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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.

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Learning Objectives

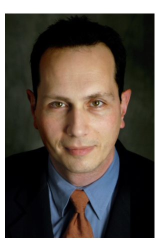
- Identify the various prequalified connections for use in moment frames.
- List the advantages and disadvantages of the reduced beam section (RBS) connection.
- Describe strong-column / weak-beam proportioning.
- Identify the steps of a moment frame base-plate design.

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

structural STEEL

Course objectives

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

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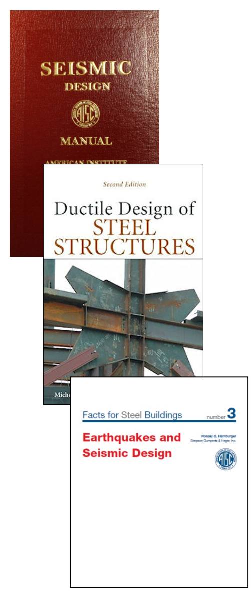
Resources



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

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Other resources

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

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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.<u>Design of the moment frames</u>

8.Design of the braced frames

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Session 7:
Design of the Moment Frames

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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

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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

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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

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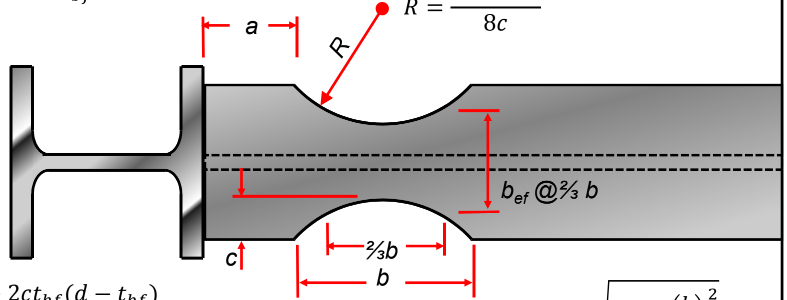
Reduced Beam Section (RBS)

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

$0.65d \le b \le 0.85d$

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

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



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

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

AISC 358 §5.8

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Frame analysis

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Frame analysis

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

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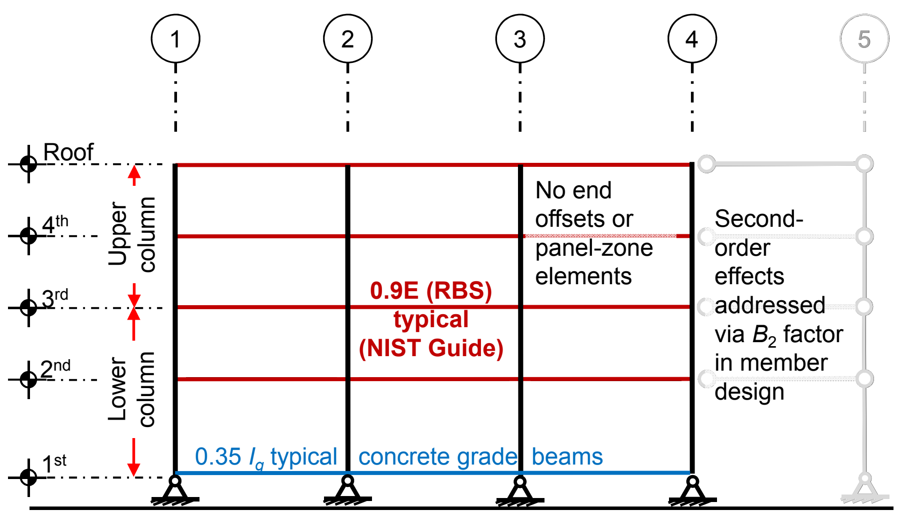
Moment-frame model



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

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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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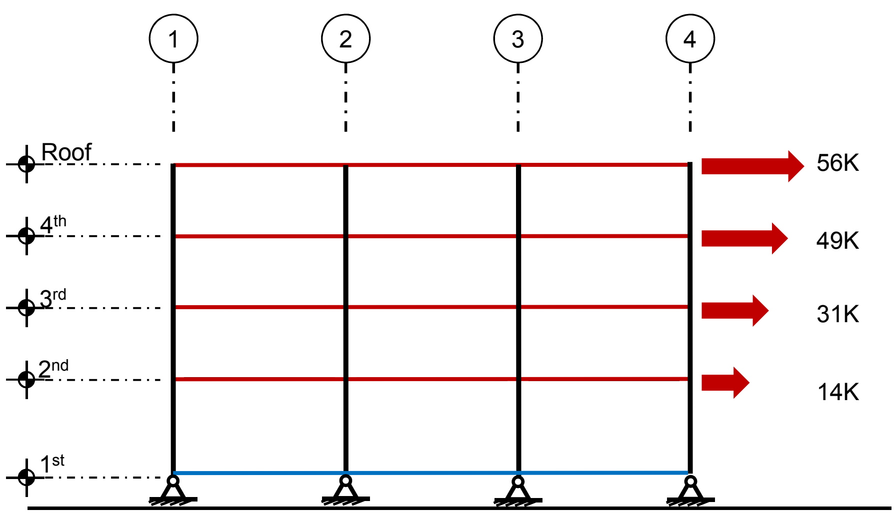
Moment-frame model



Loads from session 6 (do not include B_2 and correspond to $C_u T_a$)

