

AISC
Night School

Thank you for joining our live webinar today.
We will begin shortly. Please standby.

Thank you.

Need Help?
Call ReadyTalk Support: 800.843.9166

AISC Live Webinars

Today's audio will be broadcast through the internet.

**Alternatively, to hear the audio through the phone,
dial 888-504-7949.**

Passcode: 956659



AISC Live Webinars

Today's live webinar will begin shortly.

Please standby.

As a reminder, all lines have been muted. Please type any questions or comments through the Chat feature on the left portion of your screen.

Today's audio will be broadcast through the internet.
Alternatively, to hear the audio through the phone, dial
888-504-7949.
Passcode: 956659



AISC Live Webinars

AISC is a Registered Provider with The American Institute of Architects Continuing Education Systems (AIA/CES). Credit(s) earned on completion of this program will be reported to AIA/CES for AIA members. Certificates of Completion for both AIA members and non-AIA members are available upon request.

This program is registered with AIA/CES for continuing professional education. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the AIA of any material of construction or any method or manner of handling, using, distributing, or dealing in any material or product.

Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation



AISC Live Webinars

Copyright Materials

This presentation is protected by US and International Copyright laws. Reproduction, distribution, display and use of the presentation without written permission of AISC is prohibited.

© The American Institute of Steel Construction 2017



Course Description

Session 2: October 10, 2017 -Fundamental Concepts, Part II

This live webinar discusses eccentric bolted and welded connections, direct loaded tension connections, block shear, the Whitmore Section, and light bracing connections. Beam bearing and column base plate design are also discussed. Design examples are presented to demonstrate concepts.



Learning Objectives

At the end of this program, participants will be able to:

- List the steps in designing an eccentric bolted and welded connection.
- List the limit states in designing a light bracing connection.
- Describe the Whitmore Section concept.
- List the limit states in designing a beam bearing plate connection.



There's always a solution in steel.

Fundamentals of Connection Design Session 2: Fundamental Concepts Part 2 October 10, 2017



Presented by
Thomas M. Murray, Ph.D., P.E.
Emeritus Professor
Virginia Tech, Blacksburg, Virginia



SCHEDULE

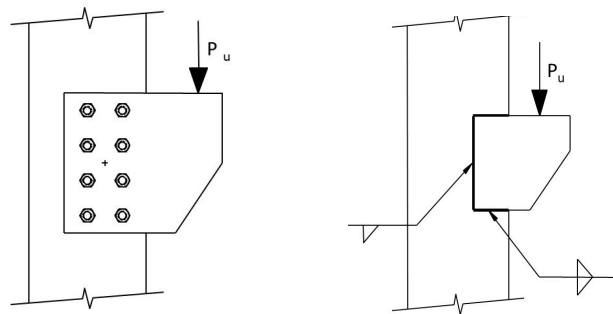
- October 03, 2017 Fundamental Concepts Part I
- October 10, 2017 **Fundamental Concepts Part II**
- October 17, 2017 **Shear Connections Part I**
- October 24, 2017 **Shear Connections Part II**
- November 07, 2017 **Moment Connections Part I**
- November 14, 2017 **Moment Connections Part II**
- November 28, 2017 **Introduction to Seismic Connections**
- December 05, 2017 **Bracing Connections and More**

FUNDAMENTAL CONCEPTS PART II

TOPICS



- Eccentric Bolted and Welded Connections
- Direct Loaded Tension Connections
- Light Bracing Connection Example
- Beam Bearing Plate Design
- Column Base Plate Design

ECCENTRIC BOLTED AND WELDED CONNECTIONS





Bolts: Eccentric Connections

Elastic Method



13

Bolts: Eccentric Connections

Instantaneous Center of Rotation Method



14

Bolts: Eccentric Connections

Table 7-7
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕP_n or R_n/Ω , is determined with

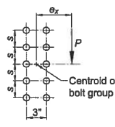
$$R_n = C \times r_n$$

or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi r_n}$	$C_{min} = \frac{\Omega P_u}{r_n}$

where

- P = required force, P_u or P_a , kips
- r_n = nominal strength per bolt, kips
- e_x = horizontal distance from the centroid of the bolt group to the line of action of P , in.
- s = bolt spacing, in.
- C = coefficient tabulated below



s, in.	e _x , in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0	
3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5	
4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7	
5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8	
6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8	
7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8	
8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8	
9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8	
10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9	
12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2	
14	0.19	0.57	1.08	1.78	2.62	3.66	4.82	6.15	7.61	9.19	10.9	12.7	
16	0.17	0.51	0.95	1.57	2.32	3.24	4.27	5.47	6.79	8.23	9.78	11.4	
18	0.15	0.45	0.85	1.41	2.07	2.90	3.83	4.92	6.11	7.43	8.85	10.4	
20	0.14	0.41	0.77	1.27	1.88	2.63	3.48	4.47	5.55	6.76	8.07	9.48	
24	0.12	0.34	0.65	1.07	1.58	2.21	2.93	3.77	4.69	5.72	6.85	8.06	
28	0.10	0.29	0.56	0.92	1.36	1.90	2.53	3.25	4.05	4.95	5.93	7.00	
32	0.09	0.26	0.49	0.80	1.19	1.67	2.22	2.86	3.57	4.36	5.23	6.18	
36	0.08	0.23	0.43	0.72	1.06	1.49	1.98	2.55	3.16	3.90	4.67	5.52	
C, in.	2.94	8.33	15.6	26.0	38.7	54.2	72.2	93.1	117	143	172	204	

$$\phi P_n = C \times \phi r_v$$

$$\phi = 0.75$$



Example: Eccentric Bolted Connection

Determine ϕP_n

$$\phi P_n = C \text{ (from Table 7-7)} \times \phi r_v$$

From *Manual* Table 7-7, Angle = 0°

with $e = 8$ in. and $n=4$

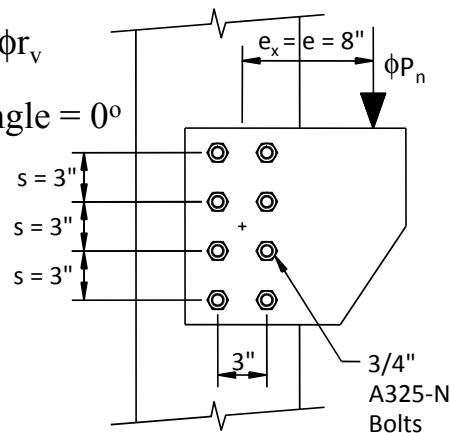
$$C = 2.93$$

From *Manual* Table 7-1,

for $\frac{3}{4}$ in. A325-N:

$$\phi r_v = \phi F_{nv} A_b = 17.9 \text{ k/bolt}$$

$$\phi P_n = 2.93 \times 17.9 = \underline{52.4 \text{ k}}$$



Example: Eccentric Bolted Connection

Table 7-7
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕR_n or R_n/Ω , is determined with
 $R_n = C \times r_n$
 or

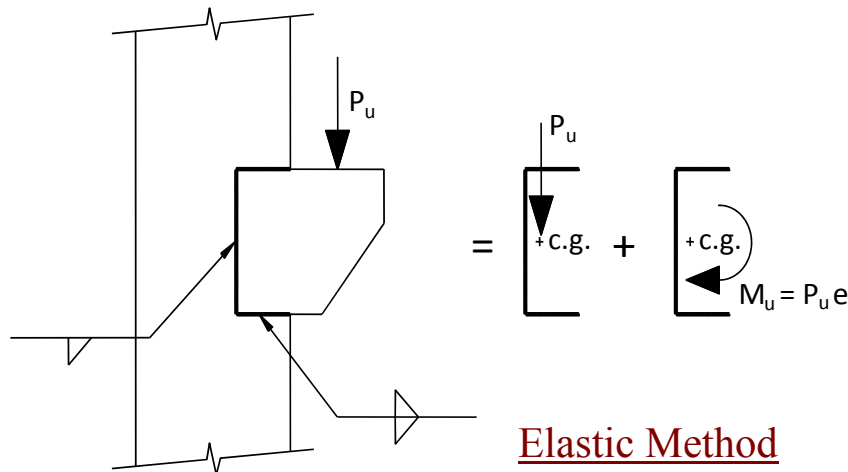
LRFD	ASD
$C_{MB} = \frac{\phi P_u}{\phi r_n}$	$C_{MB} = \frac{\Omega P_u}{r_n}$

where
 P_u = required force, P_u or P_u , kips
 r_n = nominal strength per bolt, kips
 e_x = horizontal distance from the centroid of the bolt group to the line of action of P_u , in.
 s = bolt spacing, in.
 C = coefficient tabulated below

