

# S.E. Exam Review: Steel Design

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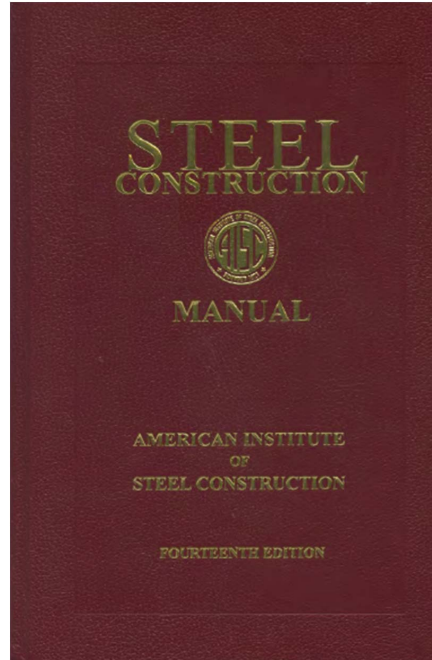
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## SCM – Steel Construction Manual

- Fourteenth Edition
- ASCE 360 – Specification for Structural Steel Buildings



## NCEES Topics

- Steel Construction Manual
- LRFD
- Beams
- Columns
- Members subjected to combined forces
- Column base plates
- Beam bracing
- Connections

## Design for Stability

### Chapter C – Design for Stability

Chapter C presents requirements for design for stability

Primary method is the *Direct Analysis Method* of design

Advantage is use  $K = 1$

Design → determination of required strength of components and proportioning the components to have adequate available strength.

## Chapter C – Design for Stability

Appendix 7 presents alternate methods:

- a) effective length method
- b) first order analysis method

Appendix 8 presents methods of approximate second order analysis.

Unless noted otherwise, all loads in this presentation are required loads computed using one of the approved methods in Chapter C.

## Chapter C – Design for Stability

C1 – General Stability Requirements

C2 – Calculation of Required Strengths

C3 – Calculation of Available Strengths

## Direct Analysis Method

### Section C1 – General Stability Requirements

Stability shall be provided for the structure as a whole and each of its elements

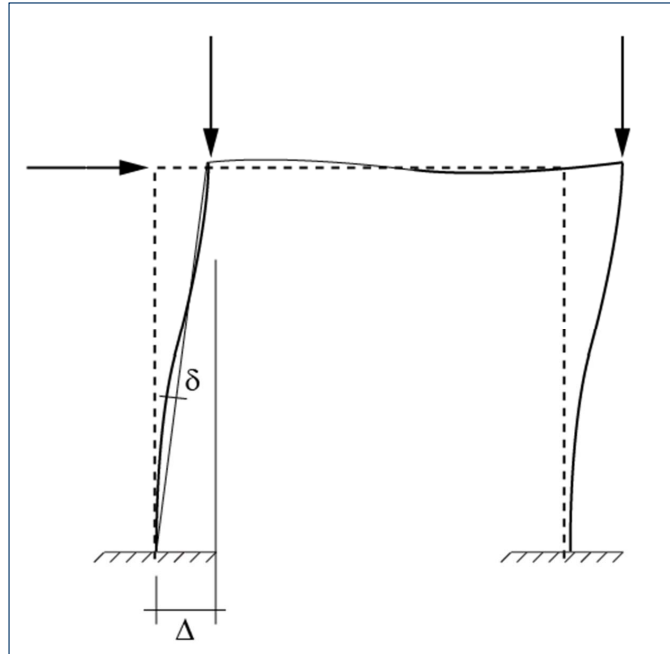
Flexural, shear and axial member deformations and all other deformations that contribute to the displacement of the structure must be *considered*

Second-order effects (both  $P-\Delta$  and  $P-\delta$ )

Geometric imperfections

Stiffness reduction due to inelasticity

## Second-order Effects



### Section C1 – General Stability Requirements

Any rational method that considers these effects is permitted

All load-dependent effects are calculated using LRFD load combinations or 1.6 times ASD load combinations

Uncertainty in strength and stiffness

For strength uncertainty, resistance factor ( $\phi$ ) or safety factor ( $\Omega$ )

## Section C2 – Calculation of Required Strengths

Consider flexural, shear and axial member deformations, and all other component and *connection* deformations that contribute to the displacement of the structure

Use any *second-order* analysis that considers  $P-\Delta$  and  $P-\delta$  effects

$P-\delta$  effects may be ignored under certain circumstances (See Section C2.1(2))

## Section C2 – Calculation of Required Strengths

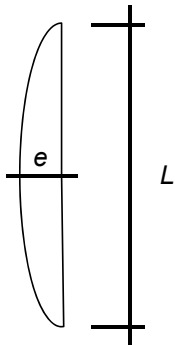
Include all gravity and other applied loads that influence the stability of the structure

Load-dependent effects are calculated using the LRFD load combinations or 1.6 times ASD load combinations

If using ASD, divide analysis results by 1.6 to obtain required strengths

## Consideration of Initial Imperfections

### Initial Imperfections



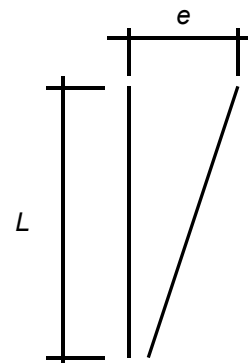
Out-of-straightness

ASTM A6 tolerance  $L / 1000$

Chapter E

Out-of-plumbness

Code of Standard Practice tolerance  
 $L / 500$





## Initial Imperfections

### C2.2 Consideration of Initial Imperfections

“The effect of initial imperfections on the stability of the structure shall be taken into account either by direct modeling of imperfections in the analysis as specified in Section 2.2a or by application of notional loads as specified in Section 2.2b”

C2.2a - Direct modeling

C2.2b - Notional loads

## Use of Notional Loads to Represent Imperfections

Permitted for structures that support gravity loads primarily through nominally-vertical columns, walls or frames

Use nominal geometry

Distributed in same manner as gravity load

Applied in the direction that provides the greatest destabilizing effect

## Use of Notional Loads to Represent Imperfections

$$N_i = 0.002\alpha Y_i$$

$N_i$  → notional load applied at level  $i$

$Y_i$  → gravity load applied at level  $i$  from LRFD load combination or ASD load combination

$\alpha = 1.0$  (LRFD);  $\alpha = 1.6$  (ASD)

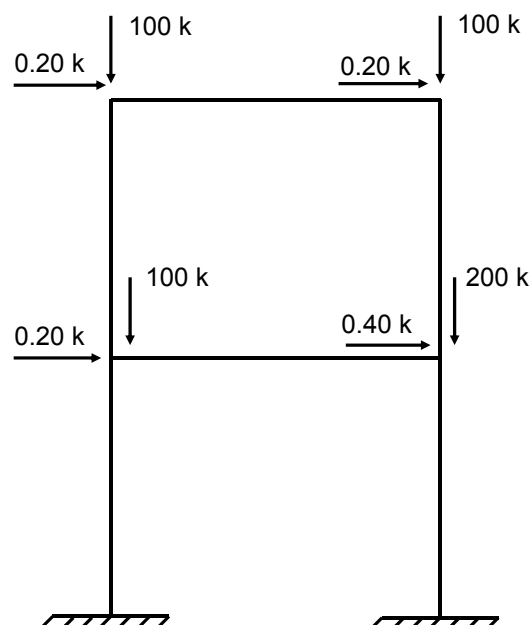
If  $\frac{\Delta_{2nd-order}}{\Delta_{1st-order}} (B_2) \leq 1.7$  in all stories, notional loads may be applied only in gravity load combinations and not in combinations that include other lateral loads.

## Notional Loads

This is a gravity only LC

$$\begin{aligned} N_1 &= 0.002(1.0)(100 \text{ k} + 200 \text{ k}) \\ &= 0.6 \text{ k} \end{aligned}$$

$$\begin{aligned} N_2 &= 0.002(1.0)(100 \text{ k} + 100 \text{ k}) \\ &= 0.4 \text{ k} \end{aligned}$$



## Adjustment to Stiffness

For stiffnesses that contribute to stability of the structure

Optionally applicable to all members

Direct adjustment – C2.3(1), (2)

Notional loads – C2.3(3)

Notional loads combine with notional loads from imperfections

## Stiffness Reduction – C2.3(1), (2)

Direct adjustment

$$EA^* = 0.8EA$$

$$EI^* = 0.8E\tau_b I$$

$$\alpha P_r / P_y \leq 0.5 \quad \tau_b = 1.0$$

$$\alpha P_r / P_y > 0.5 \quad \tau_b = 4(\alpha P_r / P_y)[1 - (\alpha P_r / P_y)]$$

$$\alpha = 1.0 \text{ (LRFD)}; \alpha = 1.6 \text{ (ASD)}$$

$P_r$  → required axial compressive strength using LRFD or ASD load combinations

$P_y$  → axial yield strength (=  $F_y A_g$ )

## Stiffness Reduction – C2.3(3)

Gravity loads supported primarily by nominally-vertical columns, walls or frames

Notional loads

For  $\frac{\alpha P_r}{P_y} > 0.5$  may use  $\tau_b = 1$

If notional load =  $0.001\alpha Y_i$  is applied at all levels for all load combinations

$Y_i \rightarrow$  gravity load applied at level  $i$  from LRFD load combination or ASD load combination

## Section C3 – Calculation of Available Strengths

Follow provisions of Chapters  $D$  through  $K$  with no further consideration of overall structural stability.

$K = 1$  unless smaller value can be justified.

Bracing requirements of Appendix 6 are not applicable to bracing that is included as part of the overall force resisting system.

Chapters  $D$  through  $K$

## Tension Members

### Tension Members – Chapter D

Yield limit state

Fracture limit state

- Net area
- Effective net area

Block shear rupture limit state

## Block Shear Rupture Strength – Chapter J

ASCE 360 Section J4.3 Page 16.1-129

Available (design) strength =  $\phi R_n$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

$$\phi = 0.75$$

$U_{bs} = 1$  when the tensile stress is uniform

$U_{bs} = 0.5$  when the tensile stress is non-uniform

$A_{gv}$  = gross area subject to shear, in<sup>2</sup>

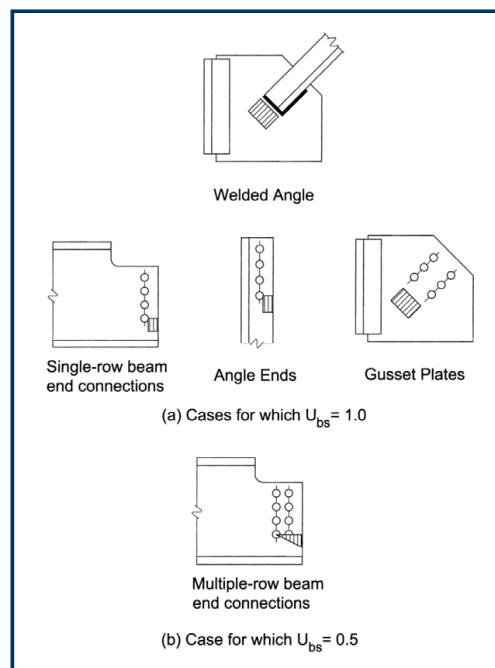
$A_{gt}$  = gross area subject to tension, in<sup>2</sup>

$A_{nv}$  = net area subject to shear, in<sup>2</sup>

$A_{nt}$  = net area subject to tension, in<sup>2</sup>

$U_{bs}$

ASCE 360 Commentary  
Figure C-J4.2  
Page 16.1-412

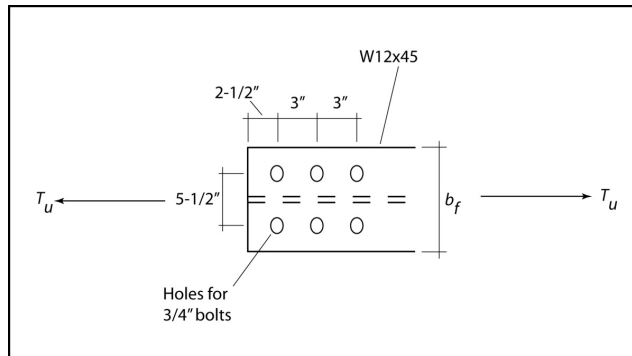


## Block Shear Rupture Example

Determine if a W12x45 of A992 steel is adequate for the following loads:  $D = 90\text{ k}$ ,  $L = 130\text{ k}$ ,  $W = 145\text{ k}$ .

The member is connected to gusset plates as shown.

Use  $3/4$  inch bolts – 4 rows, 3 bolts each row.



## Block Shear Rupture Example

Determine the maximum load effect

$$1.4D = 1.4(90) = 126\text{ k}$$

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) =$$

$$1.2(90\text{ k}) + 1.6(130\text{ k}) = 316\text{ k}$$

$$1.2D + 1.0W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$$

$$= 1.2(90\text{ k}) + 1.0(145\text{ k}) + 0.5(130\text{ k}) =$$

$$318\text{ k} \leftarrow \text{Controls}$$

See footnote 1, page 2-10 of SCM for more information on the load factor to use with  $L$ .