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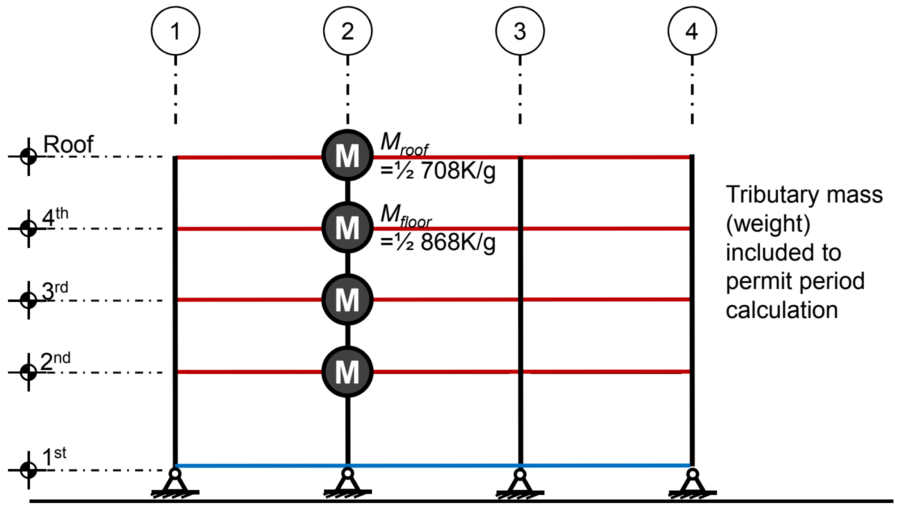
Moment-frame model



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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
  - o Maximum depth = $^{(360”-14”)}/_7$ = 50”
  - o W36 maximum nominal depth
  - o 300 PLF maximum
  - o t_f ≤ 1¾“
  - o Prefer r_y ≥ 2”
    - For bottom-flange brace spacing ≥ 8’
      - o Maximum L_b ~ $50r_y$ for $R_y F_y$ = 55ksi
        - AISC 341 §D1.2b

AISC 358 §5.3

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Member Selection

- Compactness
  - o Use SDM Table
  - o Note that RBS beam compactness is measured at the $^1/_6$ point of the cut
    - Additional sections not indicated in the table may work

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

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

Shape	IMF	SMF	STMF	SCCS	OCBF	SCBF			$L_{b\,max}$, ft	
	Beams and Columns	Beams and Columns	Chord Segment	Columns	Diagonal Braces	Diagonal Braces	Columns	Beams	λ_{hd}	λ_{md}
W14×730	•	•	•	•	•	•	•	•	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
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
×68	•	•	•	•	•	•	•	•	10.2	20.2
×61	•				•			•	10.2	20.1
W14×53	•	•	•	•	•	•	•	•	7.98	15.8
×48	•	•	•	•	•	•	•	•	7.94	15.7
×43	•							•	7.86	15.5
W14×38	•	•	•	•			•	•	6.44	12.7
×34	•							•	6.36	12.6
×30	•							•	6.19	12.2

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

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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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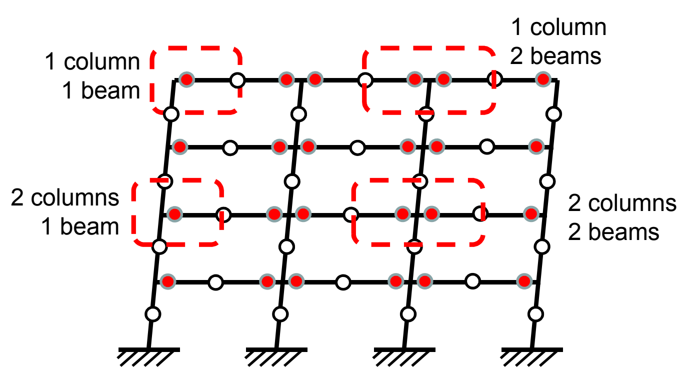
Beam and column numbers



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Strong-column/Weak-beam

P19 =P$2*$A$2/($D19*B2*B3)

	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
52			W24X103	280	1.81	1.63	1.46	1.33	1.19	1.08
53			W24X94	254	2.00	1.80	1.61	1.46	1.32	1.19
54			W24X84	224	2.26	2.04	1.83	1.66	1.49	1.35
55			W24X76	200	2.54	2.29	2.05	1.86	1.67	1.51
56			W24X68	177	2.87	2.58	2.32	2.10	1.89	1.71
57			W24X62	153	3.31	2.99	2.68	2.43	2.18	1.98
58			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

Preliminary design

- Hand methods (see Naeim, assumptions added)
  - $\Delta_i = C_d \dfrac{V_i {h_i}^2}{12}\left[\dfrac{1}{\Sigma\frac{EI_{c\,i}}{h_i}+\Sigma\frac{EI_{b\,i}}{L}}\right] = 0.02h_i$
  - Assume typical $I_{c\,i} = 2I_{b\,i}$
    - End column $I_{c\,i} = I_{b\,i}$
  - $I_{b\,i} = \dfrac{V_i h_i}{12EN}\dfrac{C_d}{0.02}\left[\dfrac{h_i}{2}+L\right]$ for *N* bays
  - Reduces iterations
    - But with computers iterations are easy

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 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

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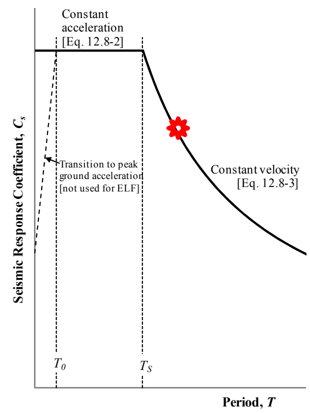
Iteration (computer analysis)

$C_u T_a$ = 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)

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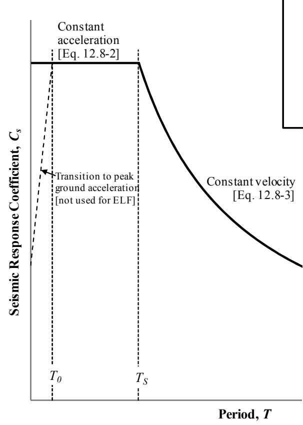
Drift-determined period