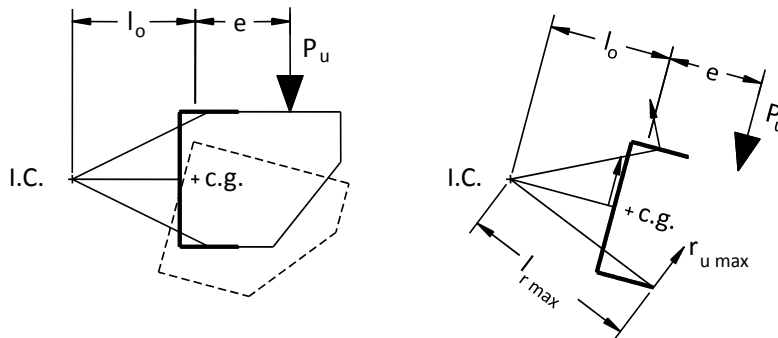
s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0	
3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5	
4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7	
5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8	
6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8	
7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8	
8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8	
9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8	
10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9	
12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2	
14	0.19	0.57	1.08	1.78	2.62	3.66	4.82	6.15	7.61	9.19	10.9	12.7	
16	0.17	0.51	0.95	1.57	2.32	3.24	4.27	5.47	6.79	8.23	9.78	11.4	
18	0.15	0.45	0.85	1.41	2.07	2.90	3.83	4.92	6.11	7.43	8.85	10.4	
20	0.14	0.41	0.77	1.27	1.88	2.63	3.48	4.47	5.55	6.76	8.07	9.48	
24	0.12	0.34	0.65	1.07	1.58	2.21	2.93	3.77	4.69	5.72	6.85	8.06	
28	0.10	0.29	0.56	0.92	1.36	1.90	2.53	3.25	4.05	4.95	5.93	7.00	
32	0.09	0.26	0.49	0.80	1.19	1.67	2.22	2.86	3.57	4.36	5.23	6.18	
36	0.08	0.23	0.43	0.72	1.06	1.49	1.98	2.55	3.18	3.90	4.67	5.52	
C, in.	2.94	8.33	15.8	26.0	38.7	54.2	72.2	93.1	117	143	172	204	

$n = 4$
 $e_x = 8$ in.
 $C = 2.93$

Welds: Eccentric Connections



Welds: Eccentric Connections



Instantaneous Center of Rotation Method

Welds: Eccentric Connections

Eccentric Weld Strength

$$R_{nx} = \sum F_{nwx} A_{wei}$$

$$R_{ny} = \sum F_{nwy} A_{wei}$$

$$M_n = \sum [F_{nwy} A_{wei} (x_i) - F_{nwx} A_{wei} (y_i)]$$

where

A_{wei} = effective area of weld throat of the i th weld element, in.² (mm²)

$F_{nwi} = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta_i) f(p_i)$

$f(p_i) = [p_i (1.9 - 0.9 p_i)]^{0.3}$

F_{nwi} = nominal stress in the i th weld element, ksi (MPa)

F_{nwx} = x -component of nominal stress, F_{nwi} , ksi (MPa)

F_{nwy} = y -component of nominal stress, F_{nwi} , ksi (MPa)

$p_i = \Delta_i / \Delta_{mi}$, ratio of element i deformation to its deformation at maximum stress

r_{cr} = distance from instantaneous center of rotation to weld element with minimum Δ_{ui} / r_i ratio, in. (mm)

r_i = distance from instantaneous center of rotation to i th weld element, in. (mm)

Welds: Eccentric Connections

Table 8-8
Coefficients, C,
for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω , is determined with
 $R_n = CC_1DL$ ($\phi = 0.75, \Omega = 2.00$)

LRFD				ASD			
$C_{min} = \frac{P_e}{\phi C_1 D l}$		$D_{min} = \frac{P_e}{\phi C C_1 l}$		$C_{min} = \frac{\Omega P_e}{C_1 D l}$		$D_{min} = \frac{\Omega P_e}{C C_1 l}$	
$I_{min} = \frac{P_e}{\phi C C_1 D}$		$I_{min} = \frac{\Omega P_e}{C C_1 D}$		$I_{min} = \frac{\Omega P_e}{C C_1 D}$		$I_{min} = \frac{\Omega P_e}{C C_1 D}$	

where
 P = required force, P_e or P_u , kips
 D = number of sixteenths-of-an-inch in the fillet weld size
 l = characteristic length of weld group, in.
 $a = h/2$
 e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
 C = coefficient tabulated below
 C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

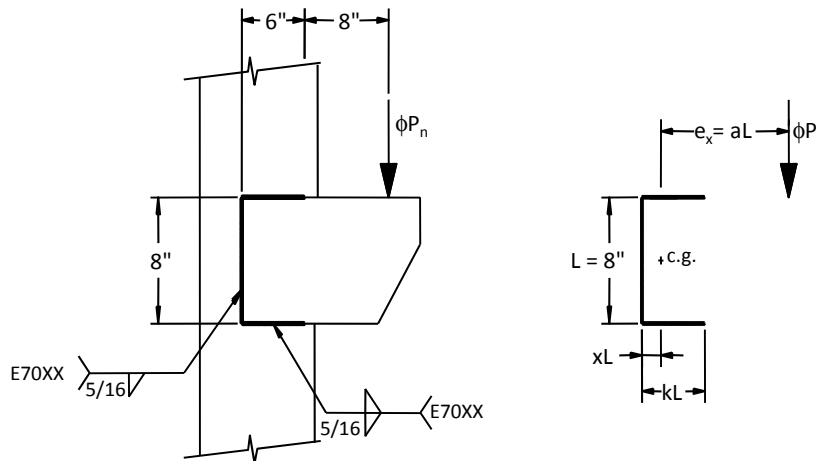
Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC Specification Sections J2.4, J4.1, and J4.2.

a	k																	
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0		
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	8.26	9.37	10.5	11.6	12.7		
0.10	1.86	2.28	2.78	3.30	3.83	4.37	4.92	5.46	6.01	6.56	7.11	8.22	9.32	10.4	11.5	12.7		
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.41	6.94	8.02	9.11	10.2	11.3	12.4		
0.20	1.76	2.18	2.63	3.11	3.60	4.11	4.61	5.13	5.64	6.16	6.68	7.72	8.77	9.83	10.9	12.0		
0.25	1.66	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37	5.86	6.36	7.37	8.39	9.42	10.5	11.5		
2.6	0.253	0.320	0.396	0.481	0.576	0.680	0.788	0.901	1.02	1.15	1.28	1.57	1.90	2.25	2.64	3.05		
2.8	0.235	0.297	0.368	0.447	0.535	0.632	0.734	0.839	0.950	1.07	1.19	1.47	1.77	2.10	2.46	2.85		
3.0	0.219	0.278	0.343	0.417	0.500	0.591	0.686	0.784	0.889	1.00	1.12	1.37	1.66	1.97	2.31	2.68		
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800		

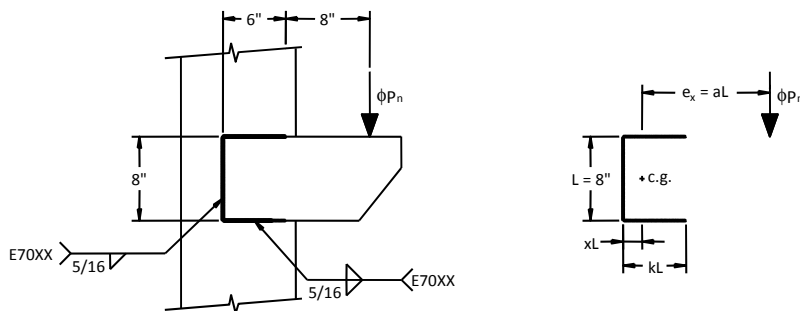
$R_n = CC_1DL$
 $\phi = 0.75$
 Parameters:
 $C_1 = E_{xx}/70$
 $k \Rightarrow x$ (at bottom of Table)
 $x \ \& \ a \Rightarrow C$



Example: Determine ϕP_n



Example: Determine ϕP_n



$$kL = 6 \text{ in.} \Rightarrow k = 6 / 8 = 0.75$$

Using *Manual* Table 8-8: $x = 0.225$

$$xL = 0.225 \times 8 = 1.8 \text{ in. (location of c.g.)}$$

Example: Determine ϕP_n

Manual Table 8-8

a	k															
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	8.26	9.37	10.5	11.6	12.7
0.10	1.86	2.28	2.78	3.30	3.83	4.37	4.92	5.46	6.01	6.56	7.11	8.22	9.32	10.4	11.5	12.7
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.41	6.94	8.02	9.11	10.2	11.3	12.4
0.20	1.76	2.18	2.63	3.11	3.60	4.11	4.61	5.13	5.64	6.16	6.68	7.72	8.77	9.83	10.9	12.0
0.25	1.66	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37	5.86	6.36	7.37	8.39	9.42	10.5	11.5
2.6	0.253	0.320	0.396	0.481	0.576	0.680	0.788	0.901	1.02	1.15	1.28	1.57	1.90	2.25	2.64	3.05
2.8	0.235	0.297	0.368	0.447	0.535	0.632	0.734	0.839	0.950	1.07	1.19	1.47	1.77	2.10	2.46	2.85
3.0	0.219	0.278	0.343	0.417	0.500	0.591	0.686	0.784	0.889	1.00	1.12	1.37	1.66	1.97	2.31	2.68
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Example: Determine ϕP_n