## Page 1-26 SCM

$$A = 13.1 \text{ in}^2$$

$$b_f = 8.05 \text{ in}$$

$$t_f = 0.575 \text{ in}$$

Table 1-1 (continued)  
W Shapes  
Dimensions

Shape	Area, A		Depth, d	Web		Flange		Distance					Workable Edge	
	in. <sup>2</sup>	in.		Thickness, t <sub>w</sub>	L <sub>z</sub> /2	Width, b <sub>f</sub>	Thickness, t <sub>f</sub>	k	k <sub>1</sub>	k <sub>2</sub>	k <sub>3</sub>	k <sub>4</sub>		
W12×58	17.0	12.2	12 1/4	0.360	3/8	10.0	10	0.640	3/8	1.24	1 1/2	1 1/8	9 1/4	5 1/2
×52	15.6	12.1	12	0.345	3/8	10.0	10	0.575	3/8	1.18	1 1/2	1 1/8	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	8.08	8 1/8	0.640	3/8	1.14	1 1/2	1 1/8	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	3/8	8.05	8	0.575	3/8	1.08	1 1/2	1 1/8	9 1/4	5 1/2
×40	11.7	11.9	12	0.295	3/8	8.01	8	0.515	3/8	1.02	1 1/2	1 1/8	9 1/4	5 1/2
W12×35 <sup>c</sup>	10.3	12.5	12 1/2	0.300	5/16	6.56	6 1/2	0.520	3/8	0.820	1 1/8	1 1/8	10 1/8	3 1/2
×30 <sup>c</sup>	8.79	12.3	12 3/8	0.260	1/4	6.52	6 1/2	0.440	3/8	0.740	1 1/8	1 1/8	10 1/8	3 1/2
×26 <sup>c</sup>	7.65	12.2	12 1/4	0.230	1/4	6.49	6 1/8	0.380	3/8	0.680	1 1/8	1 1/8	10 1/8	3 1/2
W12×22 <sup>c</sup>	6.48	12.3	12 1/4	0.260	1/4	4.03	4	0.425	3/8	0.725	1 1/8	1 1/8	10 1/8	2 1/4 <sup>d</sup>
×19 <sup>c</sup>	5.57	12.2	12 1/4	0.235	1/4	4.01	4	0.350	3/8	0.650	1 1/8	1 1/8	10 1/8	2 1/4 <sup>d</sup>
×16 <sup>c</sup>	4.71	12.0	12	0.220	1/4	3.99	4	0.265	3/8	0.565	1 1/8	1 1/8	10 1/8	2 1/4 <sup>d</sup>
×14 <sup>c</sup>	4.16	11.9	11 5/8	0.205	3/8	3.97	4	0.225	3/8	0.525	1 1/8	1 1/8	10 1/8	2 1/4 <sup>d</sup>
W10×112	32.9	11.4	11 1/4	0.755	3/4	10.4	10 1/8	1.25	1/2	1.75	1 1/2	1 1/2	7 1/2	5 1/2
×100	29.8	11.3	11 1/4	0.690	3/4	10.3	10 1/8	1.10	1/2	1.62	1 1/2	1 1/2	7 1/2	5 1/2
×90	27.2	11.2	11 1/4	0.640	3/4	10.2	10 1/8	1.00	1/2	1.49	1 1/2	1 1/2	7 1/2	5 1/2
×84	25.3	11.1	11 1/4	0.600	3/4	10.1	10 1/8	0.92	1/2	1.37	1 1/2	1 1/2	7 1/2	5 1/2
×80	23.8	11.0	11 1/4	0.570	3/4	10.0	10 1/8	0.85	1/2	1.27	1 1/2	1 1/2	7 1/2	5 1/2
×76	22.5	10.9	11 1/4	0.545	3/4	9.9	10 1/8	0.78	1/2	1.18	1 1/2	1 1/2	7 1/2	5 1/2
×72	21.3	10.8	11 1/4	0.520	3/4	9.8	10 1/8	0.72	1/2	1.12	1 1/2	1 1/2	7 1/2	5 1/2
×68	20.2	10.7	11 1/4	0.500	3/4	9.7	10 1/8	0.66	1/2	1.06	1 1/2	1 1/2	7 1/2	5 1/2
×64	19.1	10.6	11 1/4	0.480	3/4	9.6	10 1/8	0.60	1/2	1.00	1 1/2	1 1/2	7 1/2	5 1/2
×60	18.1	10.5	11 1/4	0.460	3/4	9.5	10 1/8	0.55	1/2	0.935	1 1/2	1 1/2	7 1/2	5 1/2
×56	17.1	10.4	11 1/4	0.440	3/4	9.4	10 1/8	0.50	1/2	0.880	1 1/2	1 1/2	7 1/2	5 1/2
×52	16.1	10.3	11 1/4	0.420	3/4	9.3	10 1/8	0.45	1/2	0.810	1 1/2	1 1/2	7 1/2	5 1/2
×48	15.1	10.2	11 1/4	0.400	3/4	9.2	10 1/8	0.40	1/2	0.740	1 1/2	1 1/2	7 1/2	5 1/2
×44	14.1	10.1	11 1/4	0.380	3/4	9.1	10 1/8	0.35	1/2	0.660	1 1/2	1 1/2	7 1/2	5 1/2
×40	13.1	10.0	11 1/4	0.360	3/4	9.0	10 1/8	0.30	1/2	0.580	1 1/2	1 1/2	7 1/2	5 1/2
×36	12.1	9.9	11 1/4	0.340	3/4	8.9	10 1/8	0.25	1/2	0.510	1 1/2	1 1/2	7 1/2	5 1/2
×32	11.1	9.8	11 1/4	0.320	3/4	8.8	10 1/8	0.20	1/2	0.440	1 1/2	1 1/2	7 1/2	5 1/2
×28	10.1	9.7	11 1/4	0.300	3/4	8.7	10 1/8	0.15	1/2	0.370	1 1/2	1 1/2	7 1/2	5 1/2
×24	9.1	9.6	11 1/4	0.280	3/4	8.6	10 1/8	0.10	1/2	0.300	1 1/2	1 1/2	7 1/2	5 1/2
×20	8.1	9.5	11 1/4	0.260	3/4	8.5	10 1/8	0.05	1/2	0.230	1 1/2	1 1/2	7 1/2	5 1/2
×16	7.1	9.4	11 1/4	0.240	3/4	8.4	10 1/8	0.00	1/2	0.160	1 1/2	1 1/2	7 1/2	5 1/2
×12	6.1	9.3	11 1/4	0.220	3/4	8.3	10 1/8	0.00	1/2	0.090	1 1/2	1 1/2	7 1/2	5 1/2
×8	5.1	9.2	11 1/4	0.200	3/4	8.2	10 1/8	0.00	1/2	0.020	1 1/2	1 1/2	7 1/2	5 1/2
×4	4.1	9.1	11 1/4	0.180	3/4	8.1	10 1/8	0.00	1/2	0.00	1 1/2	1 1/2	7 1/2	5 1/2

<sup>c</sup> Shape is standard for compression with F<sub>y</sub> = 50 ksi.  
<sup>d</sup> Shape exceeds compact limit for flexure with F<sub>y</sub> = 50 ksi.  
<sup>e</sup> The actual size, combination, and orientation of flange components should be compared with the geometry of the cross-section to ensure compatibility.  
<sup>f</sup> Shape does not meet the A<sub>t</sub> limit for shear in Specification Section G2.1a with F<sub>y</sub> = 50 ksi.

×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15/16	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/8	1.14	1 1/2	15/16	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 3/8	15/16	9 1/4	5 1/2
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 3/8	7/8	9 1/4	5 1/2
W12×35 <sup>c</sup>	10.3	12.5	12 1/2	0.300	5/16	3/16	6.56	6 1/2	0.520	1/2	0.820	1 3/16	3/4	10 1/8	3 1/2
×30 <sup>c</sup>	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1 1/8	3/4	10 1/8	3 1/2

## Block Shear Rupture Example

$$F_y = 50 \text{ ksi}$$

$$F_u = 65 \text{ ksi}$$

$$\text{Yield limit state: } \phi_t T_n = \phi_t A_g F_y = (0.9)(13.1 \text{ in}^2)(50 \text{ ksi}) = 590 > 318 \text{ k OK}$$

$$A_e = U A_n$$

$$A_n = A_g - 4d_{\text{beff}} t_f = 13.1 \text{ in}^2 - 4 \left( \frac{3}{4} \text{ in} + \frac{1}{8} \text{ in} \right) (0.575 \text{ in}) = 11.09 \text{ in}^2$$

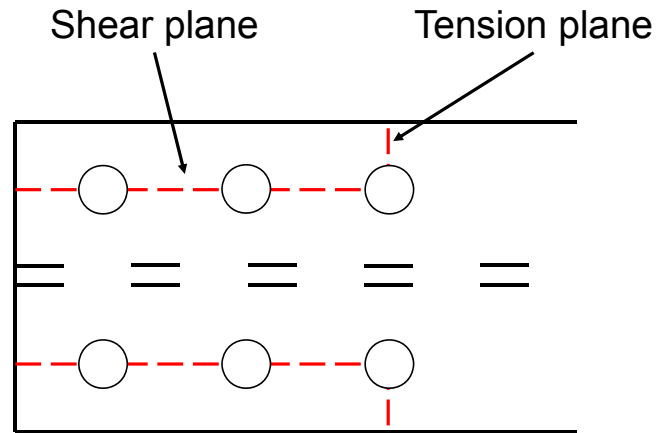
$$U = 1$$

$$A_e = 11.09 \text{ in}^2$$

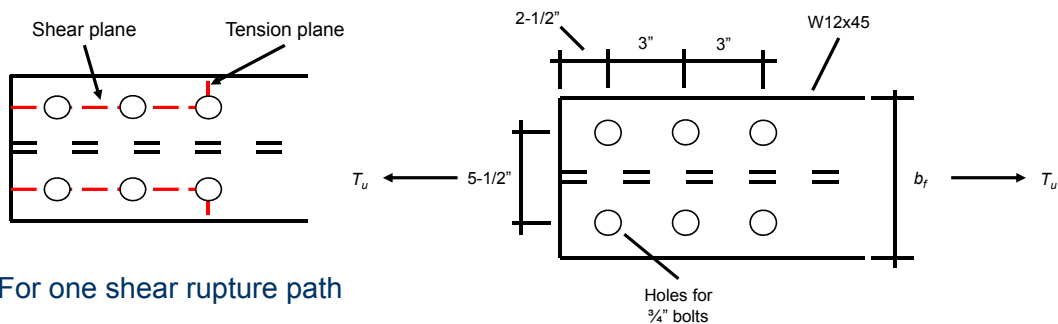
$$\text{Fracture limit state: } \phi_t T_n = \phi_t A_e F_u = (0.75)(11.09 \text{ in}^2)(65 \text{ ksi}) = 541 \text{ k} > 318 \text{ k OK}$$



## Block Shear Rupture Example



## Block Shear Rupture Example



For one shear rupture path

$$A_{gt} = \left( \frac{8.05 \text{ in} - 5.5 \text{ in}}{2} \right) (0.575 \text{ in})$$

$$= 0.7331 \text{ in}^2$$

$$A_{gv} = (2.5 \text{ in} + 3 \text{ in} + 3 \text{ in}) (0.575 \text{ in}) = 4.887 \text{ in}^2$$

$$A_{nt} = 0.7331 \text{ in}^2 - (0.5) \left( 0.75 \text{ in} + \frac{1}{8} \text{ in} \right) (0.575 \text{ in}) = 0.4815 \text{ in}^2$$

$$A_{nv} = 4.887 \text{ in}^2 - (2.5) \left( 0.75 \text{ in} + \frac{1}{8} \text{ in} \right) (0.575 \text{ in}) = 3.629 \text{ in}^2$$

## Block Shear Rupture Example

$U_{bs} = 1$  (see Commentary Figure C-J4.2 Page 16.1-412)

There are four BSR planes; calculate for one plane and multiply by four:

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

$$0.6F_uA_{nv} + U_{bs}F_uA_{nt} = 0.6(65 \text{ ksi})(3.629 \text{ in}^2) + (1)(65 \text{ ksi})(0.4815 \text{ in}^2) = 172.8 \text{ k controls}$$

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(50 \text{ ksi})(4.887 \text{ in}^2) + (1)(65 \text{ ksi})(0.4815 \text{ in}^2) = 177.9 \text{ k}$$

$$\phi_t R_n = 0.75(4)(172.8 \text{ k}) = 518.4 \text{ k} > 318 \text{ k OK}$$

## ASD/LRFD Comparison

## ASD/LRFD Comparison

Compare for the limit state of tension yield

Nominal strength ( $P_n$ ) calculation is identical for both methods

LRFD

$$\phi_t P_n = 0.9 P_n$$

ASD

$$\frac{P_n}{\Omega_t} = \frac{P_n}{1.67}$$

$$\frac{1}{\Omega_t} \rightarrow \phi_t$$

Assume an average LRFD load factor of 1.5

$$\frac{1.5}{1.67} = 0.898 \approx 0.9 = \phi_t$$

## Members Subjected to Combined Forces and Torsion – Chapter H

## Combined Forces Notation

$P_r$  → required axial strength

$P_n$  → nominal axial strength

$P_c$  → design axial strength ( $= \phi_c P_n$ )

$\phi_c$  → resistance factor for compression ( $= 0.90$ )

$M_r$  → required flexural strength

$M_n$  → nominal flexural strength

$M_c$  → design flexural strength ( $= \phi_b M_n$ )

$\phi_b$  → resistance factor for flexure ( $= 0.90$ )

$x$  → subscript relating symbol for strong axis of bending

$y$  → subscript relating symbol for weak axis of bending

## LRFD Combined Forces Provisions

$$\frac{P_r}{P_c} + \frac{8}{9} \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \text{ when } \frac{P_r}{P_c} \geq 0.2 \text{ (ASCE 360 H1-1a)}$$

$$\frac{P_r}{2P_c} + \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \text{ when } \frac{P_r}{P_c} < 0.2 \text{ (ASCE 360 H1-1b)}$$

Equation H1-1b controls for members with relatively small axial load

## LRFD Combined Forces Provisions

Inequalities H1-1a and H1-1b may be written, respectively:

$$pP_r + b_x M_{rx} + b_y M_{ry} \leq 1.0$$

$$\frac{1}{2}pP_r + \frac{9}{8}(b_x M_{rx} + b_y M_{ry}) \leq 1.0$$

where:

$$p = \frac{1}{\phi_c P_n}$$

$$b_x = \frac{8}{9(\phi_b M_{nx})}$$

$$b_y = \frac{8}{9(\phi_b M_{ny})}$$

Table 6-1

Caveats:

$$F_y = 50 \text{ ksi}$$

$$C_b = 1$$

Column buckling  
about the y axis

Values of  $p$ ,  $b_x$  and  $b_y$  are tabulated in SCM Table 6-1, Pages 6-5

## LRFD Combined Forces Provisions

Confirm that y-axis buckling controls. If x axis buckling controls, enter Table 6-1 with  $(KL)_{y\text{-equiv}} = (KL)_x / (r_x / r_y)$ .

If  $C_b > 1$ , adjust the tabular value of  $b_x$  by dividing by  $C_b$ . The adjusted value of  $b_x$  may not be less than that corresponding to  $\phi_b M_{nx} = \phi_b M_{px}$ , or the minimum value of  $b_x$  found in Table 6-1. Using  $C_b = 1$  is conservative.

## Combined Forces Example 1

Determine if a W14x99 section of A992 steel is adequate to resist the following factored loads:  $P_r = 400\text{ k}$ ,  $M_{rx} = 250\text{ k-ft}$ ,  $M_{ry} = 80\text{ k-ft}$ . Use  $(KL)_x = 20\text{ ft}$ ,  $(KL)_y = 14\text{ ft}$ ,  $L_b = 12\text{ ft}$ ,  $C_b = 1.0$ .