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

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

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



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

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

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

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

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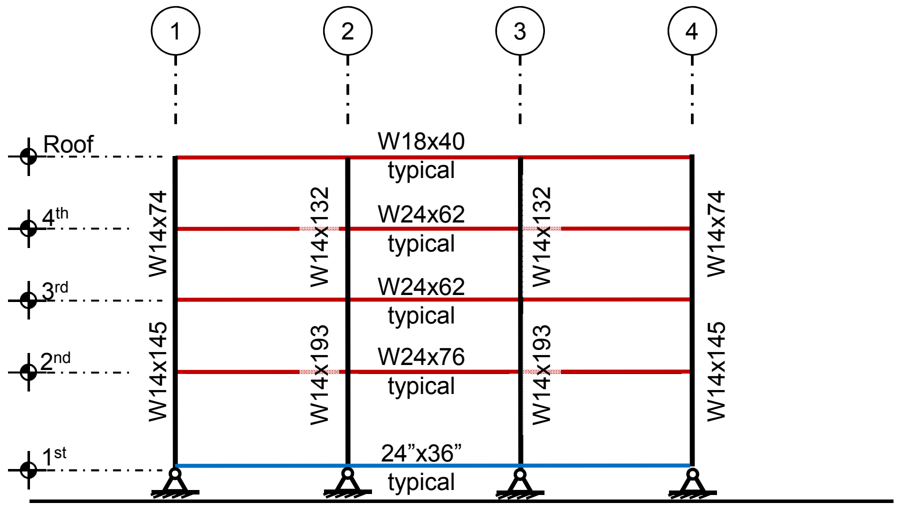
Final iteration



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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

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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

Final iteration

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

Second Order Effects: SMF

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

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

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

- Drift-controlled
  - $\Delta = 0.02h$
- Use P-Δ gravity load
  - $P = 1.0D + 0.5L$
- Assume $P_{mf} = ½ P_{story}$
- B_2 = 1.06 (largest)
  - Use ELM
  - $K=1$

AISC 360 Appendix 8

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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 ≤1.1 need not be considered but was for procedural consistency

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Stability check (ASCE 7 12.8.7)

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

ASCE 7 §12.8.7

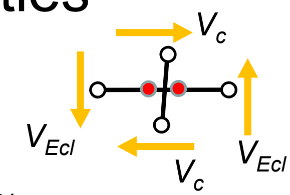
Check irregularities

- Vertical soft story
  - Results from model

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

  - Story stiffness increases with story shear
  - Irregularity not present

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
4th	W24x62	591	44.0	633
3rd	W24x62	591	44.0	633
2nd	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

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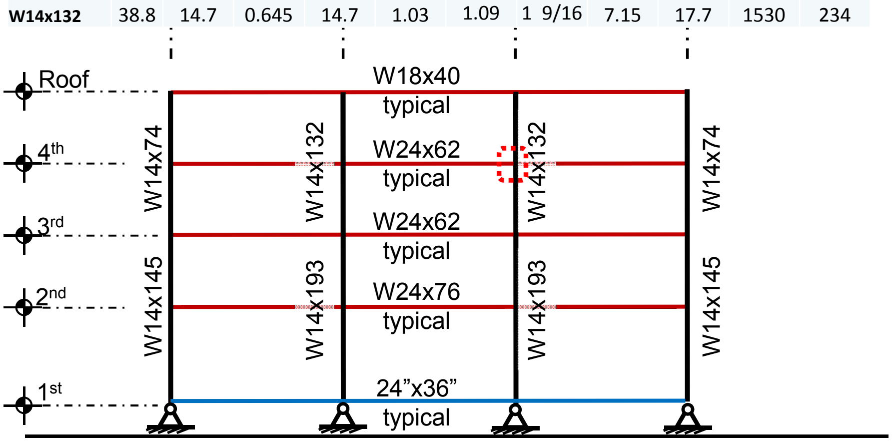
Connection design

	A	d	t_w	b_f	t_f	k	k_1	$b_f/2t_f$	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



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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

2 W24x62 beams
2 W14x132 columns

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Compute beam plastic-hinge moment

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

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

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

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

AISC 358 §5.8

2 W24x62 beams
2 W14x132 columns

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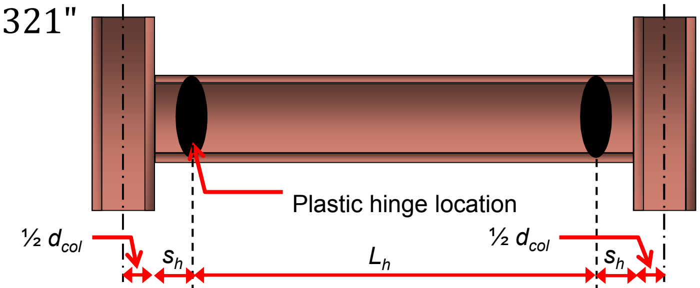
Beam analysis

- 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
  - o High span-to-depth ratios
    - ≥7 required for SMF
  - o Typical flange/web thickness ratios
    - Rolled WF beams
- Sabelli approximate equation:

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

2 W24x62 beams
2 W14x132 columns

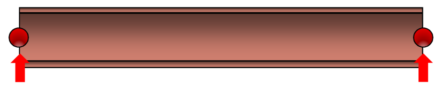
Beam analysis

- Gravity distributed load
  - o w_u = 1.2D +0.5L
  - o w_u = 1.2(0.840klf)D +0.5(0.600klf) = 1.31klf

2 W24x62 beams
2 W14x132 columns

Beam analysis

- Gravity shear
  - $V_g = ½\, w_u\, L_h$ = ½(1.31klf)(321in)(1ft/12in)
  - V_g = 17.5K