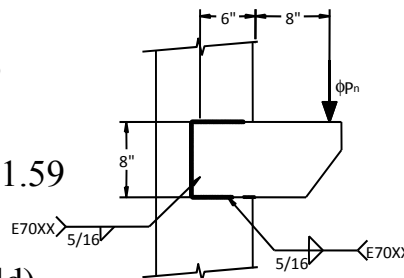
With $e_x = aL$

$$a = (6.0 + 8.0 - 1.8) / 8.0 = 1.53$$

Using *Manual* Table 8-8: $C = 1.59$

$D = 5$ (since 5/16" weld)

$C_1 = 1.0$ (since E70XX weld)



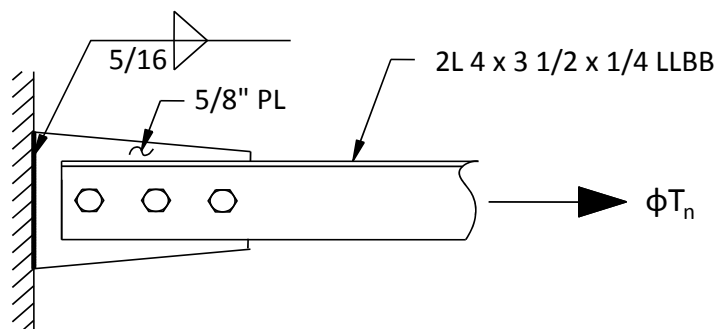
$$\begin{aligned} \phi P_n &= \phi C C_1 D L \\ &= 0.75 \times 1.59 \times 1.0 \times 5 \times 8.0 \\ &= \mathbf{47.7 \text{ k}} \end{aligned}$$

Example: Determine ϕP_n

Manual Table 8-8

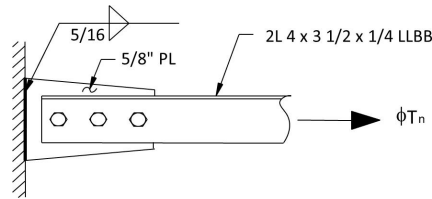
a	k															
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
1.4	0.464	0.589	0.729	0.883	1.05	1.23	1.42	1.61	1.82	2.04	2.27	2.77	3.31	3.89	4.50	5.15
1.6	0.408	0.517	0.640	0.775	0.924	1.09	1.25	1.43	1.61	1.81	2.02	2.46	2.95	3.48	4.04	4.64
1.8	0.363	0.461	0.570	0.691	0.825	0.970	1.12	1.28	1.45	1.62	1.81	2.22	2.66	3.14	3.66	4.21
2.0	0.328	0.415	0.514	0.623	0.744	0.877	1.01	1.16	1.31	1.47	1.64	2.01	2.42	2.86	3.34	3.85
2.2	0.298	0.378	0.468	0.567	0.678	0.800	0.926	1.06	1.20	1.35	1.50	1.84	2.22	2.62	3.07	3.54
2.4	0.274	0.347	0.429	0.521	0.623	0.735	0.852	0.973	1.10	1.24	1.38	1.70	2.04	2.42	2.84	3.28
2.6	0.253	0.320	0.396	0.481	0.576	0.680	0.788	0.901	1.02	1.15	1.28	1.57	1.90	2.25	2.64	3.05
2.8	0.235	0.297	0.368	0.447	0.535	0.632	0.734	0.839	0.950	1.07	1.19	1.47	1.77	2.10	2.46	2.85
3.0	0.219	0.278	0.343	0.417	0.500	0.591	0.686	0.784	0.889	1.00	1.12	1.37	1.66	1.97	2.31	2.68
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

DIRECT LOADED TENSION CONNECTIONS



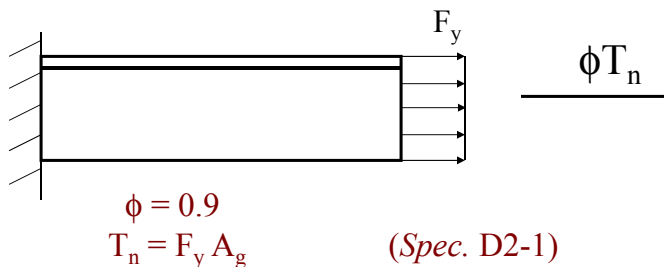
Applicable Limit States

- Tension Yielding
- Tension Rupture
- Block Shear
- Bearing and Tear Out
- Bolt Rupture
- Weld Rupture
- Whitmore Section Considerations
- For Design: $T_u \leq \phi T_n$



Limit State: Tension Rupture

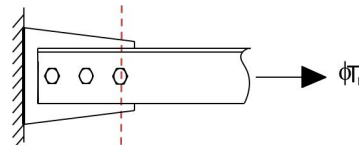
Specification D2 Tension Yielding



Note: Tension yielding is actually a member limit state.

Limit State: Tension Rupture

Specification D2 Tension Rupture



$\phi = 0.75$
 $\phi T_n = 0.75 F_u A_e$ (Spec. D2-2)

F_u = tensile strength
 = 58 ksi for A36; 65 ksi for A992 & Gr50

A_e = effective net area = $U A_n$

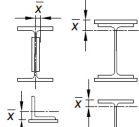
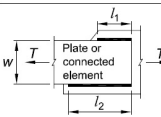
U = reduction or shear lag coefficient

A_n = net area

Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

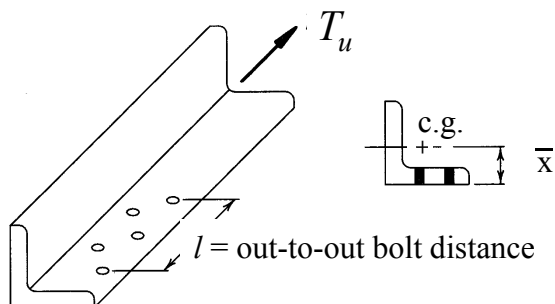
Case	Description of Element	Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).	$U = 1.0$	-
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)	$U = 1 - \frac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and $A_n = \text{area of the directly connected elements}$	-
4 ^{a)}	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of \bar{x} .	$U = \frac{3l^2}{3l^2 + w^2} \left(1 - \frac{\bar{x}}{l} \right)$	

Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

Ex. *Specification* Table D3.1, Case 2

$$U = 1 - \bar{x} / l$$

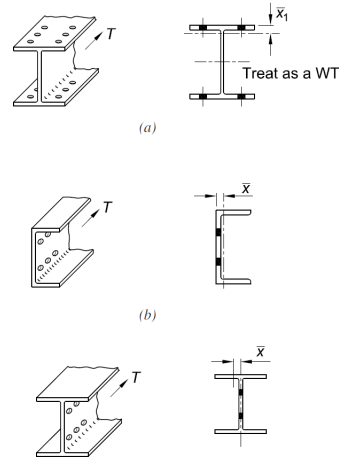


Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

$$U = 1 - \bar{x}/l$$

l = out-out bolt distance



Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

Ex. *Specification* Table D3.1 Case 4

$U = \frac{3l^2}{3l^2 + w^2} \left(1 - \frac{\bar{x}}{l} \right)$	
--	--

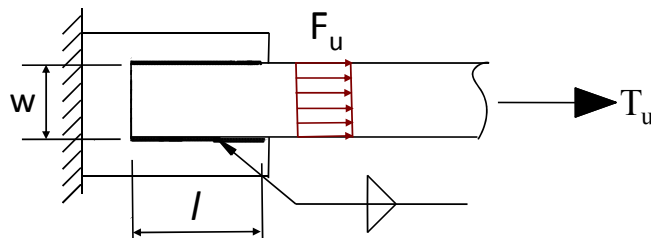
where $l = \frac{l_1 + l_2}{2}$

Note: l_1 and l_2 not less than 4 times fillet weld size.

Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

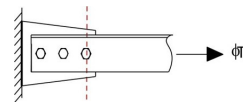
Ex. Common Connection



$$U = \frac{3l^2}{3l^2 + w^2}$$

Limit State: Tension Rupture

Net Area



$$A_n = \text{Net Area} = A_g - \Sigma A_h + \Sigma \text{Stagger}$$

A_g = gross area of cross-section

A_h = effective area of hole

$$= (\text{hole diameter} + 1/16 \text{ in.}) t_p$$

$$\text{Stagger} = (s^2/4g)t_p \text{ (Specification B4.3b)}$$

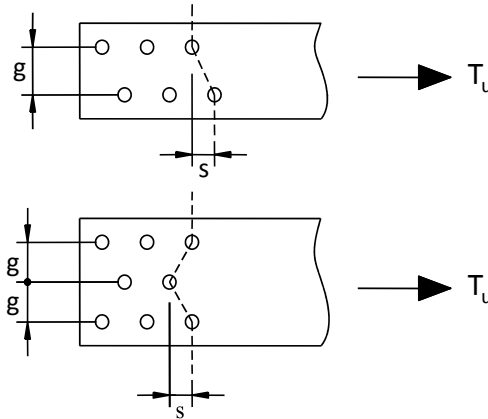
~~Note: $A_n \leq 0.85 A_g$ for Tension Splice Plates~~

~~(Rule does not apply to members.)~~

Limit State: Tension Rupture

Net Area

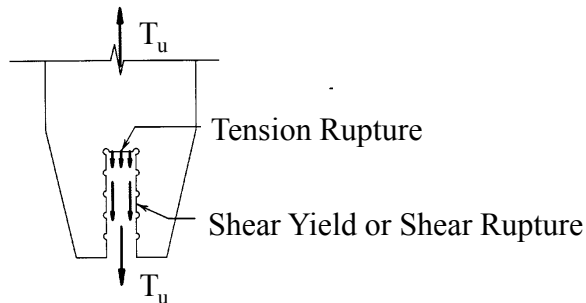
$$\text{Stagger Term} = (s^2/4g)t_p$$



Limit State: Block Shear Strength

Specification J4.3

Block Shear Strength



- Failure occurs when shear forces reach the smaller of shear yield and shear rupture.
- The tension area is at rupture when failure occurs.

Limit State: Block Shear



Limit State: Block Shear Strength

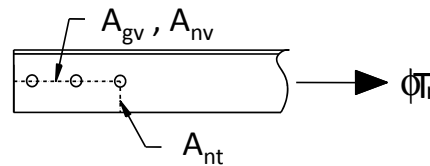
Specification J4.3 Block Shear Strength

$$\phi = 0.75$$

$$R_n = [0.6F_u A_{nv} + U_{bs} F_u A_{nt}] \leq [0.6F_y A_{gv} + U_{bs} F_u A_{nt}] \quad (J4-5)$$

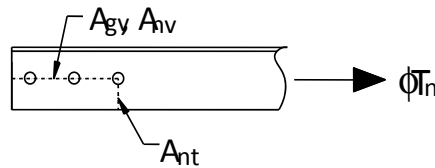
Or,

$$R_n = \min \left\{ \begin{array}{l} \text{Shear Rupture} \\ \text{Shear Yield} \end{array} \right\} + U_{bs} \text{ Tension Rupture}$$



Limit State: Block Shear Strength

Specification J4.3 Block Shear Strength



$$R_n = \min \left\{ \begin{array}{l} \text{Shear Rupture} \\ \text{Shear Yield} \end{array} \right. + U_{bs} \text{ Tension Rupture}$$

- Shear Rupture = $0.6F_u A_{nv}$ (net shear area)
- Shear Yield = $0.6F_y A_{gv}$ (gross shear area)
- Tension Rupture = $F_u A_{nt}$ (net tension area)
- $U_{bs} = 1.0$ for Direct Loaded Connections

Limit State: Block Shear Strength

Example: $A_{nv} = 2.53 \text{ in}^2$ $A_{gv} = 3.625 \text{ in}^2$
 $A_{nt} = 0.781 \text{ in}^2$ $U_{bs} = 1.0$
 A36 Steel: $F_y = 36 \text{ ksi}$ $F_u = 58 \text{ ksi}$

$$R_n = \min \left\{ \begin{array}{l} 0.6 \times 58 \times 2.53 = 88.0 \\ 0.6 \times 36 \times 3.625 = \underline{78.3} \end{array} \right. + 1.0 \times 58.0 \times 0.781$$

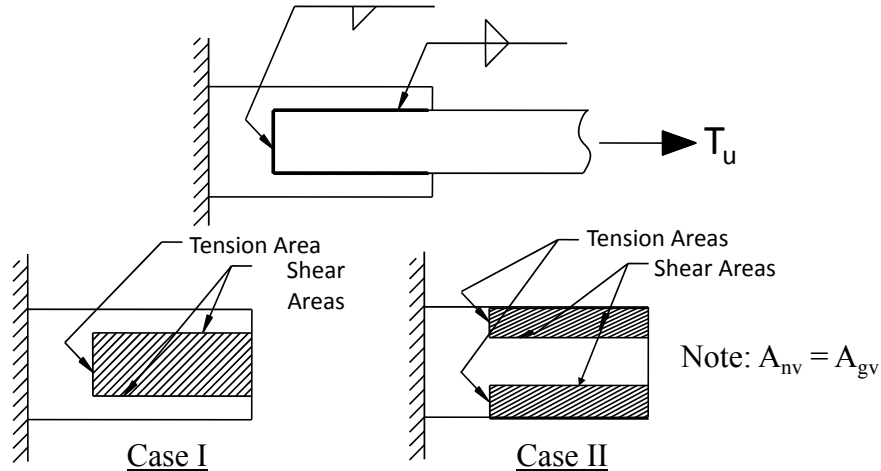
$$= 78.3 + 45.3 = 123.6 \text{ k}$$

$$\phi R_n = 0.75 \times 123.6 = \underline{92.7 \text{ k}}$$

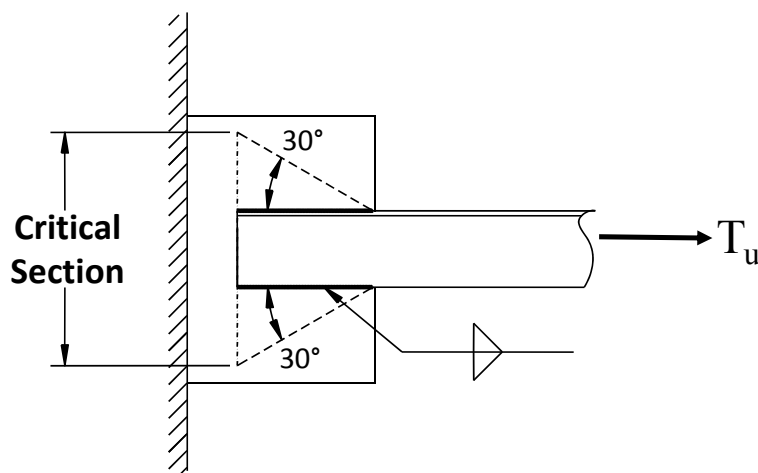
(Note: See following example for area calculations)

Limit State: Block Shear Strength

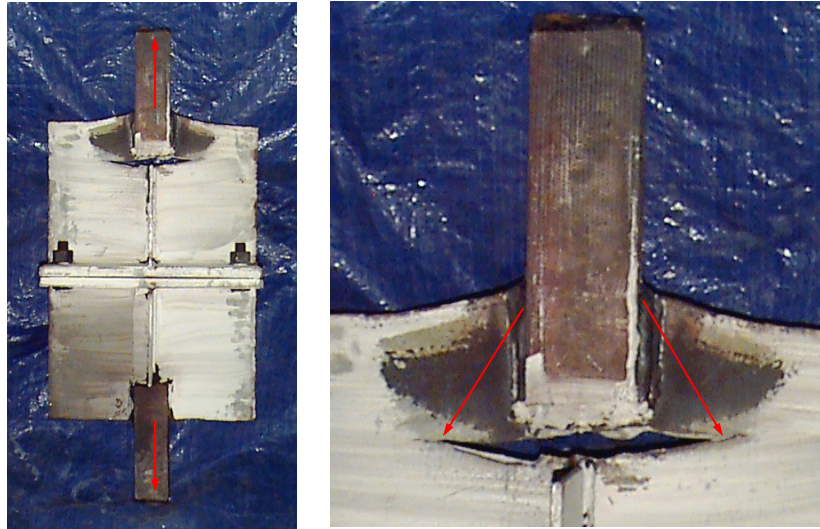
Welded Connections



Whitmore Section -- Welds



Whitmore Section -- Welds

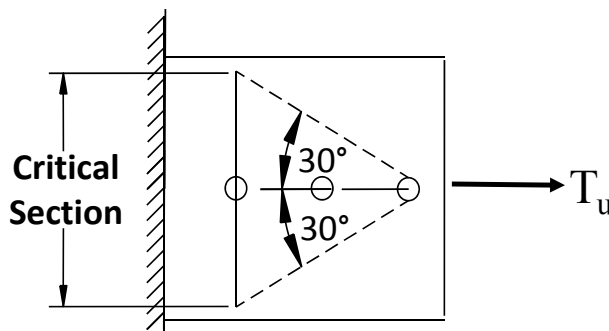


VirginiaTech
Invent the Future



45

Whitmore Section -- Bolts



VirginiaTech
Invent the Future

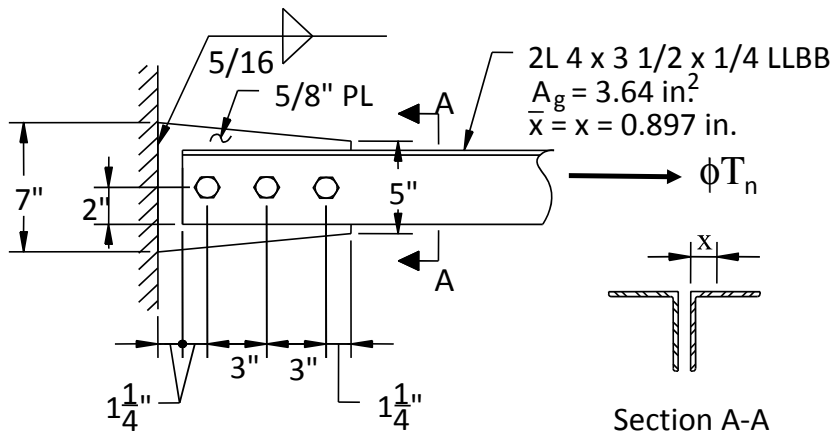


46

Light Bracing Connection

Example. Determine ϕT_n .

