Min. Washer Size, in.	Min. Washer Thickness
1½	2 <sup>9</sup> / <sub>16</sub>	3½	5/8
1¾	2¾	4	7/8
2	3¼	5	3/4
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.

- Unpublished recommendations to prevent plate dishing:

- Grade 36
  - Use Table 14-2
- Grade 55
  - Use  $t = 0.4d_{rod}$
- Grade 105
  - Use  $t = 0.69d_{rod}$

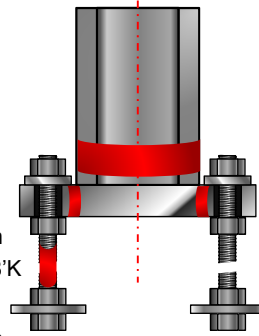


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### Base connection design

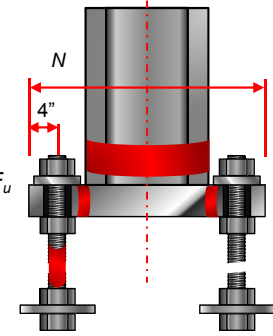
- Potential sources of base inelastic rotation
  - Column inelastic rotation
    - Significant overstrength
    - Affects column
  - Anchor-rod elongation
    - Compression increases overstrength
    - $M_c = P_u(N-2*4)/2 = 298K(2')/2 = 298'K$
  - Base plate flexure
    - Compression increases overstrength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



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### Base connection design

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



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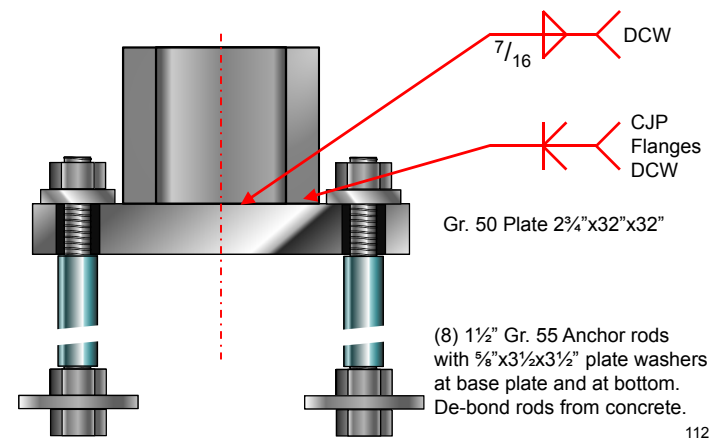
### Base connection design

- 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<sub>w</sub>



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### Base connection design




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## Completion of design




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## 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

---




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## 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


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## Summary



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## 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



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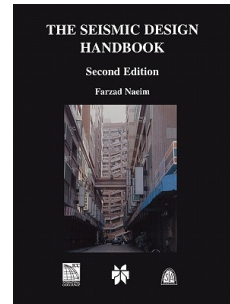
## End of session 7

Next:

## Design of the Braced frames



## Additional resources



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## Question time



## Individual Webinar Registrants

### CEU/PDH Certificates

Within 2 business days...

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



## Individual Webinar Registrants

### CEU/PDH Certificates

Within 2 business days...

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



## 8-Session Registrants

### CEU/PDH Certificates

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



## 8-Session Registrants

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

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

Reasons for quiz:

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

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



### 8-Session Registrants

**Access to the recording:** Information for accessing the recording will be emailed to you by this Wednesday. The recording will be available for three weeks. For 8-session registrants only. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

**CEUs/PDHS** – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHS.



### Night School Resources for 8-session package Registrants

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



### Night School Resources for 8-session package Registrants

Go to [www.aisc.org](http://www.aisc.org) and sign in.

The screenshot shows the MyAISC user interface. On the left, a sidebar menu lists various options: 'IN THIS SECTION', 'Edit Profile', 'My Downloads', 'My Pending Quizzes', 'My Events', 'Order History', 'Course History', and 'Course Resources'. The 'Course Resources' option is circled in red. The main content area is titled 'MyAISC' and contains three sections: 'MY PROFILE' (with an 'EDIT PROFILE' button), 'MY PURCHASED DOWNLOADS' (with a 'VIEW DOWNLOADS' button), and 'MY COURSE RESOURCES' (with a 'VIEW RESOURCES' button). The 'MY COURSE RESOURCES' section is also circled in red.

# Thank You

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

There's always a solution in steel.