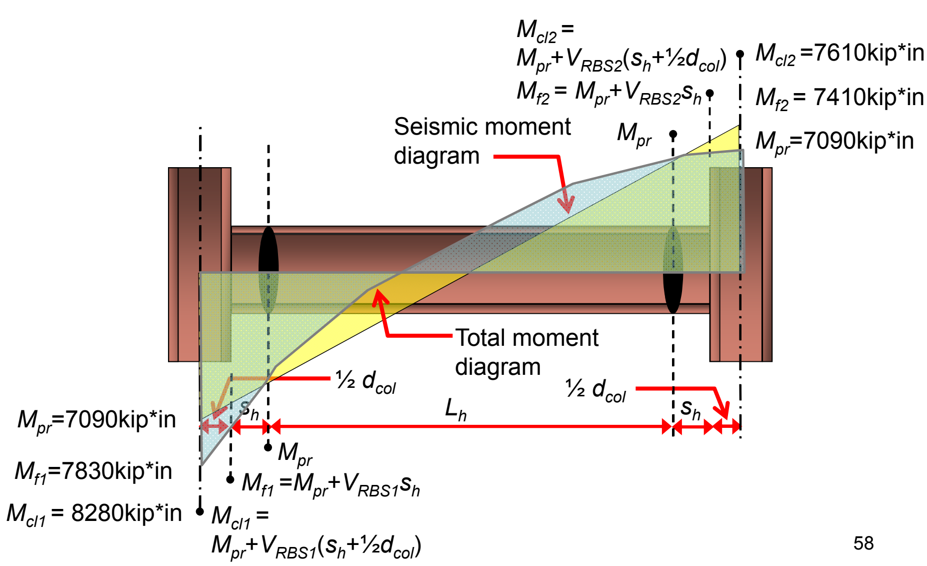


- Seismic shear
  - $V_{ecl} = 2M_{pr}/L_h$ = 2(7090kip*in)/(321in)=44.1K
- Total shear

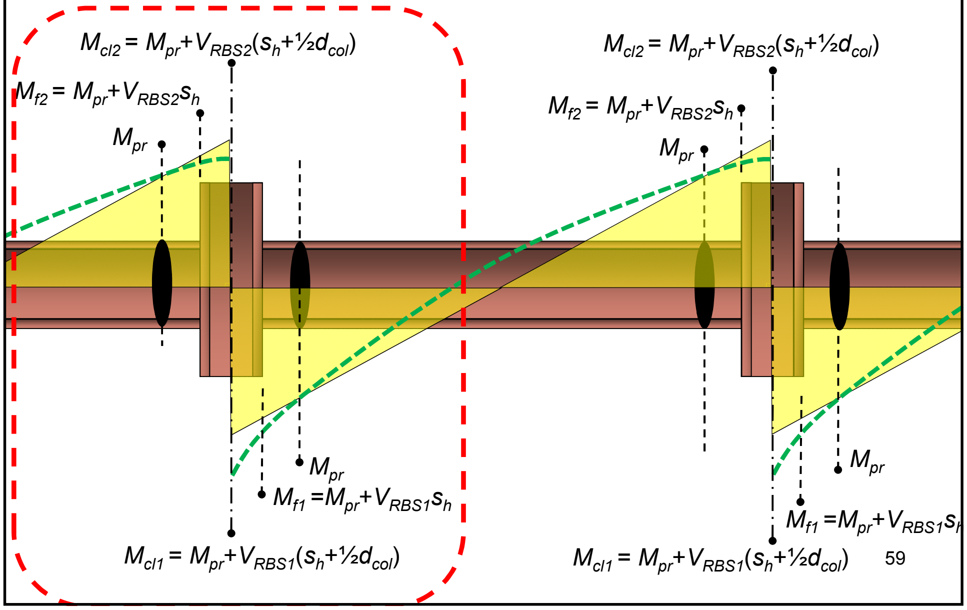
  - V_{RBS1} = 17.5K+44.1K=61.6K
  - V_{RBS2} = 17.5K−44.1K=−26.6K (i.e., up)

2 W24x62 beams
2 W14x132 columns

Beam analysis



Beam analysis (interior column)



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Check section at column face

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

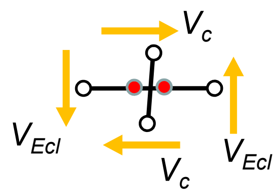
2 W24x62 beams
2 W14x132 columns

60

Strong-Column/Weak-Beam

AISC 341 §E3.4a

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



2 W24x62 beams
2 W14x132 columns

61

Strong-Column/Weak-Beam

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

$\frac{\sum Z_c}{\sum Z_{RBS}} = 2.1$ (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

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

AISC 360 §J.10

2 W24x62 beams
2 W14x132 columns

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Forces at column face

- Web crippling
  - $øR_n = ø0.80t_{cw}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{cw}}{t_{cf}}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_{cf}}{t_{cw}}}$
  - $øR_n = 0.75 * 0.80t_{cw}^2 \left[1 + 3\left(\frac{t_{bf}}{d_c}\right)\left(\frac{t_{cw}}{t_{cf}}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_{cf}}{t_{cw}}}$
  - $øR_n$ =402K, OK

AISC 360 §J.10

2 W24x62 beams
2 W14x132 columns

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Forces at column face

- Flange local bending
  - $\emptyset R_n = 0.9 * 6.25 t_{cf}{}^2 F_{yc}$ =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
      - $6.25 t_{cf}{}^2 \geq 1.8 b_{bf} t_{bf}\left({}^{F_{yb}R_{yb}}/_{F_{yc}R_{yc}}\right)$
      - $0.9 * 6.25 F_{yc} t_{cf}{}^2 \geq {}^{0.9*1.8}/_{R_{yc}}\, b_{bf} t_{bf} F_{yb} R_{yb}$
    - $R_u \sim 1.5 b_{bf} t_{bf} R_y F_{yb} = 343K$
      - 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

- $V_u = \frac{M_{f1}+M_{f2}}{d_b} - V_c = 643K - 106K = 537K$
- $\phi V_n = 1.0F_y\left[0.6t_{cw}d_c + \frac{1.8b_{cf}t_{cf}^2}{d_b}\right]$ AISC 341 E3.6e(1)
- ϕV_n =285K+59K=344k
- Doubler required
  - 537K-344K=193K
  - $193K\,\frac{t_{cw}}{285k}$ =0.44", ½"doubler required