Table 6-1, Page 6-70

**Table 6-1 (continued)**  
**Combined Flexure and Axial Force**  
 $F_y = 50\text{ ksi}$   
**W-Shapes**

W14x99

Shape	$(KL)_x = 10\text{ ft}$						$(KL)_x = 20\text{ ft}$						$(KL)_x = 30\text{ ft}$					
	$p \times 10^3$		$\Delta_y \times 10^3$		$p \times 10^3$		$\Delta_y \times 10^3$		$p \times 10^3$		$\Delta_y \times 10^3$		$p \times 10^3$		$\Delta_y \times 10^3$			
Design	(kips) <sup>2</sup>		(kip-ft) <sup>-1</sup>		(kips) <sup>-1</sup>		(kip-ft) <sup>-1</sup>		(kips) <sup>-1</sup>		(kip-ft) <sup>-1</sup>		(kips) <sup>-1</sup>		(kip-ft) <sup>-1</sup>			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD		
10	3.64	0.894	1.86	1.23	1.15	0.794	2.07	1.38	1.26	0.839	2.33	1.55						
11	1.14	0.761	1.86	1.23	1.26	0.838	2.07	1.38	1.38	0.920	2.33	1.55						
12	1.16	0.774	1.86	1.23	1.26	0.853	2.07	1.38	1.41	0.957	2.33	1.55						
13	1.19	0.789	1.86	1.23	1.31	0.869	2.07	1.38	1.44	0.955	2.33	1.55						
14	1.21	0.805	1.87	1.23	1.33	0.887	2.08	1.38	1.47	0.975	2.33	1.55						
15	1.24	0.823	1.89	1.26	1.36	0.907	2.10	1.40	1.50	0.997	2.33	1.55						
16	1.27	0.843	1.91	1.27	1.40	0.929	2.13	1.42	1.53	1.02	2.35	1.57						
17	1.30	0.864	1.93	1.29	1.43	0.953	2.15	1.43	1.57	1.05	2.38	1.59						
18	1.32	0.884	1.94	1.29	1.45	0.974	2.17	1.44	1.60	1.07	2.40	1.60						
19	1.37	0.913	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.43	1.63						

$p = 0.887 \times 10^{-3}$

Shape	$p \times 10^3$		$\Delta_y \times 10^3$		$p \times 10^3$		$\Delta_y \times 10^3$		$p \times 10^3$		$\Delta_y \times 10^3$	
Design	(kips) <sup>2</sup>		(kip-ft) <sup>-1</sup>		(kips) <sup>-1</sup>		(kip-ft) <sup>-1</sup>		(kips) <sup>-1</sup>		(kip-ft) <sup>-1</sup>	
12	1.19	0.789	1.86	1.23	1.28	0.853	2.07	1.38	1.41	0.937	2.33	1.55
13	1.21	0.789	1.86	1.23	1.31	0.869	2.07	1.38	1.44	0.955	2.33	1.55
14	1.21	0.805	1.87	1.25	1.33	0.887	2.08	1.38	1.47	0.975	2.33	1.55
15	1.24	0.823	1.89	1.26	1.36	0.907	2.10	1.40	1.50	0.997	2.33	1.55
16	1.27	0.843	1.91	1.27	1.40	0.929	2.13	1.42	1.53	1.02	2.35	1.57
17	1.30	0.864	1.93	1.29	1.43	0.953	2.15	1.43	1.57	1.05	2.38	1.59

$b_y = 2.85 \times 10^{-3}$

Shape	$p \times 10^3$		$\Delta_y \times 10^3$		$p \times 10^3$		$\Delta_y \times 10^3$	
Design	(kips) <sup>2</sup>		(kip-ft) <sup>-1</sup>		(kips) <sup>-1</sup>		(kip-ft) <sup>-1</sup>	
46	4.30	3.83	2.00	1.88	3.06	3.37	3.31	2.20
48	4.05	3.30	2.02	1.94	3.01	3.67	3.49	2.32
50	5.38	3.58	3.05	2.03	3.98	3.98	3.96	2.43

Other Constants and Properties

Shape	$\lambda_p$	$\lambda_c$	$\lambda_{p1}$	$\lambda_{p2}$	$\lambda_{c1}$	$\lambda_{c2}$
W14x99	3.94	2.56	4.29	2.85	3.94	2.56
W14x90	1.04	0.894	1.15	0.907	1.26	0.839
W14x82	1.28	0.855	1.41	0.940	1.55	0.939

Shape does not meet compact limit for flexure with  $F_y = 50\text{ ksi}$ .

Table 6-1, Page 6-70

Table 6-1 (continued)  
Combined Flexure and Axial Force  
W-Shapes  
 $F_y = 50$  ksi

W14

W14x99

$b_x = 1.38 \times 10^{-3}$

$r_x/r_y = 1.66$

Shape	100				90				90				
	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	
Design	(kips) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kips) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kips) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kips) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kips) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kips) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	
ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1.04	0.884	1.06	1.23	1.15	0.764	2.07	1.38	1.26	0.859	2.33	1.35	
11	1.14	0.781	1.00	1.23	1.20	0.830	2.07	1.38	1.38	0.920	2.33	1.35	
12	1.16	0.774	1.00	1.23	1.20	0.853	2.07	1.38	1.41	0.955	2.33	1.35	
13	1.19	0.789	1.00	1.23	1.31	0.869	2.07	1.38	1.44	0.955	2.33	1.35	
14	1.21	0.805	1.07	1.25	1.33	0.887	2.08	1.38	1.47	0.975	2.33	1.35	
15	1.24	0.823	1.09	1.26	1.36	0.907	2.10	1.40	1.50	0.997	2.33	1.35	
16	1.27	0.843	1.91	1.27	1.40	0.929	2.13	1.42	1.53	1.02	2.35	1.37	
17	1.30	0.864	1.93	1.29	1.43	0.953	2.15	1.43	1.57	1.05	2.38	1.39	
18	1.33	0.887	1.93	1.29	1.43	0.953	2.15	1.43	1.57	1.05	2.38	1.39	
19	1.37	0.913	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
20	1.40	0.937	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
21	1.44	0.961	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
22	1.48	0.985	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
23	1.52	1.009	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
24	1.56	1.033	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
25	1.60	1.057	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
26	1.64	1.081	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
27	1.68	1.105	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
28	1.72	1.129	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
29	1.76	1.153	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
30	1.80	1.177	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
31	1.84	1.201	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
32	1.88	1.225	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
33	1.92	1.249	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
34	1.96	1.273	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
35	2.00	1.297	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
36	2.04	1.321	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
37	2.08	1.345	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
38	2.12	1.369	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
39	2.16	1.393	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
40	2.20	1.417	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
41	2.24	1.441	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
42	2.28	1.465	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
43	2.32	1.489	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
44	2.36	1.513	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
45	2.40	1.537	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
46	2.44	1.561	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
47	2.48	1.585	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
48	2.52	1.609	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
49	2.56	1.633	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	
50	2.60	1.657	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.43	

Other Constants and Properties

$\lambda \times 10^3$ (kip-ft) <sup>-1</sup>	3.64	2.56	4.23	2.85	4.90	3.26
$\lambda \times 10^3$ (kips) <sup>-1</sup> <td>1.04</td> <td>0.694</td> <td>1.15</td> <td>0.764</td> <td>1.26</td> <td>0.830</td>	1.04	0.694	1.15	0.764	1.26	0.830
$\lambda \times 10^3$ (kips) <sup>-1</sup> <td>1.28</td> <td>0.855</td> <td>1.43</td> <td>0.953</td> <td>1.57</td> <td>1.03</td>	1.28	0.855	1.43	0.953	1.57	1.03
$r_x/r_y$ <td>1.67</td> <td>1.67</td> <td>1.67</td> <td>1.66</td> <td>1.66</td> <td>1.66</td>	1.67	1.67	1.67	1.66	1.66	1.66
$r_x$ , in. <td>3.73</td> <td>3.73</td> <td>3.73</td> <td>3.73</td> <td>3.70</td> <td>3.70</td>	3.73	3.73	3.73	3.73	3.70	3.70

Shape does not meet compact limit for flexure with  $F_y = 50$  ksi.

## Combined Forces Example 1

From Table 6-1, Pages 6-70, for a W14x99;  $p = 0.887 \times 10^{-3}$ ;  $b_x = 1.38 \times 10^{-3}$ ;  $b_y = 2.85 \times 10^{-3}$

$$\frac{r_x}{r_y} = 1.66 > \frac{(KL)_x}{(KL)_y} = \frac{20 \text{ ft}}{14 \text{ ft}} = 1.429 \therefore y \text{ axis buckling controls}$$

Determine which equation applies:

$$p = \frac{1}{\phi_c P_n} = \frac{1}{P_c} = 0.887 \times 10^{-3}; P_c = 1,130 \text{ k}$$

$$\frac{P_r}{P_c} = \frac{400 \text{ k}}{1,130 \text{ k}} = 0.3540 \therefore \text{Use H1-1a}$$

Evaluate the section:

$$0.887 \times 10^{-3}(400 \text{ k}) + 1.38 \times 10^{-3}(250 \text{ k} \cdot \text{ft}) + 2.85 \times 10^{-3}(80 \text{ k} \cdot \text{ft}) = 0.928 < 1.0 \text{ OK}$$

A W14x99 is adequate.

## Combined Forces Example 2

Determine if a W14x99 section of A992 steel is adequate to resist the following factored loads:  $P_r = 400\text{ k}$ ,  $M_{rx} = 250\text{ k-ft}$ ,  $M_{ry} = 80\text{ k-ft}$ . Use  $(KL)_x = 20\text{ ft}$ ,  $(KL)_y = 14\text{ ft}$ ,  $L_b = 26\text{ ft}$ ,  $C_b = 1.10$ .

Table 6-1, Page 6-70

**Table 6-1 (continued)**  
**Combined Flexure and Axial Force**  
 $F_y = 50\text{ ksi}$   
**W-Shapes**

W14x99

Shape	100'						99'					
	$p \times 10^3$		$\Delta_y \times 10^3$		$p \times 10^3$		$\Delta_y \times 10^3$		$p \times 10^3$		$\Delta_y \times 10^3$	
Design	(kips) <sup>2</sup>		(kip-ft) <sup>-1</sup>		(kips) <sup>-1</sup>		(kip-ft) <sup>-1</sup>		(kips) <sup>-1</sup>		(kip-ft) <sup>-1</sup>	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
10	3.64	0.894	1.86	1.23	1.15	0.794	2.07	1.38	1.26	0.839	2.33	1.55
11	1.14	0.761	1.86	1.23	1.26	0.838	2.07	1.38	1.38	0.920	2.33	1.55
12	1.16	0.774	1.86	1.23	1.26	0.853	2.07	1.38	1.41	0.957	2.33	1.55
13	1.19	0.789	1.86	1.23	1.31	0.869	2.07	1.38	1.44	0.955	2.33	1.55
14	1.21	0.805	1.87	1.23	1.33	0.887	2.08	1.38	1.47	0.975	2.33	1.55
15	1.24	0.823	1.89	1.26	1.36	0.907	2.10	1.40	1.50	0.997	2.33	1.55
16	1.27	0.843	1.91	1.27	1.40	0.929	2.13	1.42	1.53	1.02	2.35	1.57
17	1.30	0.864	1.93	1.29	1.43	0.953	2.15	1.43	1.57	1.05	2.38	1.59
18	1.32	0.887	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.63
19	1.37	0.913	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.41	1.63

$p = 0.887 \times 10^{-3}$

Shape	$p \times 10^3$	$\Delta_y \times 10^3$	$p \times 10^3$	$\Delta_y \times 10^3$	$p \times 10^3$	$\Delta_y \times 10^3$	$p \times 10^3$	$\Delta_y \times 10^3$	$p \times 10^3$	$\Delta_y \times 10^3$	$p \times 10^3$	$\Delta_y \times 10^3$
12	0.774	1.86	1.23	1.28	0.853	2.07	1.38	1.41	0.937	2.33	1.55	
13	0.789	1.86	1.23	1.31	0.869	2.07	1.38	1.44	0.955	2.33	1.55	
14	0.805	1.87	1.25	1.33	0.887	2.08	1.38	1.47	0.975	2.33	1.55	
15	0.823	1.89	1.26	1.36	0.907	2.10	1.40	1.50	0.997	2.33	1.55	
16	0.843	1.91	1.27	1.40	0.929	2.13	1.42	1.53	1.02	2.35	1.57	
17	0.864	1.93	1.29	1.43	0.953	2.15	1.43	1.57	1.05	2.38	1.59	

$b_y = 2.85 \times 10^{-3}$

Shape	$p \times 10^3$	$\Delta_y \times 10^3$	$p \times 10^3$	$\Delta_y \times 10^3$	$p \times 10^3$	$\Delta_y \times 10^3$
46	4.35	3.83	2.00	1.88	3.87	3.31
48	4.05	3.30	2.02	1.94	3.67	3.49
50	5.38	3.58	3.05	2.03	5.88	3.98

**Other Constants and Properties**

Shape	$b_x \times 10^3$	$b_y \times 10^3$	$r_x$	$r_y$	$r_{p0}$
12	3.84	2.56	4.29	2.85	3.84
13	1.04	0.894	1.15	0.907	1.26
14	1.28	0.855	1.41	0.940	1.55

Shape does not meet compact limit for flexure with  $F_y = 50\text{ ksi}$ .



Table 6-1, Page 6-70

**Table 6-1 (continued)  
Combined Flexure and Axial Force  
and Axial Force**  $F_y = 50$  ksi

**W-Shapes**

W14x99

Shape	109'				99'				90'			
	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$	$p \times 10^3$	$b_x \times 10^3$
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
19	1.31	0.913	1.98	1.31	1.51	1.01	2.21	1.47	1.66	1.11	2.45	1.63
20	1.41	0.970	2.00	1.33	1.56	1.04	2.23	1.49	1.71	1.14	2.48	1.65
22	1.51	1.01	2.04	1.36	1.66	1.11	2.29	1.52	1.83	1.22	2.55	1.70
24	1.61	1.07	2.09	1.39	1.78	1.19	2.35	1.56	1.96	1.31	2.62	1.74
26	1.74	1.16	2.14	1.43	1.92	1.28	2.41	1.60	2.12	1.41	2.70	1.80
28	1.89	1.26	2.20	1.46	2.09	1.39	2.48	1.65	2.30	1.53	2.78	1.85
30	2.06	1.37	2.25	1.50	2.28	1.52	2.55	1.69	2.52	1.68	2.87	1.91

**Other Constants and Properties**

$p \times 10^3$ (kip-ft <sup>-1</sup> )	3.84	2.56	4.29	2.85	4.90	3.28
$b_x \times 10^3$ (kip-ft <sup>-1</sup> )	1.04	0.694	1.15	0.764	1.26	0.839
$b_y \times 10^3$ (kip-ft <sup>-1</sup> )	1.28	0.855	1.41	0.940	1.54	1.020
$r_x/r_y$	1.67	1.66	1.65	1.66	1.66	1.66
$r_x$ in.	3.73	3.73	3.73	3.73	3.73	3.73

Shape does not meet compact limit for flexure with  $F_y = 50$  ksi.