A36 Steel 3/4 in. A325-N Bolts



Light Bracing Connection

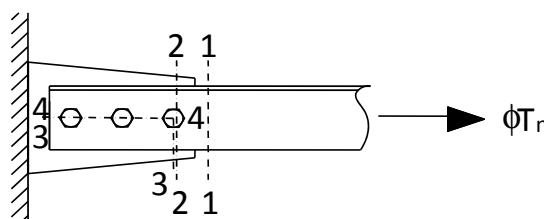
Limit States:

Angles:

- 1-1 Tension Yield
- 2-2 Tension Rupture
- 3-3 Block Shear

Shear Transfer Between Elements:

- 4-4 Angle Bearing and Tear Out
- 4-4 Bolt Shear Rupture
- 4-4 Plate Bearing and Tear Out



Light Bracing Connection

Limit States:

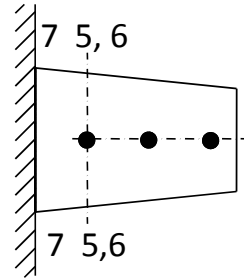
Plate:

5-5 Tension Yield

6-6 Tension Rupture

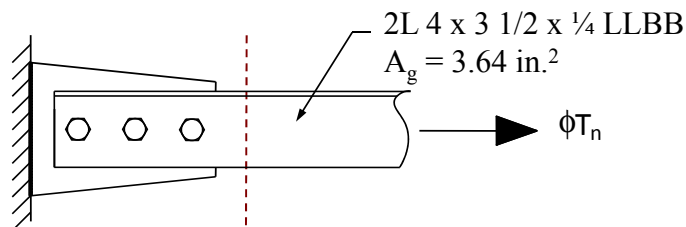
Weld:

7-7 Weld Rupture



Light Bracing Connection

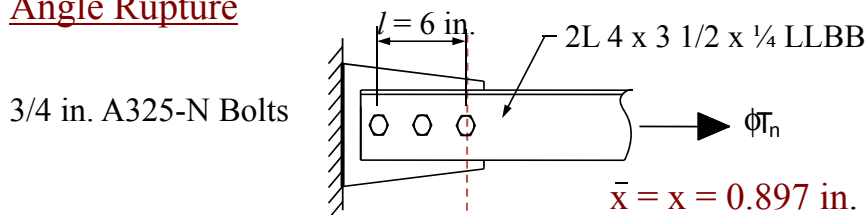
Angle Yielding



$$\begin{aligned} \phi T_n &= 0.9 F_y A_g \\ &= 0.9 \times 36 \times 3.64 = \underline{118 \text{ k}} \end{aligned}$$

Light Bracing Connection

Angle Rupture



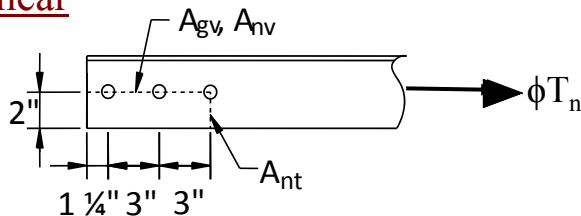
3/4 in. A325-N Bolts

$$\begin{aligned} \phi T_n &= 0.75 F_u A_e = 0.75 F_u U A_n \\ A_n &= A_g - A_h = 3.64 - (0.5) (3/4 + 1/16 + 1/16) \\ &= 3.18 \text{ in}^2 \\ U &= 1 - \bar{x}/l = 1 - 0.897/6 = 0.850 \end{aligned}$$

$$\phi T_n = 0.75 \times 58 \times 0.850 \times 3.18 = \underline{118 \text{ k}}$$

Light Bracing Connection

Angle Block Shear



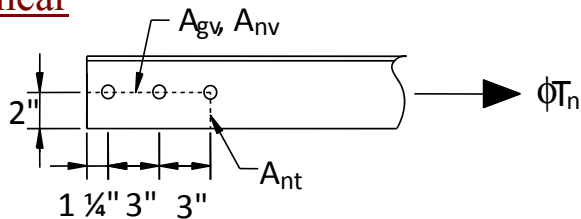
$$A_{nv} = 0.5 [7.25 - (2.5 \times 7/8)] = 2.53 \text{ in}^2$$

$$A_{gv} = 0.5 \times 7.25 = 3.625 \text{ in}^2$$

$$A_{nt} = 0.5 [2.0 - (0.5 \times 7/8)] = 0.781 \text{ in}^2$$

Light Bracing Connection

Angle Block Shear



$$R_n = \min \left\{ \begin{array}{l} \text{Shear Rupture} \\ \text{Shear Yield} \end{array} \right. + U_{bs} \text{ Tension Rupture}$$

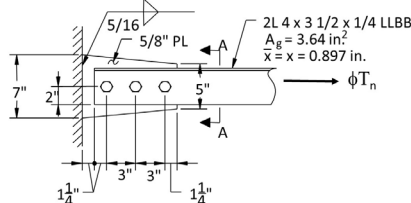
$$= \min \left\{ \begin{array}{l} 0.6 \times 58 \times 2.53 = 88.0 \\ 0.6 \times 36 \times 3.625 = \underline{78.3} \end{array} \right. + 1.0 \times 58 \times 0.781$$

$$\phi R_n = 0.75 (78.3 + 45.3) = \underline{92.7 \text{ k}}$$

Light Bracing Connection

Shear Transfer Between Elements

*Specification Section
J3.6 User Note*



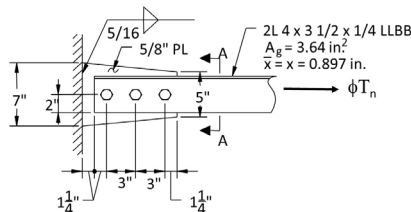
Strength at Each Bolt/Hole is the minimum of:

- Angle Bearing / Tear Out
- Bolt Shear Rupture
- Plate Bearing / Tear Out

Light Bracing Connection

Shear Transfer Between Elements

Angle Bearing / Tear Out:



$$\text{Bearing: } 2.4F_u t d = (2.4 \times 58) (0.5 \times 3/4) = \mathbf{52.2k}$$

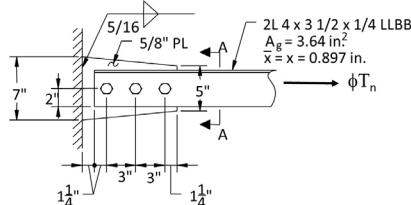
$$\text{Edge: } 1.2F_u l_c t = (1.2 \times 58) (1.25 - 13/32) (0.5) \\ = \mathbf{29.4k} < 52.2k \text{ (Tear-Out Controls)}$$

$$\text{Other: } 1.2F_u l_c t = (1.2 \times 58) (3.0 - 13/16) (0.50) \\ = 76.1k > \mathbf{52.2k} \text{ (Bearing Controls)}$$

Light Bracing Connection

Shear Transfer Between Elements

Bolt Shear Rupture:



3/4 in. A325-N Bolts in Double Shear

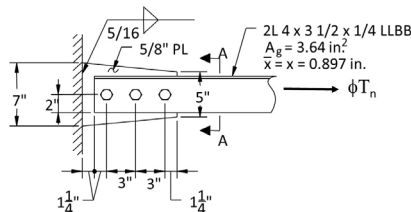
With $F_{nv} = 54.0$ ksi from *Specification* Table J3.2

$$r_v = F_{nv} A_b n_s = 54 \times 0.4418 \times 2 \\ = \mathbf{47.7k/bolt} \text{ (nominal strength)}$$

Light Bracing Connection

Shear Transfer Between Elements

Plate Bearing / Tear Out:



Bearing: $2.4F_u t d = (2.4 \times 58) (0.625 \times 3/4) = 65.3 \text{ k}$

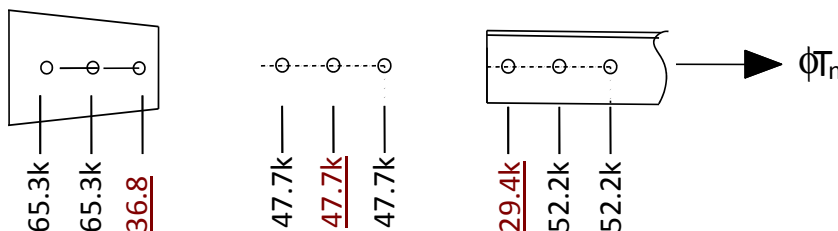
Edge: $1.2F_u L_c t = (1.2 \times 58) (1.25 - 13/32) (0.625) = 36.8 \text{ k} < 65.3 \text{ k}$ (Tear-Out Controls)

Other: $1.2F_u L_c t = (1.2 \times 58) (3.0 - 13/16) (0.625) = 95.1 \text{ k} > 65.3 \text{ k}$ (Bearing Controls)

Light Bracing Connection

Shear Transfer Between Elements

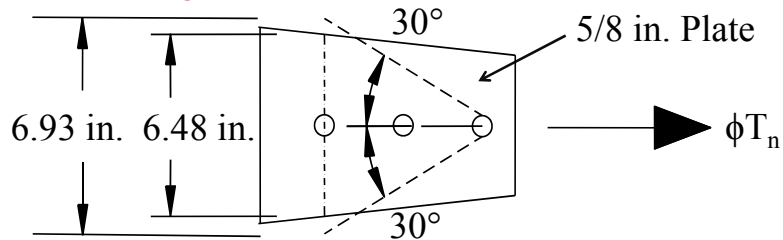
Plate Brg./T.O.
Bolt Rupture
Angles Brg./T.O.



$\phi T_n = 0.75(36.8 + 47.7 + 29.4) = 85.4 \text{ k}$

Light Bracing Connection

Plate Yielding at Whitmore Section:

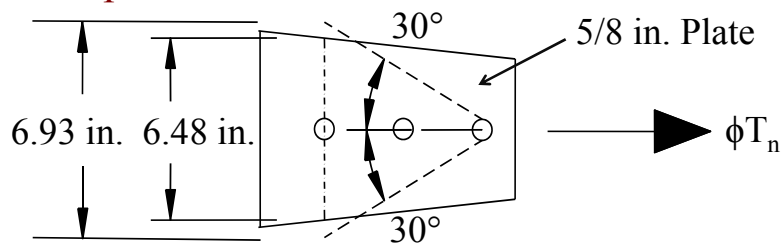


$$A_g = 0.625 \times 6.48 = 4.05 \text{ in}^2$$

$$\begin{aligned} \phi T_n &= 0.9 F_y A_g \\ &= (0.9 \times 36) (4.05) = \underline{131 \text{ k}} \end{aligned}$$

Light Bracing Connection

Plate Rupture at Whitmore Section



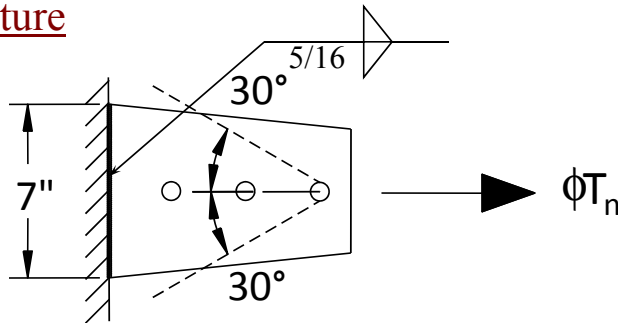
$$A_e = U A_n \quad U = 1.0 \text{ for plates}$$

$$\begin{aligned} A_n &= (6.48 - 0.875)(0.625) \\ &= 3.50 \text{ in}^2 \end{aligned}$$

$$\begin{aligned} \phi T_n &= 0.75 F_u A_e \\ &= (0.75 \times 58) (1.0 \times 3.50) = \underline{152 \text{ k}} \end{aligned}$$

Light Bracing Connection

Weld Rupture

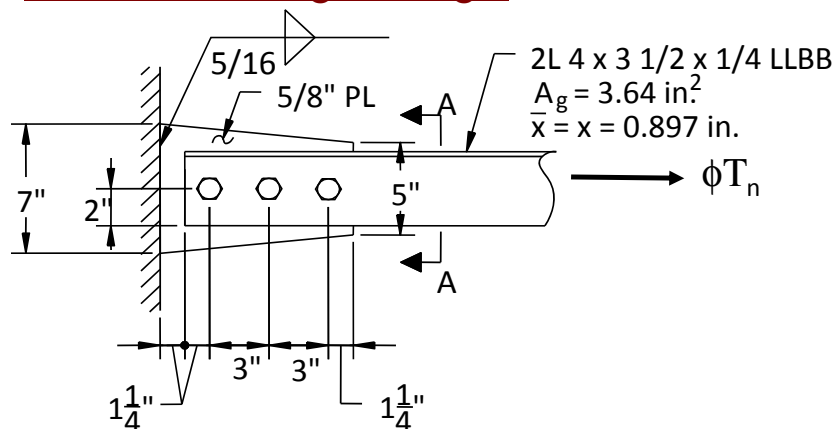


$$T_n = 1.392 (1.5) D L_{\text{weld}}$$

$$= 1.392 (1.5) (2 \times 5) 7.0 = \underline{146 \text{ k}}$$

Light Bracing Connection

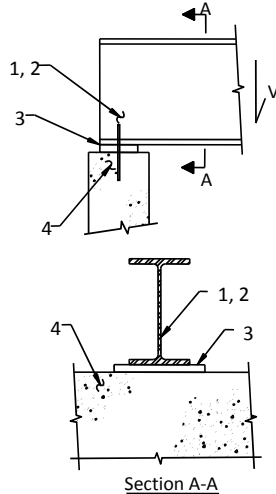
Connection Design Strength



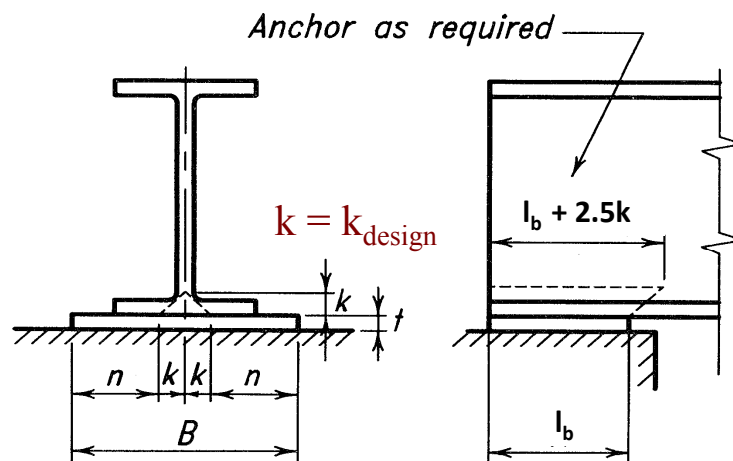
$$\phi T_n = \underline{85.4 \text{ k (Shear Transfer Between Elements)}}$$

BEAM BEARING LIMIT STATES

1. Beam Web Local Yielding
(Specification J10.2)
2. Beam Web Local Crippling
(Specification J10.3)
3. Bearing Plate Bending
4. Bearing on Concrete
(Specification J8)

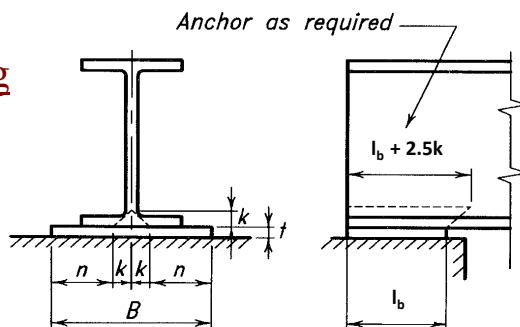


Beam Web Local Yielding



Beam Web Local Yielding

Specification J10.2 Web Local Yielding



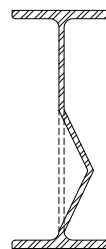
$$\phi = 1.0$$

$$R_n = (2.5k_{\text{design}} + l_b)F_{yw} t_w \quad \text{at supports}$$

$$R_n = (5.0k_{\text{design}} + l_b)F_{yw} t_w \quad \text{interior loads}$$

Beam Web Local Crippling

Specification J10.3 Web Local Crippling @ $> d/2$



$$\phi = 0.75$$

$$R_n = 0.8 t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f \quad (\text{Spec. J10-4})$$

Note: $Q_f = 1.0$ for wide flange sections.