2 W24x62 beams
2 W14x132 columns

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Optimization

There's always a solution in steel.

structural 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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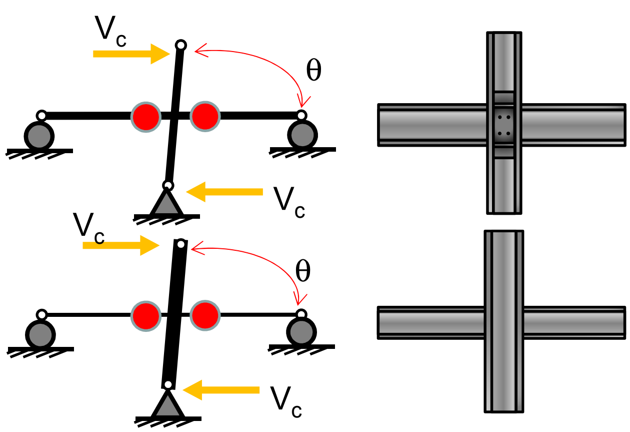
Elimination of doublers and continuity plates



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

Elimination of doublers and continuity plates

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

<u>Frame:</u>
$N_c = N_b = 2$
$h = 12.5'$
$L = 30'$

<u>Original design:</u>
W24x62: $I_{b1} = 1550\text{in}^4$
W14x132: $I_{c1} = 1530\text{in}^4$
$\Delta_{all}/\Delta = 1.02$

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

Optimization

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

$2570 savings

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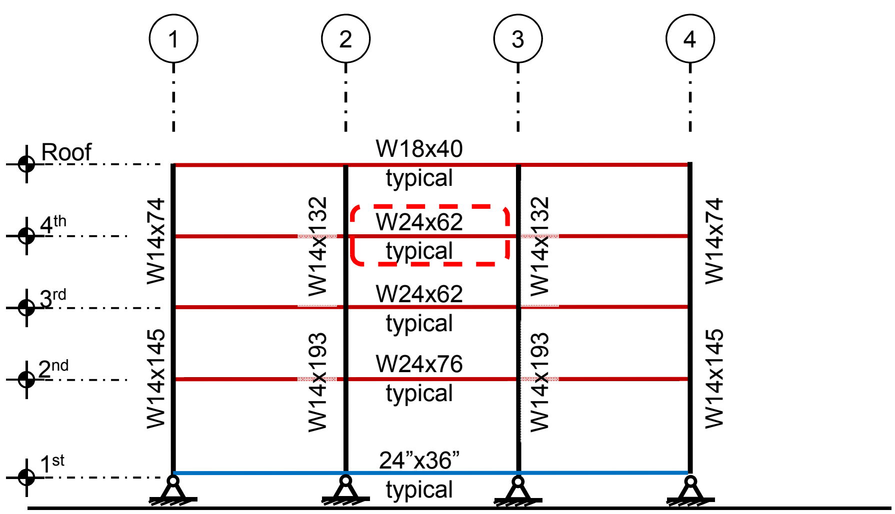
There's always a solution in steel.

Beam strength design

structural
STEEL

Beam design



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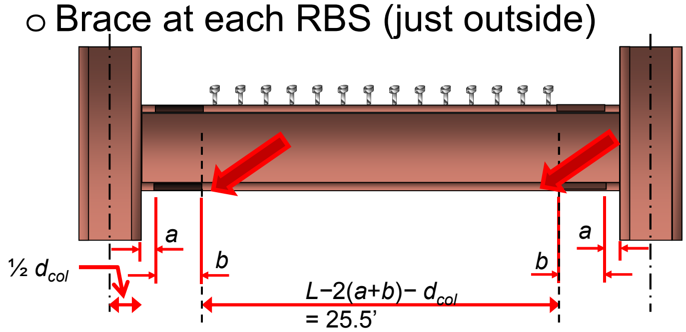
Beam design

- 4^{th} 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

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Beam design

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

AISC 341 §D1.2b

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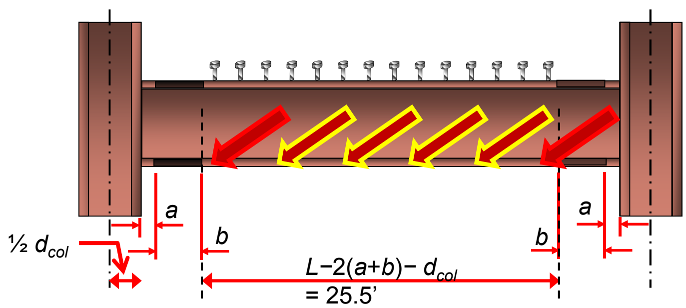
Beam design

- Lateral bracing



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Beam design

- 4^{th} 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^{st} 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)
  - ϕM_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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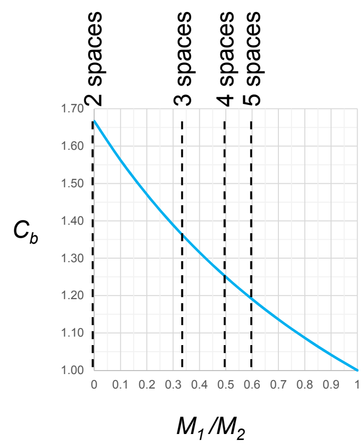
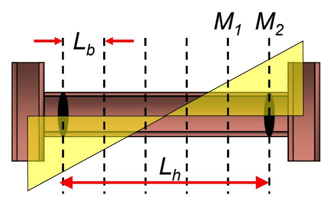
Beam design