$b_x = 1.60 \times 10^{-3}$

$r_x/r_y = 1.66$

## Combined Forces Example 2

From Table 6-1, Pages 6-70, try a W14x99;  $p = 0.887 \times 10^{-3}$ ;  $b_x = 1.60 \times 10^{-3}$ ;  $b_y = 2.85 \times 10^{-3}$

For  $C_b = 1.10$ ,  $b_x = 1.60 \times 10^{-3} / 1.10 = 1.455 \times 10^{-3} > 1.38 \times 10^{-3}$

$\therefore$  Use  $1.455 \times 10^{-3}$

$$\frac{r_x}{r_y} = 1.66 > \frac{(KL)_x}{(KL)_y} = \frac{20 \text{ ft}}{14 \text{ ft}} = 1.426 \therefore y \text{ axis buckling controls}$$

Determine which equation applies:

$$p = \frac{1}{\phi_c P_n} = \frac{1}{P_c} = 0.887 \times 10^{-3}; P_c = 1,130 \text{ k}$$

$$\frac{P_r}{P_c} = \frac{400 \text{ k}}{1,130 \text{ k}} = 0.3540 \therefore \text{Use H1-1a}$$

Evaluate the section:

$$0.887 \times 10^{-3}(400 \text{ k}) + 1.455 \times 10^{-3}(250 \text{ k} \cdot \text{ft}) + 2.85 \times 10^{-3}(80 \text{ k} \cdot \text{ft}) = 0.947 < 1.0 \text{ OK}$$

A W14x99 is adequate.

## Combined Forces Example 3

Determine if a W14x99 section of A992 steel is adequate to resist the following factored loads:  $P_r = 200\text{ k}$ ,  $M_{rx} = 250\text{ k-ft}$ ,  $M_{ry} = 80\text{ k-ft}$ . Use  $(KL)_x = 30\text{ ft}$ ,  $(KL)_y = 14\text{ ft}$ ,  $L_b = 26\text{ ft}$ ,  $C_b = 1.80$ .

Table 6-1, Page 6-70

Table 6-1 (continued)  
Combined Flexure and Axial Force  
W-Shapes  
 $F_y = 50\text{ ksi}$

W14

W14x99

Shape	LRFD						ASD					
	$p < 10^3$	$p < 10^3$	$p < 10^3$	$p < 10^3$	$p < 10^3$	$p < 10^3$	$p < 10^3$	$p < 10^3$	$p < 10^3$	$p < 10^3$	$p < 10^3$	
Design	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	(kip-ft) <sup>-1</sup>	
Effective length, $L_e$ (ft), with respect to least radius of gyration $r_y$ , or Unbraced Length, $L_b$ (ft), for $K_x$ axis-bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	
10	1.04	0.894	1.96	1.23	1.15	0.764	2.0	1.38	0.839	2.33	1.55	
11	1.14	0.761	1.86	1.23	1.26	0.836	2.0	1.38	0.839	2.33	1.55	
12	1.16	0.774	1.86	1.23	1.28	0.853	2.0	1.38	0.839	2.33	1.55	
13	1.19	0.789	1.86	1.23	1.31	0.869	2.0	1.38	0.839	2.33	1.55	
14	1.21	0.805	1.87	1.25	1.33	0.887	2.0	1.38	0.839	2.33	1.55	
15	1.24	0.823	1.89	1.26	1.36	0.907	2.1	1.38	0.839	2.33	1.55	
16	1.27	0.843	1.91	1.27	1.40	0.929	2.1	1.38	0.839	2.33	1.57	
17	1.30	0.864	1.93	1.29	1.43	0.952	2.1	1.38	0.839	2.33	1.57	
18	1.33	0.887	1.95	1.30	1.47	0.976	2.1	1.38	0.839	2.33	1.61	
19	1.37	0.913	1.98	1.31	1.51	1.001	2.1	1.38	0.839	2.33	1.63	
20	1.41	0.940	2.00	1.33	1.56	1.027	2.1	1.38	0.839	2.33	1.63	
22	1.51	1.00	2.04	1.36	1.68	1.094	2.2	1.38	0.839	2.33	1.71	
24	1.61	1.07	2.09	1.39	1.78	1.164	2.2	1.38	0.839	2.33	1.74	
26	1.74	1.16	2.14	1.43	1.92	1.238	2.2	1.38	0.839	2.33	1.83	
28	1.89	1.26	2.20	1.46	2.09	1.316	2.2	1.38	0.839	2.33	1.93	
30	2.05	1.37	2.25	1.50	2.28	1.398	2.2	1.38	0.839	2.33	1.91	
32	2.27	1.51	2.31	1.54	2.51	1.484	2.2	1.38	0.839	2.33	1.97	
34	2.50	1.67	2.37	1.58	2.78	1.574	2.2	1.38	0.839	2.33	2.03	
36	2.79	1.86	2.44	1.62	3.10	1.668	2.2	1.38	0.839	2.33	2.13	
38	3.11	2.07	2.51	1.67	3.45	1.766	2.2	1.38	0.839	2.33	2.18	
40	3.44	2.29	2.58	1.72	3.83	1.868	2.2	1.38	0.839	2.33	2.28	
42	3.80	2.53	2.66	1.77	4.22	1.974	2.2	1.38	0.839	2.33	2.34	
44	4.17	2.77	2.74	1.82	4.63	2.084	2.2	1.38	0.839	2.33	2.41	
46	4.55	3.03	2.82	1.88	5.06	2.198	2.2	1.38	0.839	2.33	2.49	
48	4.96	3.30	2.92	1.94	5.51	2.316	2.2	1.38	0.839	2.33	2.57	
50	5.38	3.58	3.05	2.03	5.98	2.438	2.2	1.38	0.839	2.33	2.65	

Other Constants and Properties

$r_y \times 10^3$ (kip-ft) <sup>-1</sup>	3.84	2.56	4.29	2.85	4.90	3.26
$r_x \times 10^3$ (kip-ft) <sup>-1</sup>	1.04	0.694	1.15	0.764	1.26	0.839
$r_y \times 10^3$ (kip-ft) <sup>-1</sup>	1.28	0.855	1.41	0.940	1.55	1.03
$r_x/r_y$	1.67	1.15	1.65	1.26	1.66	1.26
$r_x$ (in.)	3.75	2.55	3.75	3.1	3.70	2.65

Shape does not meet compact limit for flexure with  $F_y = 50\text{ ksi}$

Minimum  $b_x = 1.38 \times 10^{-3}$

## Combined Forces Example 3

From Table 6-1, Pages 6-70, try a W14x99;  $p = 0.887 \times 10^{-3}$ ;  $b_x = 1.60 \times 10^{-3}$ ;  $b_y = 2.85 \times 10^{-3}$

For  $C_b = 1.80$ ,  $b_x = 1.60 \times 10^{-3} / 1.80 = 0.889 \times 10^{-3} < 1.38 \times 10^{-3}$

$\therefore$  Use  $1.38 \times 10^{-3}$

$$\frac{r_x}{r_y} = 1.66 > \frac{(KL)_x}{(KL)_y} = \frac{30 \text{ ft}}{14 \text{ ft}} = 2.143 \therefore x \text{ axis buckling controls}$$

Re-enter table with  $(KL)_{y,equiv} = \frac{30 \text{ ft}}{1.66} = 18.07 \text{ ft} \approx 18 \text{ ft}$

$$p = 0.978 \times 10^{-3}$$

Determine which equation applies:

$$p = \frac{1}{\phi_c P_n} = \frac{1}{P_c} = 0.978 \times 10^{-3}; P_c = 1,020 \text{ k}$$

$$\frac{P_r}{P_c} = \frac{200 \text{ k}}{10,200 \text{ k}} = 0.1961 \therefore \text{Use H1-1b}$$

## Combined Forces Example 3

$$\frac{1}{2} p P_y + \frac{9}{8} (b_x M_{rx} + b_y M_{ry}) \leq 1.0$$

Evaluate the section:

$$\begin{aligned} & \left[ \frac{0.978 \times 10^{-3}}{2} \right] (200 \text{ k}) \\ & + \frac{9}{8} [1.38 \times 10^{-3} (250 \text{ k} \cdot \text{ft}) + 2.85 \times 10^{-3} (80 \text{ k} \cdot \text{ft})] \\ & = 0.742 < 1.0 \text{ OK} \end{aligned}$$

A W14x99 is adequate.

## Column Base Plates

### Column Base Plates

#### Reference –

Fisher, J. M. and Kloiber, L. A., 2006, AISC Design Guide No. 1, *Base Plate and Anchor Rod Design*, Second Edition, AISC, Chicago, IL.

See also pages 14-4 to 14-7 and Section J8, page 16.1-132  
SCM

## Axially Loaded Base Plates

Concentric compressive axial loads

Tensile axial loads

Design of column base plates with small moments

Design of column base plates with large moments

Design for shear

## Axially Loaded Base Plates

$$P_p = 0.85f'_c A_1 \sqrt{\frac{A_2}{A_1}} \leq 1.7f'_c A_1 \quad (\text{J8-1})$$

$$\phi_c = 0.65$$

$$\sqrt{\frac{A_2}{A_1}} \leq 2$$

Case I –  $A_2 = A_1$

$$m = \frac{N - 0.95d}{2}$$

Case II –  $A_2 \geq 4A_1$

$$n = \frac{B - 0.80b_f}{2}$$

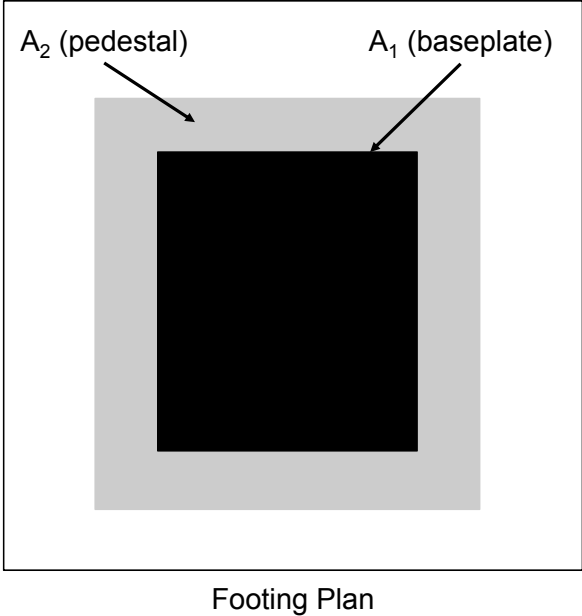
Case III –  $A_1 < A_2 < 4A_1$

$$X = \left[ \frac{4db_f}{(d+b_f)^2} \right] \frac{P_u}{\phi_c P_p}$$

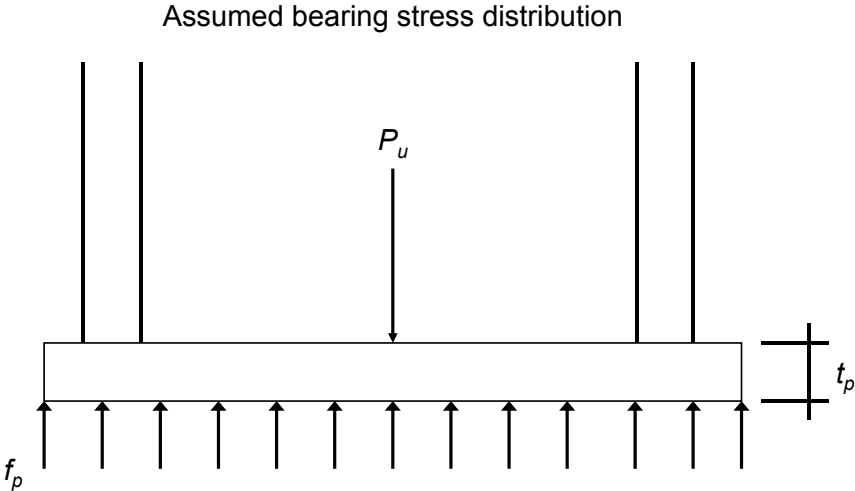
$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \leq 1$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4}$$

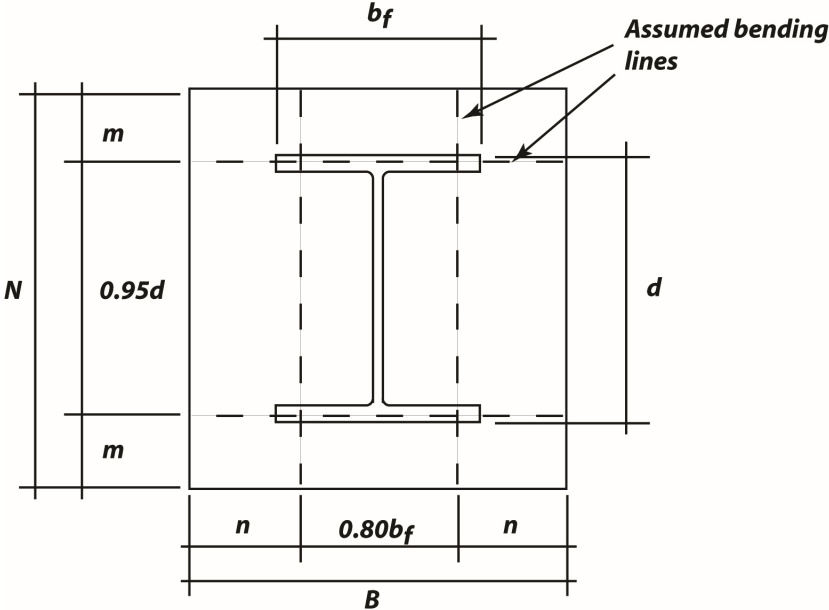
# Axially Loaded Base Plates



# Axially Loaded Base Plates

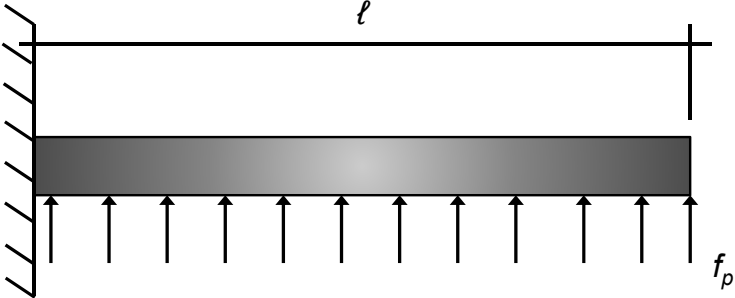


# Axially Loaded Base Plates



# Axially Loaded Base Plates

$$\ell = \max (m, n, \lambda n')$$



## Axially Loaded Base Plate Example

Design a base plate for a W12x106 ( $d = 12.9$  in;  $b_f = 12.2$  in.) column supporting a dead load of 300 k and a live load of 400 k. The plate bears on a 30 x 30 in. concrete pedestal. The specified concrete strength is 3 ksi and the column is made of A992 steel. The base plate is made of A36 steel.