Beam Web Local Crippling

Specification J10.3

Web Local Crippling @ $< d/2$

$$l_b/d \leq 0.2$$

$$R_n = 0.40 t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f \quad (\text{J10-5a})$$

$$l_b/d > 0.2$$

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4 l_b}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f \quad (\text{J10-5b})$$

Bearing on Concrete

Specification J8

Column Bases and Bearing on Concrete

$$\phi_c = 0.65$$

(a) On the full area of a concrete support

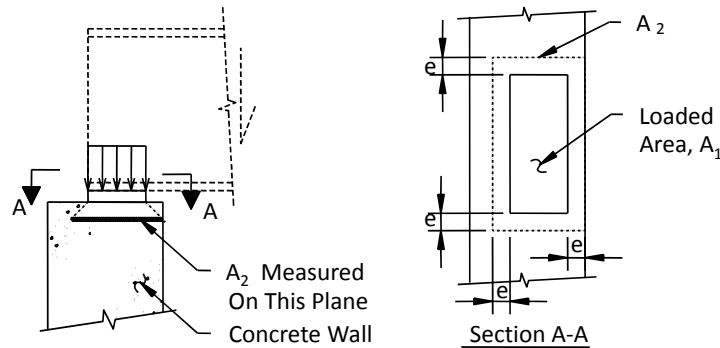
$$P_p = 0.85 f'_c A_1 \quad (\text{Spec. J8-1})$$

(b) On less than the full area of a concrete support

$$P_p = 0.85 f'_c A_1 \sqrt{A_2/A_1} \leq 1.7 f'_c A_1 \quad (\text{Spec. J8-2})$$

Note: The limit $\leq 1.7 f'_c$ is equivalent to $A_2/A_1 \leq 4$

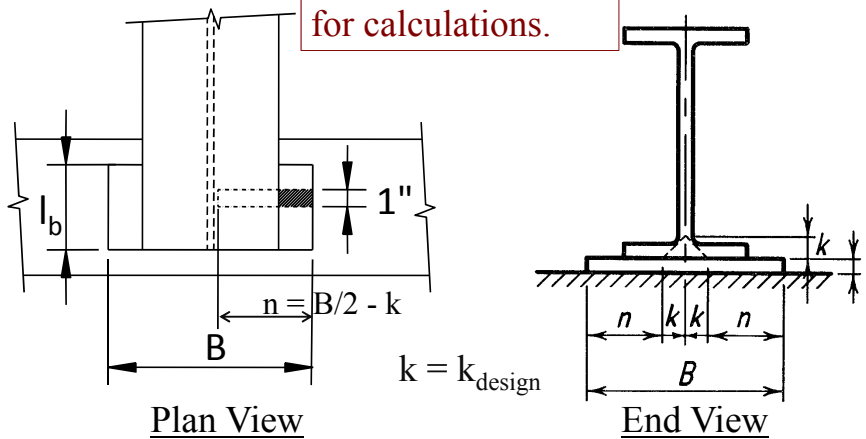
Beam Bearing: Concrete Crushing



A_1 = area of steel bearing on concrete, in.²
 A_2 = area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.²

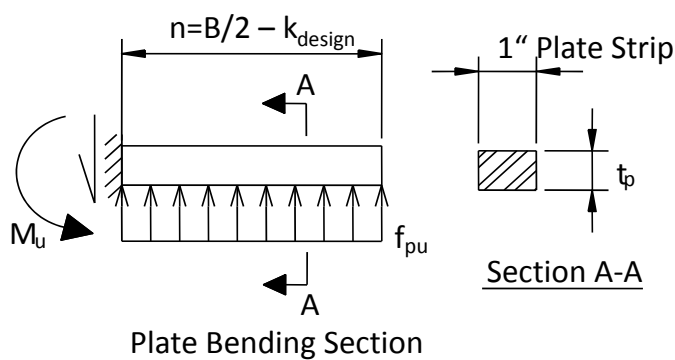
Beam Bearing: Plate Bending

Note: 1 in. wide plate strip is used for calculations.



$k = k_{\text{design}}$

Beam Bearing: Plate Bending

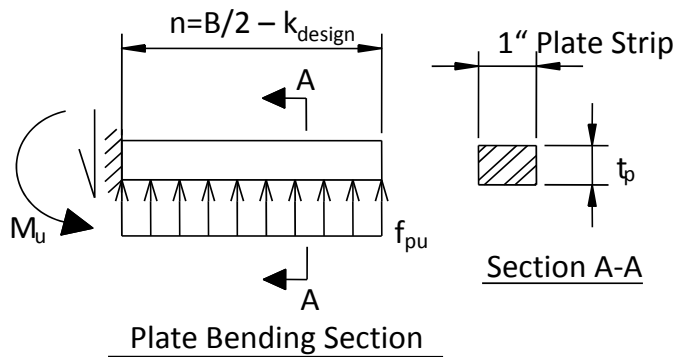


$$f_{pu} = R_u / A_1 = \text{pressure due to reaction}$$

$$M_u = f_{pu} (1) n^2 / 2 \leq \phi M_p$$

$$\phi M_p = 0.9 F_y Z_x = 0.9 F_y (1 \times t_p^2 / 4)$$

Beam Bearing Plate Example



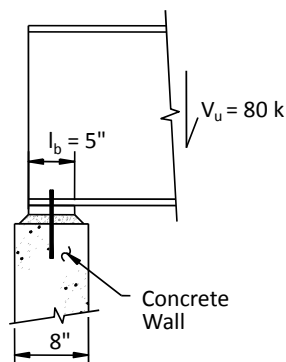
Substituting:

$$t_{p, \text{min}} = \sqrt{\frac{2 f_{pu} n^2}{0.9 F_y}}$$

Beam Bearing Plate Example

Ex. Determine if $l_b = 5$ in. is adequate.
Determine required Plate Width, B .
Determine required Plate Thickness, t_p .

$R_u = 80$ k
Plate $F_y = 36$ ksi
Concrete $f'_c = 3.0$ ksi



Beam Bearing Plate Example

Properties: W18x76 A992
 $F_y = 50$ ksi
 $b_f = 11.0$ in.
 $t_f = 0.680$ in.
 $d = 18.2$ in.
 $t_w = 0.425$ in.
 $k = 1.08$ in. (design)

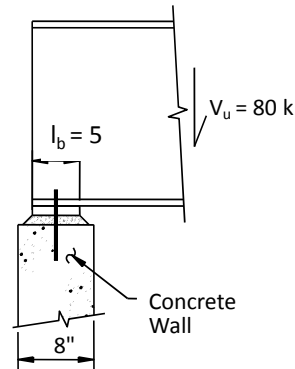
Beam Bearing Plate Example

Web Local Yielding:

$$F_{yw} = 50 \text{ ksi}$$

$$k_{\text{design}} = 1.08 \text{ in.}$$

$$t_w = 0.425 \text{ in.}$$



$$\phi R_n = 1.0 (2.5k_{\text{design}} + l_b) F_{yw} t_w$$

$$= 1.0 (2.5 \times 1.08 + 5.0) (50 \times 0.425)$$

$$= 164 \text{ k} > R_u = 80 \text{ k} \quad \text{OK}$$

Beam Bearing Plate Example

Web Local Crippling: $F_y = 50 \text{ ksi}$

$$At \leq d/2:$$

$$l_b / d = 5 / 18.21 = 0.27 > 0.2 \quad Q_f = 1.0$$

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4 l_b}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E_s F_y t_f}{t_w}} Q_f \quad (\text{Spec. J10-5b})$$

$$= 0.40 \times 0.425^2 \left[1 + \left(\frac{4 \times 5.0}{18.21} - 0.2 \right) \left(\frac{0.425}{0.680} \right)^{1.5} \right] \sqrt{\frac{29000 \times 50 \times 0.680}{0.425}}$$

$$= 159 \text{ k}$$

$$\phi R_n = 0.75 \times 159 = 119 \text{ k} > R_u = 80 \text{ k} \quad \text{OK}$$

$l_b = 5 \text{ in.}$ is Adequate

Beam Bearing Plate Example

Plan View

A_1 (Try Plate 5 x 13)
 $B > b_f = 11.0$ in.
 A_2 (8 in. x 16 in.)

16" 13" 11" 5" 8"

VirginiaTech 77
Invent the Future

Beam Bearing Plate Example

Concrete Crushing:
 With: $B = 13$ in. $> b_f = 11$ in.
 $l_b = 5$ in.