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



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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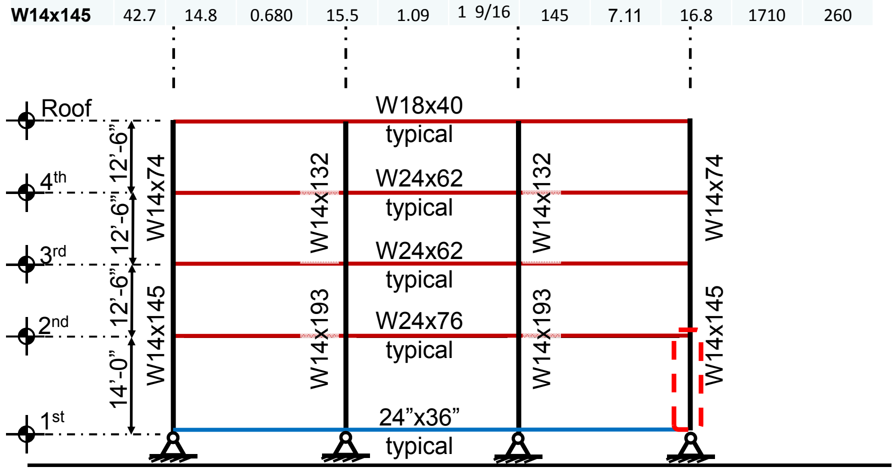
Column strength design

There's always a solution in steel.

Column design

	A	d	t_w	b_f	t_f	k_1	W	$b_f/2t_f$	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



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 <u>in the absence of flexure considered</u>
    - B_2 applies

AISC 341 §D1.4a

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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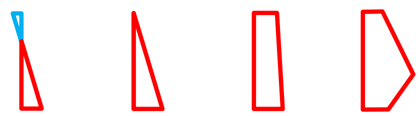
Column design (case 1)

- $B_1 = \frac{C_m}{1 - {P_u}/{P_{E1}}} \geq 1$
  - 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
$B_1 = 1 \; for P_u \leq 0.4 P_{E1}$

AISC 360 Appendix 8

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Column design (case 1)

- ϕP_n = 1690K (Table 4-1)
  - $P_u / \phi P_n$ = 0.18
- ϕM_n = 975'K (L_b<L_p, Table 3-2)
- $½P_u / \phi P_n + M_u / \phi M_n$ = 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

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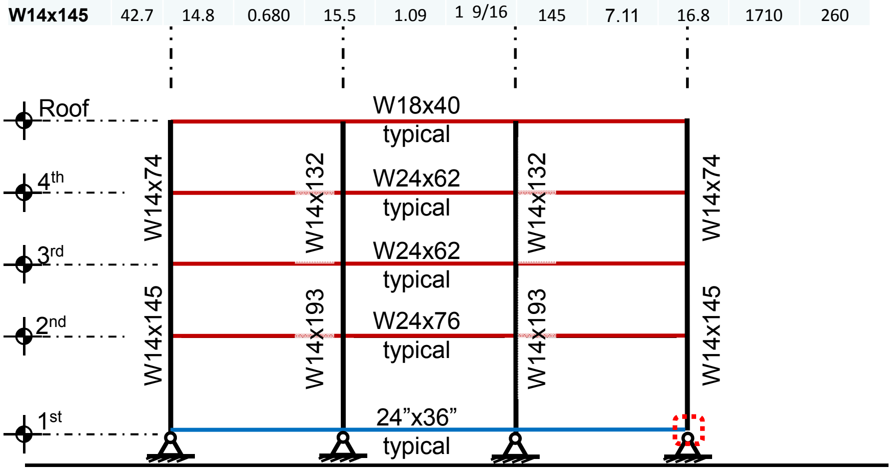
Base-plate design

There's always a solution in steel.

structural STEEL

Base-plate design

	A	d	t_w	b_f	t_f	k_1	W	$b_f/2t_f$	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

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:
    - $1.1R_yF_yZ_x$ = 1310'K D2.6c(b)

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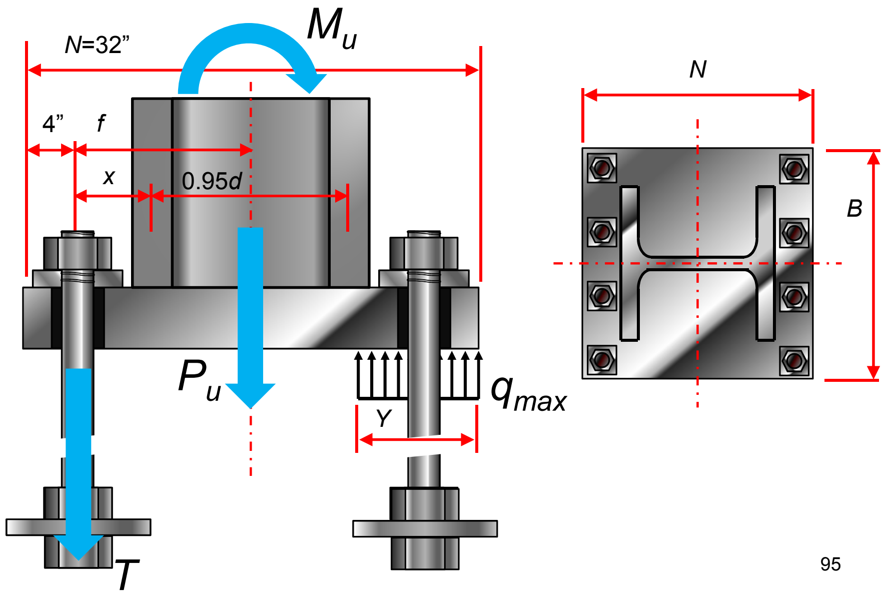
Base-plate design



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Base-plate design

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

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”, <u>1.81”</u>
- $T = q_{max}Y + P_u$ =141K/in*1.81”+59.3K=314K

78.6K/rod

AISC Design Guide 1 §3.4

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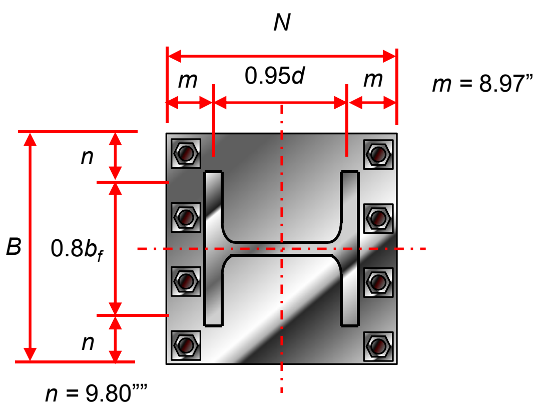
Base-plate design