## Axially Loaded Base Plate Example

$$P_u = 1.2(300 \text{ k}) + 1.6(400 \text{ k}) = 1,000 \text{ k}$$

Assume full concrete confinement (Case II –  $A_2 \geq 4A_1$ )

$$A_2 = (30 \text{ in})^2 = 900 \text{ in}^2$$

$$A_{1( reqd)} = \frac{P_u}{2\phi_c \times 0.85 \times f'_c} = \frac{1,000 \text{ k}}{2(0.65)(0.85)(3 \text{ ksi})} = 301.7 \text{ in}^2$$

Optimize base plate dimension

$$\Delta = 0.5[0.95d - 0.8b_f] = 0.5[0.95(12.9 \text{ in}) - 0.8(12.2 \text{ in})] = 1.248 \text{ in}$$

$$N \approx \sqrt{A_1} + \Delta = \sqrt{301.7 \text{ in}^2} + 1.248 \text{ in} = 18.62 \text{ in}$$

Try  $N = 19$  in

$$B = \frac{301.7 \text{ in}^2}{19 \text{ in}} = 15.9 \text{ in}$$

Try  $B = 16$  in



## Axially Loaded Base Plate Example

$$A_1 = (19 \text{ in})(16 \text{ in}) = 304 \text{ in}^2 > 301.7 \text{ in}^2$$

Calculate the area  $A_2$  geometrically similar to  $A_1$

Based on the 30 inch pedestal

$$N_2 = 30 \text{ in}$$

$$B/N = 16 \text{ in}/19 \text{ in} = 0.8421$$

$$B_2 = 0.8421(30 \text{ in}) = 25.26 \text{ in}$$

$$A_2 = (30 \text{ in})(25.26 \text{ in}) = 757.9 \text{ in}^2$$

$$757.9 \text{ in}^2 < 4(301.7 \text{ in}^2) = 1,207 \text{ in}^2 \therefore \text{Case III applies}$$

## Axially Loaded Base Plate Example

Use trial and error

Try  $N = 23 \text{ in}$ ;  $B = 19 \text{ in}$

$$A_1 = (23 \text{ in})(19 \text{ in}) = 437 \text{ in}^2$$

$$N_2 = 30 \text{ in}$$

$$B/N = 19 \text{ in}/23 \text{ in} = 0.8261$$

$$B_2 = 0.8261(30 \text{ in}) = 24.78 \text{ in}$$

$$A_2 = (30 \text{ in})(24.78 \text{ in}) = 743.4 \text{ in}^2$$

$$\phi_c P_p = \phi_c \times 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} = 0.65(0.85)(3 \text{ ksi})(437 \text{ in}^2) \sqrt{\frac{743.4 \text{ in}^2}{437 \text{ in}^2}}$$

$$= 944.8 \text{ k} < 1,000 \text{ k}$$

## Axially Loaded Base Plate Example

Try 24 in x 20 in

$$A_1 = (24 \text{ in})(20 \text{ in}) = 480 \text{ in}^2$$

$$N_2 = 30 \text{ in}$$

$$B/N = 20 \text{ in}/24 \text{ in} = 0.8333$$

$$B_2 = 0.8333(30 \text{ in}) = 25.0 \text{ in}$$

$$A_2 = (30 \text{ in})(25.0 \text{ in}) = 750.0 \text{ in}^2$$

$$\begin{aligned}\phi_c P_p &= \phi_c \times 0.85 A_1 \sqrt{\frac{A_2}{A_1}} = 0.65(0.85)(3 \text{ ksi})(480 \text{ in}^2) \sqrt{\frac{750.0 \text{ in}^2}{480 \text{ in}^2}} \\ &= 994.5 \text{ k} < 1,000 \text{ k}\end{aligned}$$

## Axially Loaded Base Plate Example

Try 25 in x 22 in

$$A_1 = (25 \text{ in})(22 \text{ in}) = 550 \text{ in}^2$$

$$N_2 = 30 \text{ in}$$

$$B/N = 22 \text{ in}/25 \text{ in} = 0.8800$$

$$B_2 = 0.8800(30 \text{ in}) = 26.4 \text{ in}$$

$$A_2 = (30 \text{ in})(26.4 \text{ in}) = 792.0 \text{ in}^2$$

$$\begin{aligned}\phi_c P_p &= \phi_c \times 0.85 A_1 \sqrt{\frac{A_2}{A_1}} = 0.65(0.85)(3 \text{ ksi})(550 \text{ in}^2) \sqrt{\frac{792.0 \text{ in}^2}{550 \text{ in}^2}} \\ &= 1,094 \text{ k} > 1,000 \text{ k}\end{aligned}$$

## Axially Loaded Base Plate Example

$$m = \frac{N - 0.95d}{2} = \frac{25 \text{ in} - 0.95(12.9 \text{ in})}{2} = 6.373$$

$$n = \frac{B - 0.80b_f}{2} = \frac{22 \text{ in} - 0.80(12.2 \text{ in})}{2} = 6.120$$

$$X = \left[ \frac{4db_f}{(d+b_f)^2} \right] \frac{P_u}{\phi_c P_p} = \left[ \frac{4(12.9 \text{ in})(12.2 \text{ in})}{(12.9 \text{ in} + 12.2 \text{ in})^2} \right] \frac{1,000 \text{ k}}{1,094 \text{ k}} = 0.9134$$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} = \frac{2\sqrt{0.9134}}{1 + \sqrt{1 - 0.9134}} = 1.477 \rightarrow 1$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4} = (1) \frac{\sqrt{(12.9 \text{ in})(12.2 \text{ in})}}{4} = 3.136$$

## Axially Loaded Base Plate Example

$$l = \max (6.373 \text{ in}, 6.120 \text{ in}, 3.136 \text{ in}) = 6.373 \text{ in}$$

$$t_p = m \sqrt{\frac{2P_u}{\phi F_y B N}} = (6.373 \text{ in}) \sqrt{\frac{2(1,000 \text{ k})}{0.9(36 \text{ ksi})(22 \text{ in})(25 \text{ in})}} = 2.135 \text{ in}$$

Use 25" x 2-1/4" x 22"

## Beam Bracing – Appendix 6

### Beam Bracing

#### ASCE 360 Appendix 6 – Stability Bracing for Beams and Columns

Applies to design of bracing that is not part of the lateral force resisting system – requirements for bracing that is part of the lateral force resisting system (that is, included in the analysis of the structure) are addressed in Chapter C.

## Beam Bracing

### Lateral bracing – columns and beams

- Relative
- Nodal

### Torsional bracing

- Nodal
- Continuous

### Reference –

- Yura, J. A., *Fundamentals of Beam Bracing*, AISC Engineering Journal, First Quarter 2001

## Beam Bracing

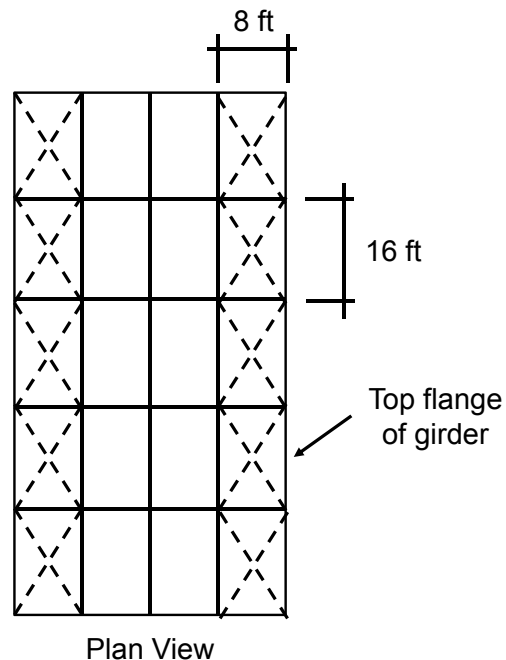
A relative brace controls the movement of the brace point with respect to adjacent brace points.

A discrete or nodal brace controls the movement at the brace point without interaction with adjacent points.

## Beam Bracing Example

Determine the bracing requirements for the beam system shown. The simply supported beams are A992 W21x62. Each bracing truss stabilizes 2-1/2 beams. The factored moment in the beams is 350 k-ft. Assume that the x-bracing (relative bracing) is to be designed as a tension only system so that in each panel only one diagonal is effective. Bracing consists of A36 rods.

## Beam Bracing Example



## Beam Bracing Example

$$M_r = 350 \text{ k} \cdot \text{ft}$$

$$L_b = 16 \text{ ft}$$

$$d = 21.0 \text{ in}$$

$$t_f = 0.615 \text{ in}$$

$$h_o = d - t_f = 21.0 \text{ in} - 0.615 \text{ in} = 20.39 \text{ in}$$

$$C_d = 1 \text{ (singular curvature)}$$

$$\Phi = 0.75$$

Required brace strength

$$P_{br} = \frac{0.008M_r C_d}{h_o} = \frac{0.008(350 \text{ k} \cdot \text{ft})(12 \text{ in/ft})(1)}{20.39 \text{ in}} = 1.648 \text{ k}$$

$$(2.5)(1.648 \text{ k}) = 4.120 \text{ k}$$

Both strength and stiffness must be considered!

Relative bracing  
Section A6.3.1a  
Eq. A-6-5

Perpendicular to  
longitudinal axis of  
beam

## Beam Bracing Example

Relative bracing  
Section A6.3.1a  
Eq. A-6-6

Perpendicular to  
longitudinal axis of beam

Required brace stiffness

$$\beta_{br} = \frac{1}{\Phi} \left( \frac{4M_r C_d}{L_b h_o} \right) = \frac{1}{0.75} \left[ \frac{4(350 \text{ k} \cdot \text{ft})(12 \text{ in/ft})(1)}{(16 \text{ ft} \times 12 \text{ in/ft})(20.39 \text{ in})} \right] = 5.722 \text{ k/in}$$

$$(2.5)(5.722 \text{ k/in}) = 14.31 \text{ k/in}$$

Brace area to satisfy strength requirement

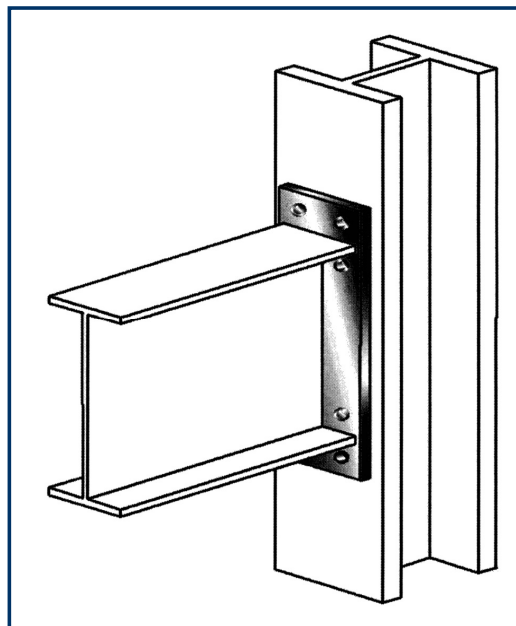
$$\phi A_b F_y = A_b (0.9)(36 \text{ ksi}) = (4.120 \text{ k})\sqrt{5}; A_b = 0.284 \text{ in}^2$$

Brace area to satisfy stiffness requirement

$$\cos^2 \theta \left( \frac{AE}{L} \right) = \left( \frac{1}{\sqrt{5}} \right)^2 \frac{A_b (29,000 \text{ ksi})}{8 \text{ ft} \times 12 \text{ in/ft} \times \sqrt{5}} = 14.31 \text{ k/in}; A_b = 0.5296 \text{ in}^2 \leftarrow$$

## Extended End Plate Connection

## End-Plate Moment Connections





## Moment End-Plate Connections

1. Carter, C. J., 2003, Steel Design Guide No. 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, AISC, Chicago, IL.
2. AISC, 2002, Steel Design Guide No. 16, *Flush and Extended Multiple-Row Moment End-Plate Connections*, AISC, Chicago, IL.
3. Murray, T. M. and Summer, E. A., 2003, Steel Design Guide No. 4, *Extended End-Plate Moment Connections, Seismic and Wind Applications*, Second Edition, AISC, Chicago, IL.

## Moment End-Plate Connections

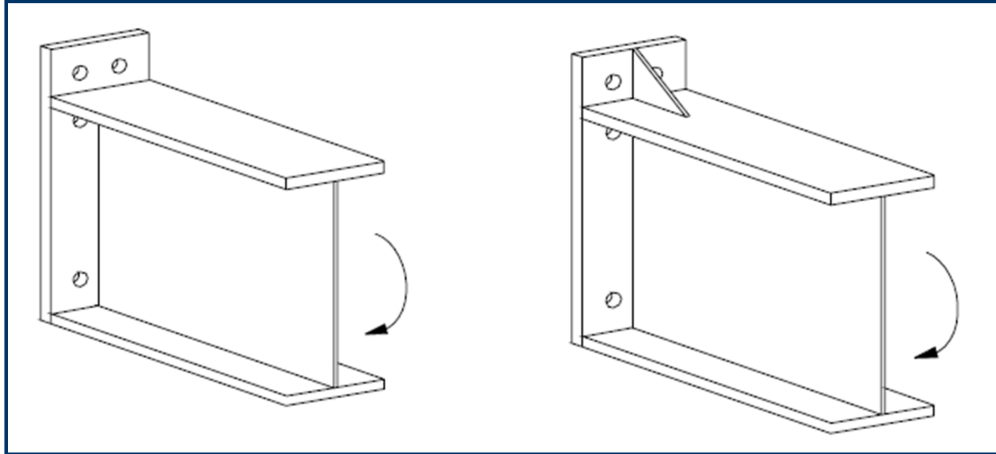
### Flush

- Two-bolt unstiffened
- Four-bolt unstiffened
- Four-bolt stiffened with stiffener between tension bolts
- Four-bolt stiffened with stiffener inside tension bolts

### Extended

- Four-bolt unstiffened
- Four-bolt stiffened
- Multiple-row 1/2 unstiffened
- Multiple-row 1/3 unstiffened
- Multiple-row 1/3 stiffened