$A_1 = 5 \times 13 = 65$ in²
 $A_2 = 8 \times 16 = 128$ in²

$A_2 / A_1 = 1.96 < 4$

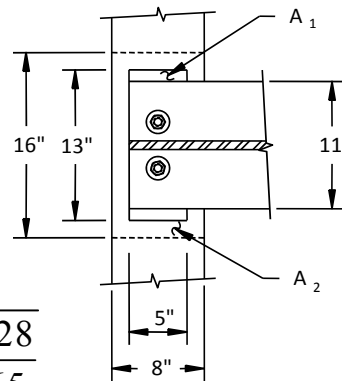
16" 13" 11" 5" 8"

VirginiaTech 78
Invent the Future

Beam Bearing Plate Example

Concrete Crushing:

$$\begin{aligned} \phi P_p &= \phi 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \\ &= 0.65 (0.85 \times 3.0) (65) \sqrt{\frac{128}{65}} \\ &= 151 \text{ k} > 80 \text{ k} \quad \underline{\text{OK}} \end{aligned}$$



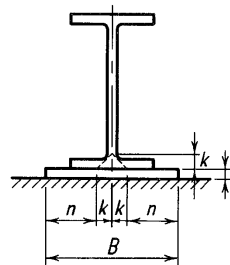
Beam Bearing Plate Example

Plate Bending:

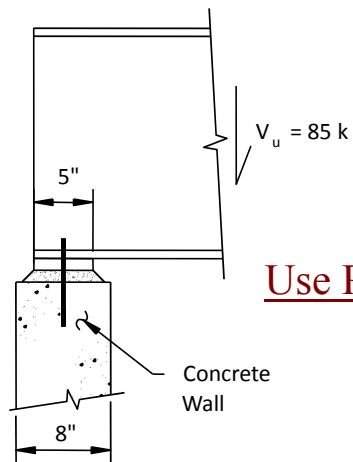
$$\begin{aligned} F_y &= 36 \text{ ksi} \\ f_{pu} &= R_u / A_1 = 80 / (5 \times 13) = 1.23 \text{ ksi} \\ n &= B/2 - k_{\text{design}} = 13/2 - 1.08 \\ &= 5.42 \text{ in.} \end{aligned}$$

$$\begin{aligned} t_p &= \sqrt{\frac{2f_{pu} n^2}{0.9F_y}} \\ &= \sqrt{\frac{2 \times 1.23 \times 5.42^2}{0.9 \times 36}} = 1.49 \text{ in.} \end{aligned}$$

Use PL 1-1/2 in.



Beam Bearing Plate Example



Use PL 1-1/2 x 5 x 1'-1" A36

COLUMN BASE PLATE DESIGN



Column Base Plate Design

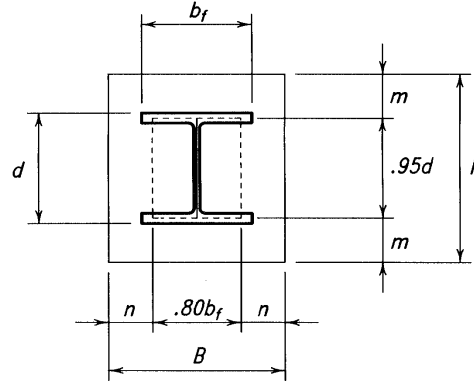
Required Column Base Plate Thickness

$$f_{pu} = R_u / BN$$

Let $m' = \max(m \text{ or } n)$

$$M_u = f_{pu} (1) m'^2 / 2$$

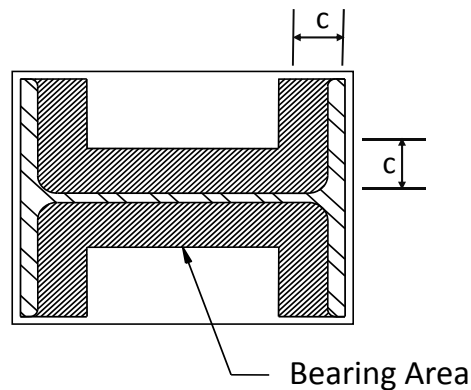
$$t_p = \sqrt{\frac{2 f_{pu} m'^2}{0.9 F_y}}$$



What is t_p if m and n are very small?

Column Base Plate Design

Lightly Loaded Column Base Plates



Column Base Plate Design

Lightly Loaded Column Base Plates

For lightly loaded base plates, replace m' with

$$l = \max \{m, n, \lambda n'\}, \text{ where}$$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1.0$$

$$n' = \frac{1}{4} \sqrt{db_f}$$

and

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

$$t_p = \sqrt{\frac{2f_{pu}\ell^2}{0.9F_y}}$$

Column Base Plate Design Example

Ex.: Determine if the column base plate shown is adequate if $P_u = 250$ k.

Column:

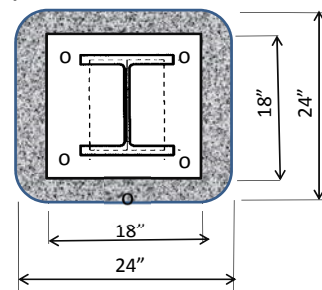
W10x33 $d = 9.73$ in. $b_f = 7.96$ in.

PL 1-1/2 x 18 x 1'-6" A36

Concrete Pedestal:

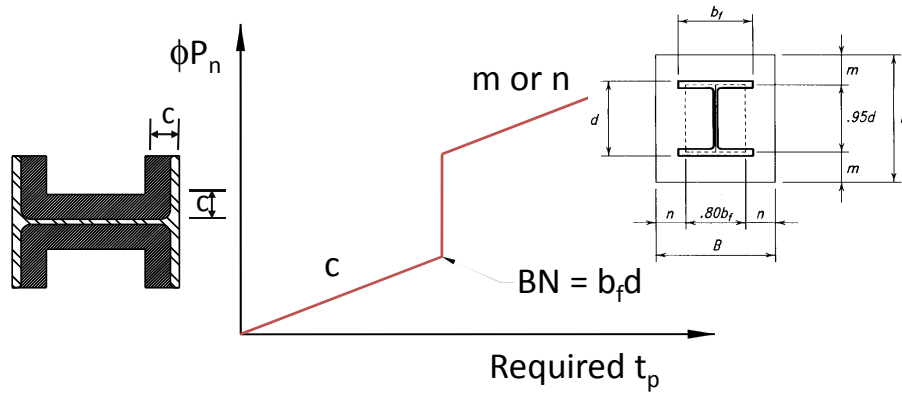
24 in. by 24 in.

$f'_c = 3.0$ ksi



Column Base Plate Design

Variation of ϕP_n with Base Plate Thickness



End of Session 2

Thank You for
 Attending

Next Up

Next Session

- October 17, 2017 Shear Connections Part I

TOPICS

- Types of Framing Connections
- Design Considerations
- New Limit States for Framing Connections
- Shear End-Plate Connections
- Double Angle Connections

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days...

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

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days...

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



8-Session Registrants

CEU/PDH Certificates

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



8-Session Registrants

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

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

Reasons for quiz:

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

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



8-Session Registrants

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

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



Night School Resources for 8-session package Registrants

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



Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.



Login

If you're an existing customer, please enter your username and password.

USERNAME

Enter your username

PASSWORD

Enter your password

Remember Me

DON'T HAVE AN ACCOUNT?

My AISC allows you to access Engineering Journal articles and Design Guides you have downloaded from the bookstore.

[REGISTER NOW](#)

Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.

IN THIS SECTION

- Edit Profile
- My Downloads
- My Pending Quizzes
- My Events
- Order History
- Course History
- Course Resources**

MyAISC

MY PROFILE
Update your contact and address information.
[EDIT PROFILE](#)

MY PURCHASED DOWNLOADS
Access articles and documents that you have purchased.
[VIEW DOWNLOADS](#)

MY COURSE RESOURCES
View online resources for Night School and Live Webinar package registrations.
[VIEW RESOURCES](#)

Night School Resources for 8-session package Registrants

EDUCATION PUBLICATIONS NASCC: THE STEEL CONFERENCE STEEL SOLUTIONS CENTER AWARDS AND COMPETITIONS TECHNICAL RESOURCES

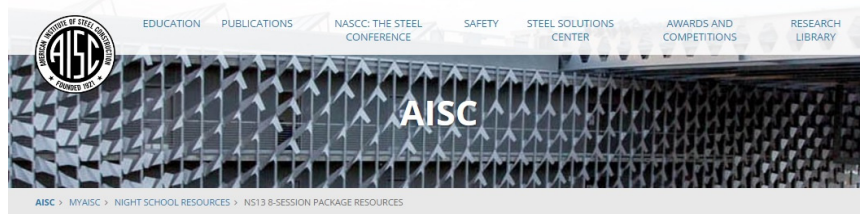
AISC

AISC > MYAISC > COURSE RESOURCES

Course Resources

Event	Start Date
NS 13 8-Session Package-Night School 13 - Design of Industrial Buildings	1/30/2017 7:00:00 PM
NS 14 8-Session Package-Night School 14 - Fundamentals of Stability	6/5/2017 7:00:00 PM

Night School Resources for 8-session package Registrants



Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	View	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Dcn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	

Night School Resources for 8-session package Registrants

- Weekly “quiz and recording” email.
- Weekly updates of the master Quiz and Attendance record found at www.aisc.org/nightschool. Scroll down to Quiz and Attendance records.
 - Updated on Wednesday mornings.

Night School Resources for All Registrants

- Webinar connection information:
 - Found in your registration confirmation/receipt.
 - Reminder email sent out Tuesday mornings.
- Link to handouts also found here.

Thank You

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

There's always a solution in steel.