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Base-plate design

- Flexure at the bearing side
- $t_p \geq 2.11\sqrt{\frac{f_{pmax}Y\left[max(m,n)-Y/_2\right]}{F_y}}$
- $t_p \geq 2.11\sqrt{\frac{4.42ksi(1.81")\left[9.80"-1.81"/_2\right]}{50ksi}} = 2.51"$
- Use 2¾” plate

AISC Design Guide 1 §3.4

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Base-plate design

- Flexure at the tension side
  - $x = f - d/2 + t_f/2 = 5.15”$
  - $t_p \geq 2.11\sqrt{\frac{Tx}{BF_y}}$= 2.1” Use 2¾” plate

AISC Design Guide 1 §3.4

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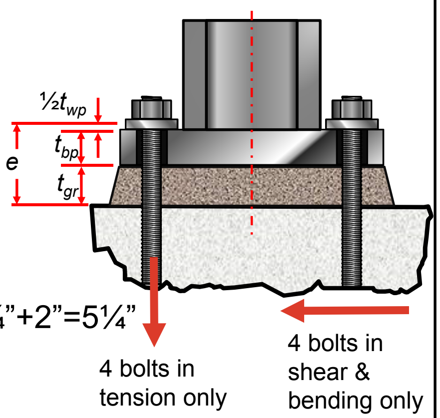
Anchor-rod design

- Shear
  - V_u =85.3K/4bolts
    = 21.4K/bolt
- Moment
  - $e = ½\, t_{wp} + t_{bp} + t_{gr}$
    - e = ½(1"[assumed])+ 2¾"+2"=5¼"
  - M_u = ½ $V_u\, e$
    = 56.2"K



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Anchor-rod design (tension)

- Check anchor rod
  - Loads
    - M_u = 0"K
    - T_u = 78.6K
    - V_u = 0K
  - Properties
    - d = 1½"
    - A = 1.77in²
    - A_{net} = 1.41in²=0.795A
      - Table 7-17
    - Z = 0.563in³
  - Stresses
    - $F_{nt} = 0.795F_u = 59.7\text{ksi}$
    - $F_{nv} = 0.45F_u = 33.8\text{ksi}$
    - $f_v = \frac{V_u}{A} = 0ksi$
    - $f_a = \frac{T_u}{A} = 44.5ksi$
    - $f_b = \frac{M_u}{Z} = 0ksi$
    - $f_t = f_a + f_b = 44.5ksi$
    - $F'_{nt} = \left[1.3 - \frac{f_v}{\emptyset F_{nv}}\right] F_{nt} \le F_{nt}$
    - $\emptyset F'_{nt} = (0.75)59.7ksi = 44.7ksi$
    - $f_t \le \emptyset F'_{nt}$ 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 = 0K
    - V_u = 21.4K
  - Properties
    - d = 2"
    - A = πin²
    - A_{net} = 2.50in²=0.795A
      - Table 7-17
    - Z = 1.33in³
  - Stresses
    - $F_{nt} = 0.795F_u = 59.7\text{ksi}$
    - $F_{nv} = 0.45F_u = 33.8\text{ksi}$
    - $f_v = \frac{V_u}{A} = 3.4ksi$
    - $f_a = \frac{T_u}{A} = 0ksi$
    - $f_b = \frac{M_u}{Z} = 42.1ksi$
    - $f_t = f_a + f_b = 42.1ksi$
    - $F'_{nt} = \left[1.3 - \frac{f_v}{\emptyset F_{nv}}\right] F_{nt} \leq F_{nt}$
    - $\emptyset F'_{nt} = (0.75)59.7ksi = 44.7ksi$
    - $f_t \leq \emptyset F'_{nt}$ OK

AISC 341 §J3.7

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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

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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
    - $f_p = \emptyset 0.85 f'_c = (0.65)0.85(4ksi) = 2.2ksi$
    - $f_p B t_p$ = 2.2ksi*32"*3" = 211K
  - Friction
    - $C\mu$ = (141K/in*1.81")(0.4)=102K   ACI 349

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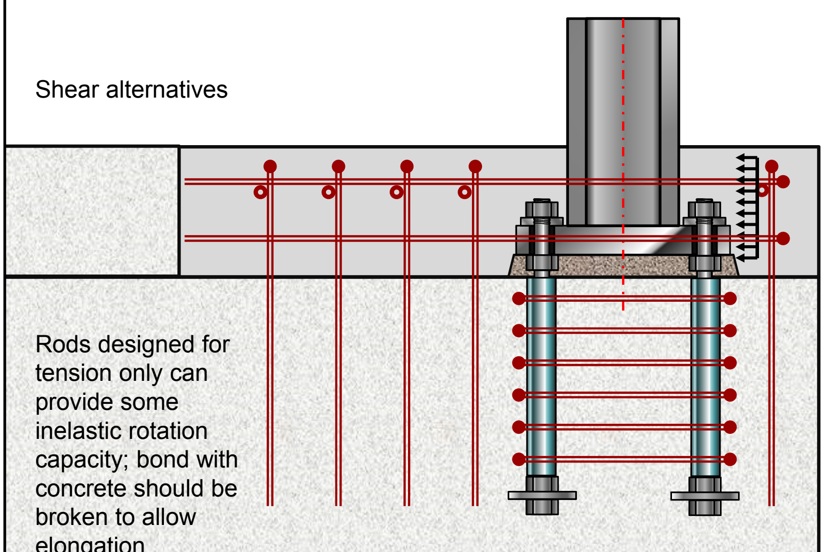
Anchor-rod design

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
1 1/2	2 5/16	3 1/2	5/8
1 3/4	2 3/4	4	5/8
2	3 1/4	5	3/4
2 1/2	3 3/4	5 1/2	7/8

Notes:
1. Circular or square washers meeting the washer size are acceptible.
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 1 1/4 in. thick, punching of holes may be an economical option. In this case, 3/4-in. anchor rods and 1 1/16-in.-diameter punched holes may be used with ASTM F844 (USS Standard) washers in place of fabricated plate washers.