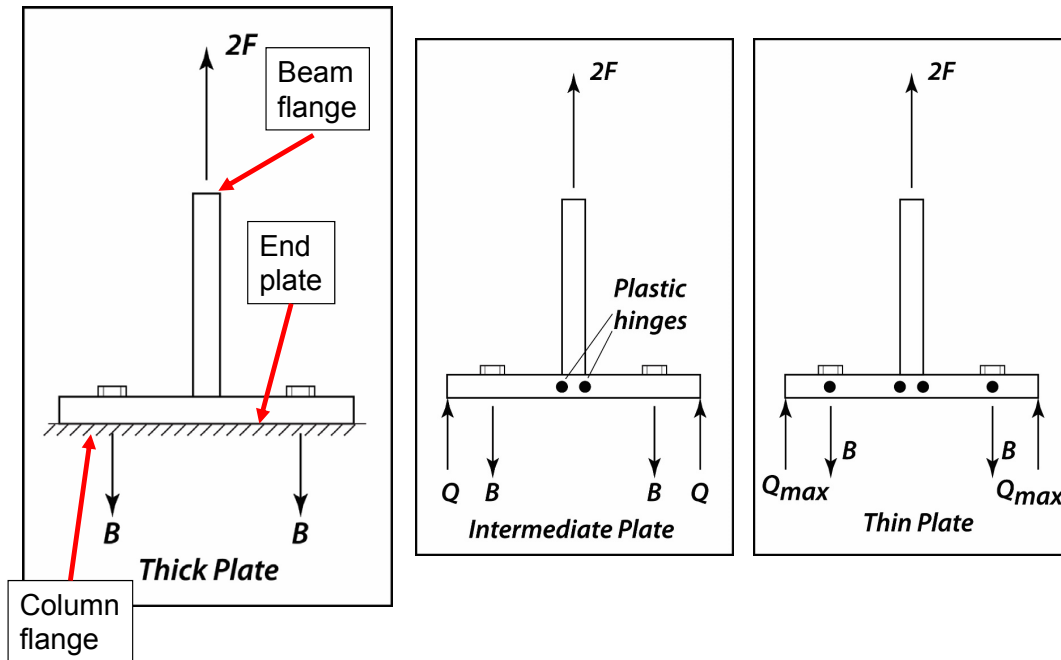
## Extended Moment End-Plate Connections



### Moment End-Plate Connection Limit States

1. Flexural yielding of the beam section
2. Flexural yielding of the endplate
3. Yielding of the column panel zone
4. Tension failure of the endplate bolts
5. Shear failure of the endplate bolts
6. Failure of welds

## Split-T Model



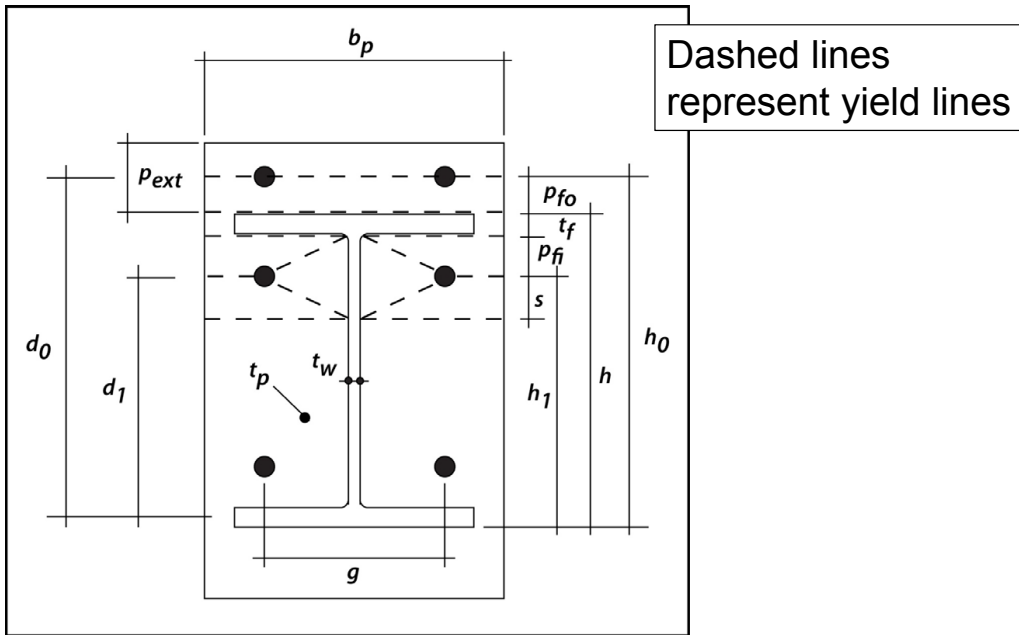
## Connection Yield Parameter – Y

### Four-bolt extended unstiffened

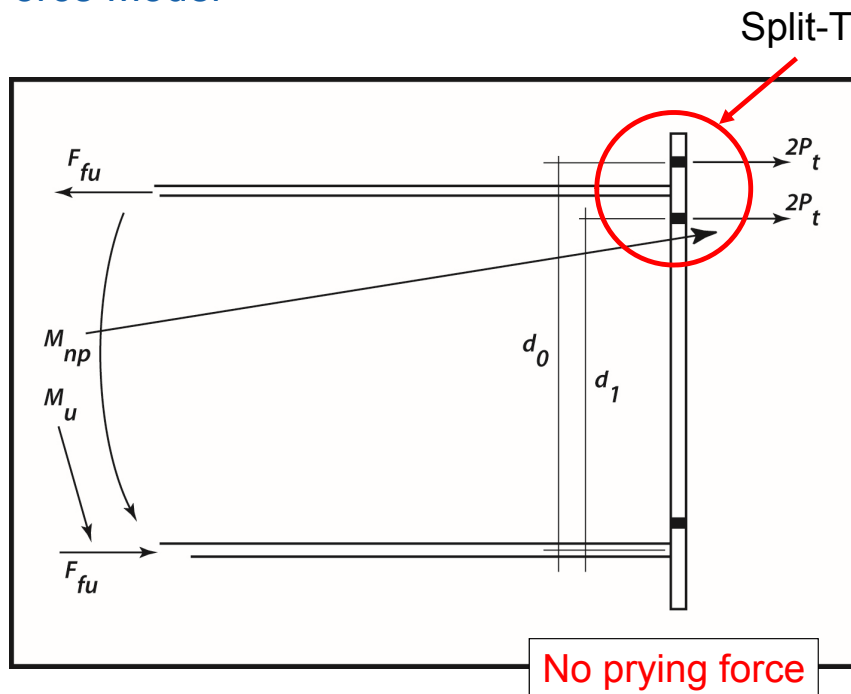
Geometry	Yield-Line Mechanism	Bolt Force Model
<p>End-Plate Yield</p>	$\phi M_n = \phi_b M_{pl} = \phi_b F_y A_{pl} Y$ $Y = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_{f,s}} + \frac{1}{s} \right) + h_0 \left( \frac{1}{p_{f,o}} - \frac{1}{2} \right) + \frac{2}{g} [h_1 (p_{f,s} + s)] \right]$ <p>Note: Use <math>p_{f,i} = s</math>, if <math>p_{f,i} &gt; s</math></p> $s = \frac{1}{2} \sqrt{b_p g}$ $\phi_b = 0.90$	

**Reference 2**

## Plate Geometry and Yield Pattern



## Bolt Force Model



## Extended Moment End-Plate Example

A W21x68 beam is to be connected to a W14x99 exterior column using a four-bolt unstiffened extended end plate connection. The moment that must be developed by the connection is 350 *k-ft* ( $M_r$ ) and the required shear resistance is 45 *k*. The required axial strength of the column is  $P_r = 600$  *k*. The connection will be used in a low-seismic application ( $R \leq 3$ ). The beam and column are made of A992 steel and the connection plate is made of A36 steel. ASTM A325-N snug-tight bolts are to be used and welds will be made with E70 electrodes.

## Section and Material Properties

### Beam data

$$A_b = 20.0 \text{ in}^2$$

$$d_b = 21.1 \text{ in}$$

$$t_{wb} = 0.430 \text{ in}$$

$$b_{fb} = 8.27 \text{ in}$$

$$t_{fb} = 0.685 \text{ in}$$

$$k_b = 1.19 \text{ in}$$

$$k_1 = \frac{7}{8} \text{ in}$$

$$\text{Workable gage} = 5\frac{1}{2} \text{ in}$$

$$Z_{xb} = 160 \text{ in}^3$$

### Column Data

$$A_c = 29.1 \text{ in}^2$$

$$d_c = 14.2 \text{ in}$$

$$t_{wc} = 0.485 \text{ in}$$

$$b_{fc} = 14.6 \text{ in}$$

$$t_{fc} = 0.780 \text{ in}$$

$$k_c = 1.38 \text{ in}$$

$$k_c = 1.38 \text{ in}$$

$$(h/t_w)_c = 23.5 \text{ in}$$

$$\text{Workable gage} = 5\frac{1}{2} \text{ in}$$

$$Z_{xc} = 173 \text{ in}^3$$

### Material Data

$$F_{y,b,c} = 50 \text{ ksi}$$

$$F_{u,b,c} = 65 \text{ ksi}$$

$$F_{y,pl} = 36 \text{ ksi}$$

$$F_{u,pl} = 58 \text{ ksi}$$

### Bolt data

$$F_t = 90 \text{ ksi}$$

## Beam Side

### Geometric Design Data

$\gamma_r = 1$  (extended end plate connection)

$$b_p = b_{fb} + 1 \text{ in} = 8.27 \text{ in} + 1 \text{ in} = 9.27 \text{ in}; \text{ Use } b_p = 9.25$$

$$g = 5.5 \text{ in}$$

$$p_{fi} = 2 \text{ in}$$

$$p_{fo} = 2 \text{ in}$$

$$d_e = 1 \frac{5}{8} \text{ in}$$

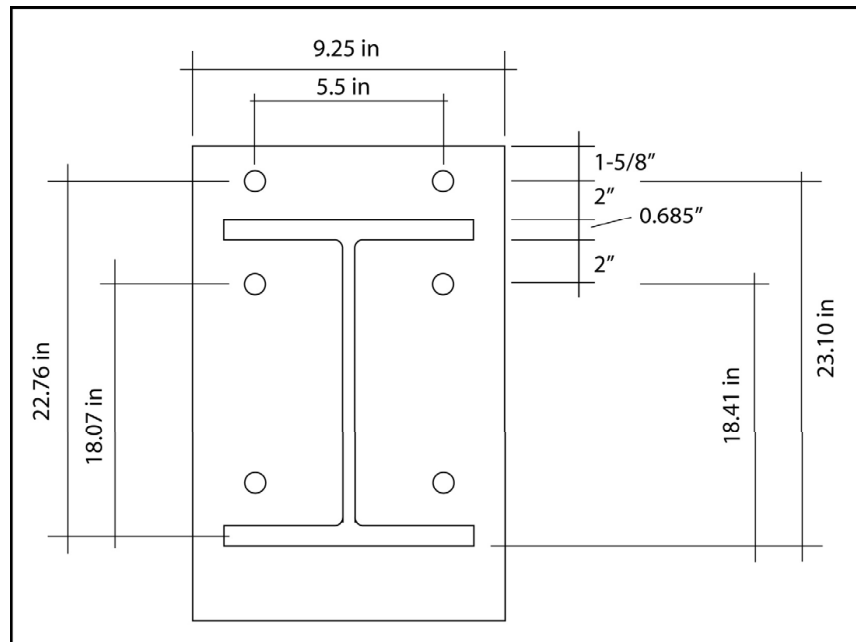
$$d_0 = d_b + p_{fo} - \frac{t_{fb}}{2} = 21.1 \text{ in} + 2 \text{ in} - \frac{0.685 \text{ in}}{2} = 22.76 \text{ in}$$

$$h_0 = d_0 + \frac{t_{fb}}{2} = 22.76 \text{ in} + \frac{0.685 \text{ in}}{2} = 23.10 \text{ in}$$

$$d_1 = d_b - t_{fb} - p_{fi} - \frac{t_{fb}}{2} = 21.1 \text{ in} - 0.685 \text{ in} - 2 \text{ in} - \frac{0.685 \text{ in}}{2} = 18.07 \text{ in}$$

$$h_1 = d_1 + \frac{t_{fb}}{2} = 18.07 \text{ in} + \frac{0.685 \text{ in}}{2} = 18.41 \text{ in}$$

## Beam/Plate Configuration



## Select Bolt Diameter

Note: the symbol  $d_b$  is used to represent both beam depth and bolt diameter. The meaning of the symbol applies should be clear from the context.

Required bolt diameter

$$M_{rc} = (350 \text{ k} \cdot \text{ft})(12 \text{ in/ft}) = 4,200 \text{ k} \cdot \text{in}$$

$$d_{b,reqd} = \sqrt{\frac{2M_{rc}}{\pi\phi F_t(d_0 + d_1)}} = \sqrt{\frac{2(4,200 \text{ k} \cdot \text{in})}{\pi(0.75)(90 \text{ ksi})(22.76 \text{ in} + 18.07 \text{ in})}}$$

$$= 0.9852 \text{ in}$$

Try  $d_b = 1 \text{ in}$

$$\text{Minimum } p_f = 1 + \frac{1}{2} = 1.5 \text{ in} < 2 \text{ in OK}$$

## Calculate No-Prying Moment

Thick plate – no prying force

$M_{np}$  → no prying force moment

Bolt tensile strength

$$P_t = F_t A_b = (90 \text{ ksi}) \left[ \frac{\pi(1 \text{ in})^2}{4} \right] = 70.65 \text{ k}$$

$$M_{np} = 2P_t(h_0 + h_1) = 2(70.65 \text{ k})(22.76 \text{ in} + 18.07 \text{ in}) \\ 5769 \text{ k} \cdot \text{in}$$

$$\phi M_{np} = 0.75(5,769 \text{ k} \cdot \text{in}) = 4,326 \text{ k} \cdot \text{in} > 4,200 \text{ k} \cdot \text{in} \text{ OK}$$

## Select End Plate Thickness

End plate yield line mechanism parameter

$$s = \frac{1}{2} \sqrt{b_p g} = \frac{1}{2} \sqrt{(9.25 \text{ in})(5.5 \text{ in})} = 3.57 \text{ in} > p_{fi} = 2.0$$

If  $p_{fi} > s$ , use  $p_{fi} = s$

∴ Use  $p_{fi} = 2.0$

$$Y_p = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left( \frac{1}{p_{fo}} \right) - \frac{1}{2} \right] + \frac{2}{g} [h_1(p_{fi} + s)] = \\ \frac{9.25 \text{ in}}{2} \left[ (18.41 \text{ in}) \left( \frac{1}{2 \text{ in}} + \frac{1}{3.57 \text{ in}} \right) + (23.10 \text{ in}) \left( \frac{1}{2 \text{ in}} \right) - \frac{1}{2} \right] + \\ \frac{2}{5.5 \text{ in}} [(18.41 \text{ in})(2 \text{ in} + 3.57 \text{ in})] = 154.8$$



## Select End Plate Thickness – Flexure

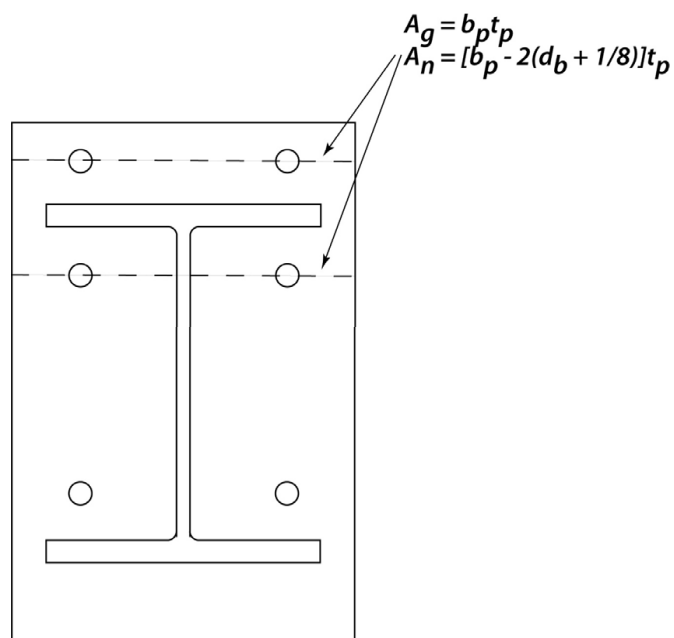
Required end plate thickness

1.11 insures  
thick plate behavior

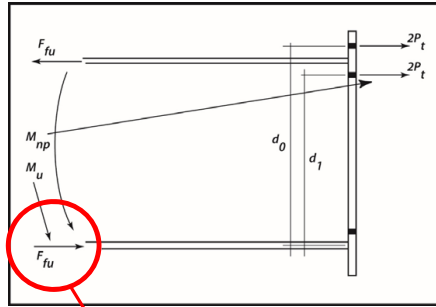
$$t_{p,reqd} = \sqrt{\frac{1.11\gamma_r(\phi M_{np})}{\phi_b F_{y,pl} Y_p}}$$
$$= \sqrt{\frac{1.11(1.0)(4,326 \text{ k}\cdot\text{in})}{0.9(36 \text{ ksi})(154.8)}} = 0.9785 \text{ in}$$

Use  $t_p = 1 \text{ in}$

## Shear Areas



## Check End Plate Thickness – Shear



Factored beam flange force

$$F_{fu} = \frac{M_{rc}}{d_b - t_{fb}} = \frac{4,200 \text{ k}\cdot\text{in}}{21.1 \text{ in} - 0.685 \text{ in}} = 205.7 \text{ k}$$

Shear yielding in the extended part of the end plate – ASCE 360 Eq. J4-3

$$\begin{aligned} \phi R_n &= 2[\phi(0.6F_{y,pl})b_p t_p] \\ &= 2[(1.00)(0.6)(36 \text{ ksi})(9.25 \text{ in})(1 \text{ in})] = 400 \text{ k} \end{aligned}$$

205.7 k < 400 k OK

## Select End Plate Thickness – Shear

Shear rupture in the two row of bolts at top of connection

$$\begin{aligned} A_n &= \left[ b_p - 2 \left( d_b + \frac{1}{8} \right) \right] t_p = [9.25 - 2(1 \text{ in} + 0.125 \text{ in})](1 \text{ in}) \\ &= 7.0 \text{ in}^2 \end{aligned}$$

$$\phi R_n = 0.75(0.6F_{u,pl})A_n = 0.75(0.6)(58 \text{ ksi})(7.0 \text{ in}^2) = 132.7 \text{ k}$$

2(132.7) > 205.7 k OK

## Compression Bolt Shear Rupture Capacity

$$V_u = 45 \text{ k}$$

ASCE 360 Section J3 and Table J3.2

$$\phi R_n = \phi n_b F_v A_b = 0.75(2)(54 \text{ ksi}) \left[ \frac{\pi(1 \text{ in})^2}{4} \right] = 63.6 \text{ k}$$

$$45 \text{ k} < 63.6 \text{ k OK}$$

## Bolt Bearing/Tearout Capacity in Endplate

$$R_n = 1.2l_c t_p F_{u,pl}$$

$$1.2l_c t_p F_{u,pl} \rightarrow \text{tearout strength}$$

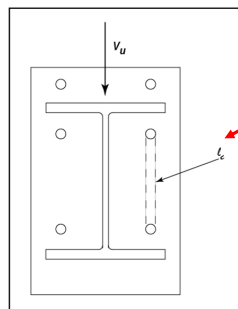
$$2.4d_b t_p F_{u,pl} \rightarrow \text{bearing strength}$$

Nominal bolt bearing strength – one bolt

$$2.4d t_p F_{u,pl} = 2.4(1 \text{ in})(1 \text{ in})(58 \text{ ksi})$$

$$= 139.2 \text{ k/bolt}$$

Section J3.10  
– ASCE 360



Tearout – one bottom bolt

$$l_c \approx d_b - 2p_{fi} = 21.1 \text{ in} - 2(2 \text{ in}) = 17.1 \text{ in}$$

$$R_n = 1.2l_c t_p F_{u,pl} = 1.2(17.1 \text{ in})(1 \text{ in})(58 \text{ ksi})$$

$$= 1,190 \text{ k}$$

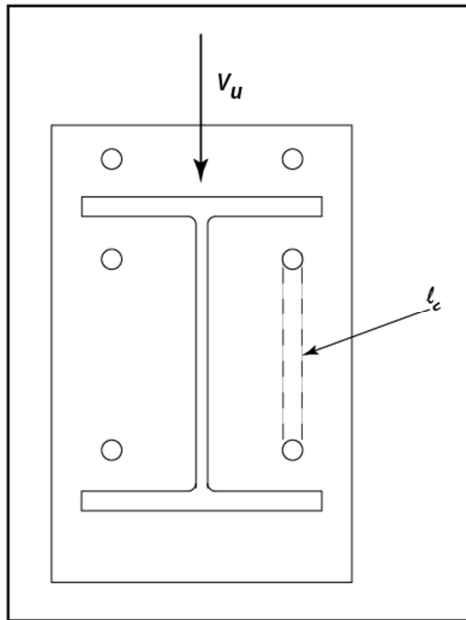
$$1,190 \text{ k} > 139.2 \text{ k} \therefore \text{bearing controls}$$

$$R_n = 139.2 \text{ k}$$

Capacity for two bolts

$$\phi R_n = 2(0.75)(139.2 \text{ k}) = 208.8 \text{ k} > 45 \text{ k OK}$$

## Bolt Bearing/Tearout End Plate



## Bolt Bearing/Tearout Column

Two bolts bearing on column flange

$$t_{fc} = 0.780 \text{ in}$$

$$\begin{aligned} 2.4d_b t_p F_{u,pl} &= 2.4(1 \text{ in})(0.780 \text{ in})(65 \text{ ksi}) \\ &= 121.7 \text{ k/bolt} \end{aligned}$$

$$\phi R_n = 2(0.75)(121.7 \text{ k}) = 182.6 \text{ k} > 45 \text{ k OK}$$

## Beam Flange to End Plate Weld Design

Minimum fillet weld size → 5/16 in

Weld design force should be equal to the calculated flange force, but not less than  $0.6F_y A_{fb}$

$$R_u = F_{uf} \geq 0.6F_y A_{fb}$$

$$F_{uf} = 205.7 \text{ k}$$

$$0.6F_y A_{fb} = 0.6(50 \text{ ksi})(8.27 \text{ in})(0.685 \text{ in}) = 170.0 \text{ k} < 205.7 \text{ k}$$

$$R_u = 205.7 \text{ k}$$

## Beam Flange to End Plate Weld Design

Beam flanges to end plate

$$L_w = b_{fb} + (b_{fb} + t_{wb}) = 8.27 \text{ in} + (8.27 \text{ in} - 0.430 \text{ in}) = 16.11 \text{ in}$$

$$D = \frac{205.7 \text{ k}}{1.5(1.392)(16.11 \text{ in})} = 6.115 \text{ sixteenths} \rightarrow 7$$

Use 7/16 in fillet weld

Eq. 8-2a  
Page 8-8 SCM

## Beam Web to End Plate Weld Design Flexure

Minimum weld size → 5/16 in

$$D = \frac{\phi F_{yb} t_{wb}}{2(1.5)(1.392)} = \frac{0.9(50 \text{ ksi})(0.430 \text{ in})}{2(1.5)(1.392)} = 4.633 \text{ sixteenths} \rightarrow 5$$

Use 5/16 in fillet weld

## Beam Web to End Plate Weld Design Flexure

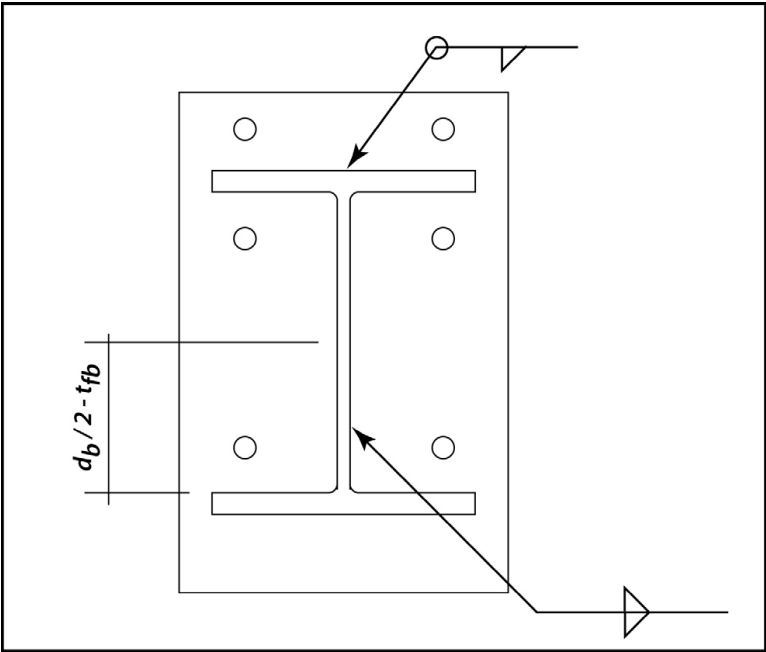
Minimum weld size → 5/16 in

$$L_w = 2 \left( \frac{d_b}{2} - t_{fb} \right) = 2 \left( \frac{21.1 \text{ in}}{2} - 0.685 \right) \text{ in} = 19.73 \text{ in}$$

$$D = \frac{V_u}{2(1.392)L_w} = \frac{45 \text{ k}}{(1.392)(19.73 \text{ in})} = 1.639 \text{ sixteenths}$$

Minimum weld size controls

# Beam to End Plate Welds



# Column Side

## Local Tensile Bending in Column Flange

$$R_n = \left( \frac{b_s}{\alpha_m p_e} \right) t_{fc}^2 F_y C_y \text{ (Equation 2.2-9 Reference 1)}$$

$$R_n = 6.25 t_f^2 F_y \text{ (Eq. J10-1 – ASCE 360)}$$

$$\phi = 0.9; C_t = 1$$

$$b_s = 2.5(2p_f + t_{fb}) = 2.5(2[2 \text{ in}] + 0.685 \text{ in}) = 11.71 \text{ in}$$

$$p_e = \frac{g}{2} - \frac{d_b}{4} - k_1 = \frac{5.5 \text{ in}}{2} - \frac{1 \text{ in}}{4} - \frac{7}{8} \text{ in} = 1.625 \text{ in}$$

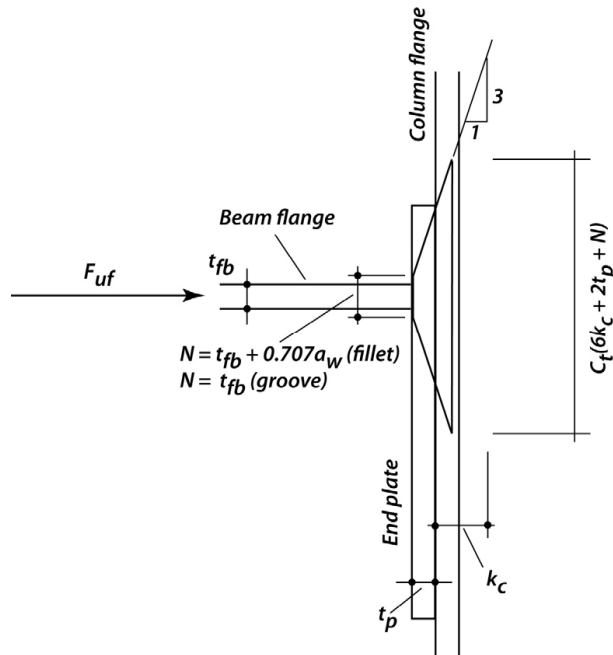
$$\alpha_m = 1.36 \left( \frac{p_e}{d_b} \right)^{1/4} = 1.36 \left( \frac{1.625 \text{ in}}{1 \text{ in}} \right)^{1/4} = 1.536$$

$$R_n = \left( \frac{11.71 \text{ in}}{1.536[1.625 \text{ in}]} \right) (0.780 \text{ in})^2 (50 \text{ ksi})(1) = 142.7 \text{ k}$$

$$\phi R_n = 0.9(142.7 \text{ k}) = 128.4 \text{ k} < 205.7 \text{ k} \therefore \text{Column flange needs stiffening}$$

$$205.7 \text{ k} - 128.4 \text{ k} = 77.3 \text{ k}$$

## Concentrated Force on Column Flange





## Column Local Web Yielding

Assume connection not at top of column ( $C_t = 1$ )

$$R_n = [C_t(6k_c + 2t_p + N)]F_{yc}t_{wc} \text{ (Equation 2.2-11 Reference 1)}$$

$$R_n = [(5k_c + l_b)]F_{yc}t_{wc} \text{ (Eq. J10-2 ASCE 360)}$$

$$\phi = 1; C_t = 1$$

$$N = t_{fb} + 0.707a_w = 0.685 \text{ in} + 0.707(0.7375 \text{ in}) = 0.9943 \text{ in}$$

$$\begin{aligned}\phi R_n &= (1)[(1)\{6(1.38 \text{ in}) + 2(1 \text{ in}) + 0.9943 \text{ in}\}](50 \text{ ksi})(0.485 \text{ in}) \\ &= 273.4 \text{ k} > F_{fu} = 205.7 \text{ k}\end{aligned}$$

Stiffeners not required

## Column Web Buckling

Because this is an exterior column, web buckling of the column does not need to be checked. If the connection was to an interior column (beams on each side of the column), the following check would be performed:

$$h = \left(\frac{h}{t_w}\right)_c t_{wc} = (23.5)(0.485 \text{ in}) = 11.40 \text{ in}$$