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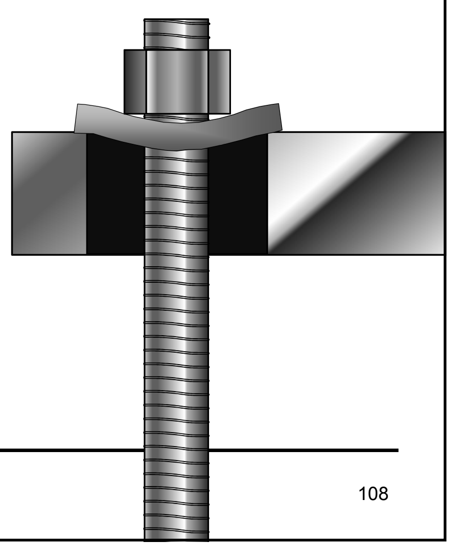
Anchor-rod design

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



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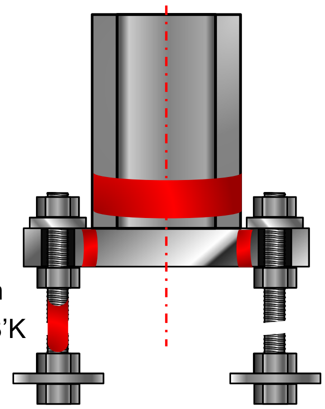
Base connection design

- 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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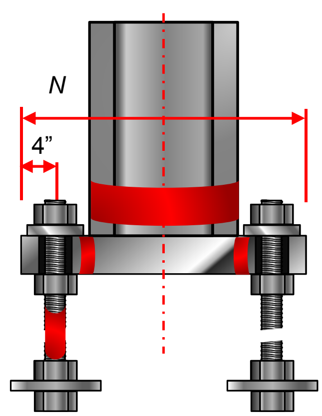
Base connection design

- Column strength
  - $R_yF_yZ_x$ = 1191'K
- Anchor rods
  - $M_y \sim \Sigma AdF_y \sim \Sigma A(N-2*4")F_y + M_c$
    - M_y ~1060'K (1½" rods); 1340'K @ F_u
    - M_y ~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 ~ 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 = $0.64t_w$

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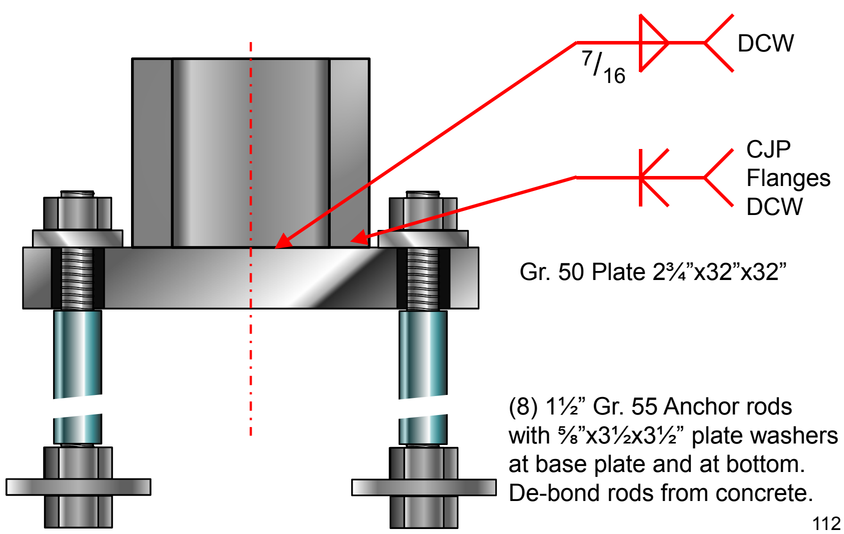
Base connection design



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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
  - o Design for drift first
  - o 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

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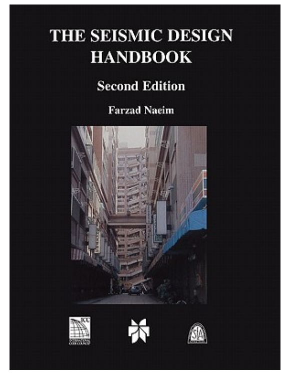
Additional resources



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Question time

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- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

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

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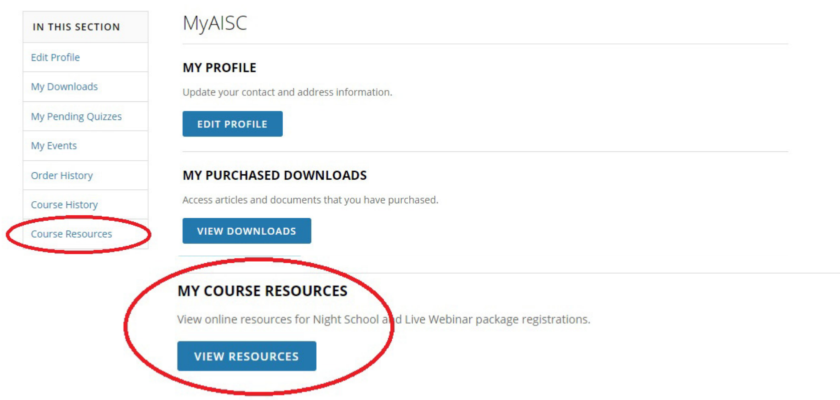

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Thank You

Please give us your feedback!

Survey at conclusion of webinar.

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