ASCE 360 Eq. J10-8

$$\begin{aligned}\phi R_n &= \frac{\phi 24 t_{wc}^3 \sqrt{E F_{yc}}}{h} = \frac{0.9(24)(0.485 \text{ in})^3 \sqrt{(29,000 \text{ ksi})(50 \text{ ksi})}}{11.40 \text{ in}} \\ &= 260.3 \text{ k} > F_{uf} = 205.7 \text{ k}\end{aligned}$$

Stiffeners not required

## Column Web Crippling

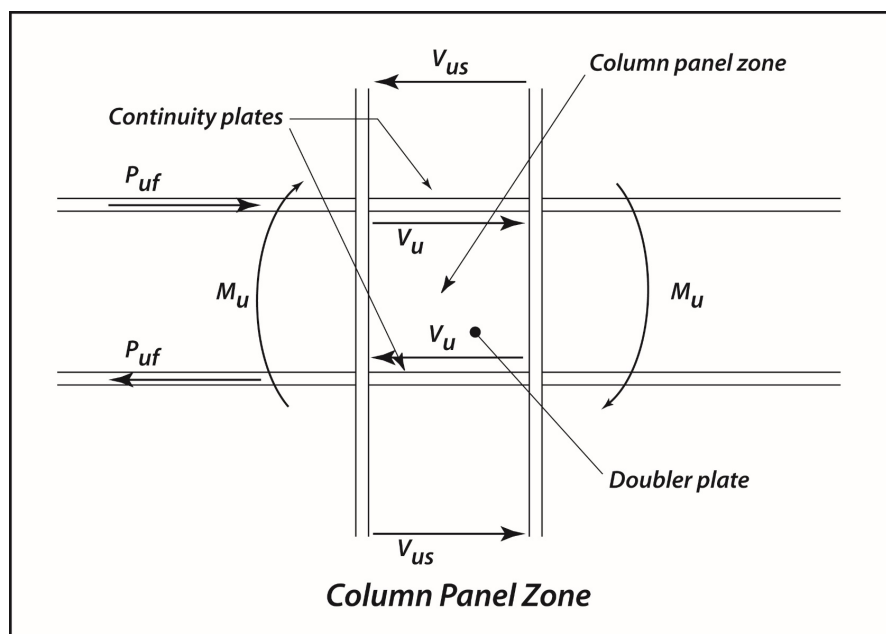
ASCE 360 Equation J10-4

Force applied at distance greater than  $d/2$  from end of member

$$\begin{aligned}\phi R_n &= \phi 0.80 t_{wc}^2 \left[ 1 + 3 \left( \frac{N}{d_c} \right) \left( \frac{t_{wc}}{t_{fc}} \right)^3 \right] \sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}} \\ &= 0.75(0.80)(0.485 \text{ in})^2 \times \\ &\quad \left[ 1 + 3 \left( \frac{0.9943 \text{ in}}{14.2 \text{ in}} \right) \left( \frac{0.485 \text{ in}}{0.780 \text{ in}} \right)^3 \right] \sqrt{\frac{(29,000 \text{ ksi})(50 \text{ ksi})(0.780 \text{ in})}{0.485 \text{ in}}} \\ &= 226.4 \text{ k} > F_{uf} = 205.7 \text{ k}\end{aligned}$$

Stiffeners not required

## Column Panel Zone



## Column Panel Zone

This check also applies to interior connection.

ASCE 360 – Section J10.6(a)

$$P_c = P_y = F_y A_c = (50 \text{ ksi})(29.1 \text{ in}^2) = 1,455 \text{ k}$$

$$V_r = V_u = P_{uf} - V_{us} = \frac{M_u}{d_c} = \frac{4,200 \text{ k}\cdot\text{in}}{14.7 \text{ in}} - 0 = 285.7 \text{ k (neglect story shear } V_{us})$$

$$0.4P_c = 582.0 \text{ k}; < P_r = 600 \text{ k} > 0.4P_c$$

$$\begin{aligned}\phi R_n &= \phi 0.6 F_y d_c t_{wc} \left( 1.4 - \frac{P_r}{P_c} \right) \\ &= 0.9(0.6)(50 \text{ ksi})(14.7 \text{ in})(0.645 \text{ in}) \left( 1.4 - \frac{600 \text{ k}}{1,455 \text{ k}} \right) \\ &= 252.8 \text{ k} < 285.7 \text{ k}\end{aligned}$$

Panel zone shear strength is not adequate – doubler plates are required

## Column Stiffening

Stiffening requirements for this column are minimal. It may be more economical to increase the column size to W14x109 rather than stiffen the W14x99.

## Column Stiffening

### Column Stiffening Example

In the previous example, change the column to a W14x90.  
Design stiffeners for this column.

## Section and Material Properties

### Column data

$$A_c = 26.5 \text{ in}^2$$

$$d_c = 14.0 \text{ in}$$

$$t_{wc} = 0.440 \text{ in}$$

$$b_{fc} = 14.5 \text{ in}$$

$$t_{fc} = 0.710 \text{ in}$$

$$k_c = 1.31 \text{ in}$$

$$(h/t_w)_c = 25.9 \text{ in}$$

$$\text{Workable gage} = 5 \frac{1}{2} \text{ in}$$

$$Z_{xc} = 157 \text{ in}^3$$

### Material Data

$$F_{y,b,c} = 50 \text{ ksi}$$

$$F_{u,b,c} = 65 \text{ ksi}$$

$$F_{y,pl} = 36 \text{ ksi}$$

$$F_{u,pl} = 58 \text{ ksi}$$

### Bolt data

$$F_t = 90 \text{ ksi}$$

## Local Tensile Bending in Column Flange

$$R_n = \left( \frac{b_s}{\alpha_m p_e} \right) t_{fc}^2 F_y C_t \text{ (Equation 2.2-9 Reference 1)}$$

$$\phi = 0.9$$

$$b_s = 2.5(2p_f + t_{fb}) = 2.5(2[2 \text{ in}] + 0.685 \text{ in}) = 11.71 \text{ in}$$

$$p_e = \frac{g}{2} - \frac{d_b}{4} - k_1 = \frac{5.5 \text{ in}}{2} - \frac{1 \text{ in}}{4} - \frac{7}{8} \text{ in} = 1.625$$

$$\alpha_m = 1.36 \left( \frac{p_e}{d_b} \right)^{1/4} = 1.36 \left( \frac{1.625 \text{ in}}{1 \text{ in}} \right)^{1/4} = 1.536$$

$$R_n = \left( \frac{11.71 \text{ in}}{1.536[1.625 \text{ in}]} \right) (0.710 \text{ in})^2 (50 \text{ ksi})(1) = 118.2 \text{ k}$$

$$\phi R_n = 0.9(118.2 \text{ k}) = 106.4 \text{ k} < 205.7 \therefore \text{Column flange needs stiffening}$$

$$205.7 \text{ k} - 106.4 \text{ k} = 99.3 \text{ k (tensile force)}$$

## Column Web Yielding

Unstiffened column capacity

Assume not at top of column ( $C_t = 1$ )

$$\phi R_n = \phi [C_t(6k_c + 2t_p + N)]F_{yc}t_{wc}$$

$$N = t_{bf} + 0.707a_w = 0.685 \text{ in} + 0.707(0.4375 \text{ in}) = 0.9943 \text{ in}$$

$$\begin{aligned}\phi R_n &= (1)[(1)\{6(1.31 \text{ in}) + 2(1 \text{ in}) + 0.9943 \text{ in}\}](50 \text{ ksi})(0.440 \text{ in}) \\ &= 238.8 \text{ k} > F_{fu} = 205.7 \text{ k}\end{aligned}$$

Stiffeners not required

## Column Web Compression Buckling

This check must be made only if beams frame into column on two sides.

Unstiffened column capacity

$$h = \left(\frac{h}{t_w}\right)_c t_{wc} = (25.9)(0.440 \text{ in}) = 11.40 \text{ in}$$

$$\begin{aligned}\phi R_n &= \frac{\phi 24 t_{wc}^3 \sqrt{E F_{yc}}}{h} = \frac{0.9(24)(0.440 \text{ in})^3 \sqrt{(29,000 \text{ ksi})(50 \text{ ksi})}}{11.40 \text{ in}} \\ &= 194.4 \text{ k} < F_{uf} = 205.7 \text{ k}\end{aligned}$$

Stiffeners are required

$$207.7 \text{ k} - 194.4 \text{ k} = 11.3 \text{ k (compressive force)}$$

## Column Web Crippling

Unstiffened column capacity

$$\begin{aligned}\phi R_n &= \phi 0.80 t_{wc}^2 \left[ 1 + 3 \left( \frac{N}{d_c} \right) \left( \frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}} \\ &= 0.75(0.80)(0.440 \text{ in})^2 \times \\ &\quad \left[ 1 + 3 \left( \frac{0.9943 \text{ in}}{14.0 \text{ in}} \right) \left( \frac{0.440 \text{ in}}{0.710 \text{ in}} \right)^{1.5} \right] \sqrt{\frac{(29,000 \text{ ksi})(50 \text{ ksi})(0.710 \text{ in})}{0.440 \text{ in}}} \\ &= 196.2 \text{ k} < F_{uf} = 205.7 \text{ k}\end{aligned}$$

Stiffeners are required

$$205.7 \text{ k} - 196.2 \text{ k} = 9.5 \text{ k (compressive force)}$$

## Column Panel Zone

AISC 360 – Section J10.6

$$P_c = P_y = F_y A_c = (50 \text{ ksi})(26.5 \text{ in}^2) = 1,325 \text{ k}$$

$$V_r = P_{uf} - V_{us} = \frac{M_u}{d_c} = \frac{4,200 \text{ k}\cdot\text{in}}{14.0 \text{ in}} - 0 = 300.0 \text{ k (neglect story shear)}$$

$$0.4P_c = 530.0 \text{ k} < P_r = 600 \text{ k}$$

$$\begin{aligned}\phi R_n &= \phi 0.6 F_y d_c t_{wc} \left( 1.4 - \frac{P_r}{P_c} \right) \\ &= 0.9(0.6)(50 \text{ ksi})(14.0 \text{ in})(0.440 \text{ in}) \left( 1.4 - \frac{600 \text{ k}}{1,325 \text{ k}} \right) \\ &= 157.0 \text{ k} < 285.7 \text{ k}\end{aligned}$$

Panel zone shear strength is not adequate

$$V_{u,dp} = 300.0 \text{ k} - 157.0 \text{ k} = 143.0 \text{ k}$$

## Stiffener Design Requirements

Stiffeners must be designed to resist the difference between the required strength and the available strength.

In this example 99.3 k tension and 11.3 k compression. Panel zone must be reinforced additional force of 143 k.

Design requirements for tension are found in J4.1 and J10.8.

Design requirements for tension are found in J4.4 and J10.8.

## Stiffener Design Requirements

Stiffeners in tension must be welded to the loaded flange and the web.

Stiffeners in compression must either bear on or be welded to the loaded flange; they must also be welded to the web.



## Stiffener Design Requirements

Prescriptive requirements:

$$b_s + \frac{t_{wc}}{2} \geq \frac{b_{fc}}{3}$$

$$t_s \geq \frac{t_{wc}}{2} \geq \frac{b_s}{16}$$

$$l_s \geq \frac{d_c}{2} \text{ except as required by section J10.5 and J10.7}$$

$b_s$  → stiffer width

$t_s$  → stiffer thickness

$l_s$  → stiffer length

## Stiffener Design – Tension

$$R_{u,dp} = 99.3 \text{ k}$$

Tensile yield

$$A_{st,min} = \frac{R_{u,st}}{\phi F_{y,st}} = \frac{99.3 \text{ k}}{0.9(36 \text{ ksi})} = 3.06 \text{ in}^2$$

Tensile rupture

$$A_{st,min} = \frac{R_{u,st}}{\phi F_{y,st}} = \frac{99.3 \text{ k}}{0.75(58 \text{ ksi})} = 2.28 \text{ in}^2$$

Try two plates  $3\frac{1}{4}'' \times \frac{1}{2}'' \times 7''$  – A36 – partial depth

$$A_{st} = 2(3.25 \text{ in})(0.5 \text{ in}) = 3.25 \text{ in}^2 > 3.06 \text{ in}^2$$

## Stiffener Design – Compression

$$R_{u,dp} = 11.3 \text{ k}$$

$$\frac{KL}{r} = \frac{(1)(7)}{0.5/\sqrt{12}} = 48.5 > 25 \therefore \text{Use Chapter E}$$

$$\frac{b}{t} = \frac{3.25}{0.5} = 7.5$$

$$\lambda_r = 0.45 \sqrt{\frac{E}{F_y}} = 0.45 \sqrt{\frac{29,500}{36}} = 12.9 > 7.5 \therefore \text{Not slender}$$

$$4.71 \sqrt{\frac{E}{F_y}} = 4.71 \sqrt{\frac{29,500}{36}} = 135.0 > 48.5$$

## Stiffener Design – Compression

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,500)}{(48.5)^2} = 123.7 \text{ ksi}$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\frac{36}{123.7}}\right] (36 \text{ ksi}) = 31.9 \text{ ksi}$$

$$P_n = F_{cr} A_g = (31.9 \text{ ksi})(0.5)(3.25) = 51.83 \text{ k}$$

$$\phi P_n = 0.9(51.83 \text{ k}) = 46.64 \text{ k} > 11.3 \text{ k} \therefore \text{OK}$$

## Stiffener Design

$$b_s + \frac{t_{wc}}{2} \geq \frac{b_{fc}}{3}; 3.25 + \frac{0.440}{2} = 3.47 \text{ in} < \frac{14.5}{3} = 4.833 \text{ in}$$

No good; Use  $b_s = 4.75 \text{ in}$

$$t_s \geq \frac{t_{wc}}{2} \geq \frac{b_s}{16}; 0.5 \text{ in} > \frac{0.440}{2} = 0.220 \text{ in}; 0.5 \text{ in} > \frac{4.75}{16} = 0.297 \text{ in}$$

$$l_s \geq \frac{d_c}{2}; 7 \text{ in} = \frac{14.0}{2}$$

Try two plates  $4\frac{3}{4}'' \times \frac{1}{2}''$  – A36 – full depth of column by Section J10.5

Recheck design using new dimension

## Double Plate Design

Double plate design requirements for shear are covered in Section J10.9 and Chapter G

Required Capacity = 143.0 k

$$t_p \geq \frac{V_{u,dp}}{0.9 \times 0.6 F_{y,dp} d_c} = \frac{143.0 \text{ k}}{0.9(0.6)(36 \text{ ksi})(14 \text{ in})}$$
$$= 0.5254 \text{ in}$$

Use 5/8" plate

Questions

Thank you!