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Fundamentals of Connection Design

Session 2: Fundamental Concepts, Part II
October 30, 2019



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Course Description

Fundamental Concepts, Part II
October 30, 2019

This session will discuss eccentric bolted and welded connections, directly-loaded tension connections, block shear, the Whitmore Section, and light bracing connections. Beam bearing and column base plate design will be discussed. Design examples will be presented to demonstrate concepts.



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Learning Objectives

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



Fundamentals of Connection Design

Session 2: Fundamental Concepts, Part II
October 30, 2019



Brad Davis, PhD, SE
Associate Professor, University of Kentucky
Owner, Davis Structural Engineering



Schedule	
• October 23, 2019	Fundamental Concepts Part I
• October 30, 2019	Fundamental Concepts Part II
• November 6, 2019	Shear Connections Part I
• November 13, 2019	Shear Connections Part II

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**FUNDAMENTAL
CONCEPTS
PART II**

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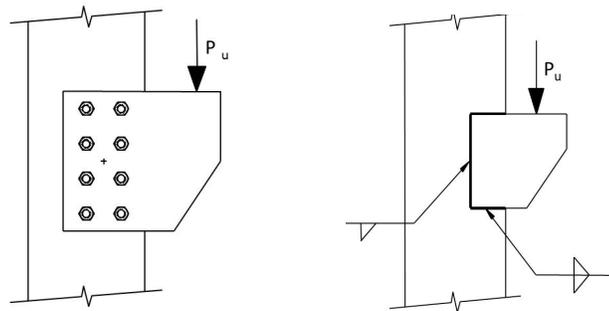
Topics

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



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ECCENTRICALLY LOADED BOLTED AND WELDED CONNECTIONS



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Bolts: Eccentric Connections

Elastic Method



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Bolts: Eccentric Connections

Instantaneous Center of Rotation Method



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Bolts: Eccentric Connections

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

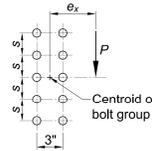
$$R_n = C r_n$$

or

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

where

P = required force, P_u or P_d , 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



$$\phi P_n = C \phi r_{nv}$$

$$\phi = 0.75$$

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
3	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



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Example: Eccentric Bolted Connection

Determine ϕP_n

$$\phi P_n = C \phi r_{nv}$$

From *Manual* Table 7-7, Angle = 0°

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

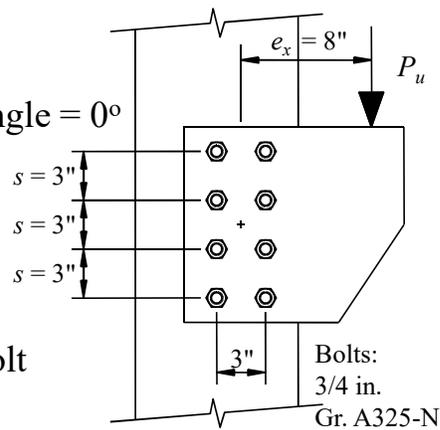
$$C = 2.93$$

From *Manual* Table 7-1,

for 3/4 in. Gr. A325-N:

$$\phi r_{nv} = \phi F_{nv} A_b = 17.9 \text{ kips/bolt}$$

$$\phi P_n = (2.93)(17.9) = 52.4 \text{ kips}$$



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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

$$R_n = C r_n$$

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_o , 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

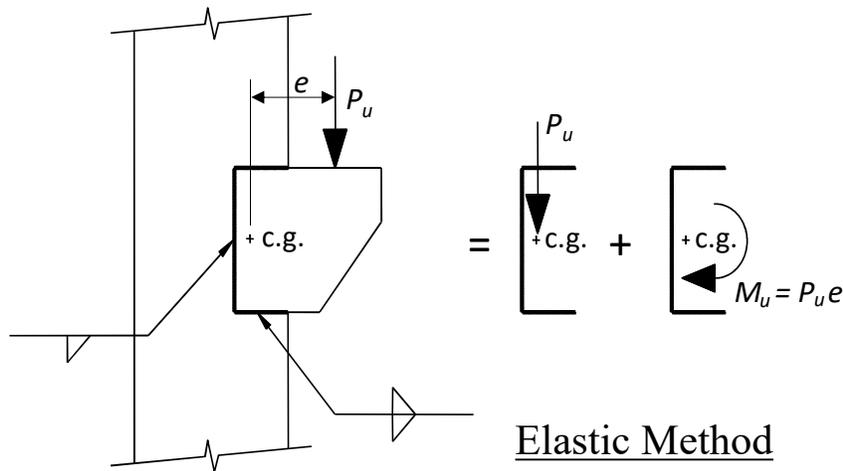
$s = 3$ in.
 $n = 4$
 $e_x = 8$ in.
 $C = 2.93$

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
3	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
3	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



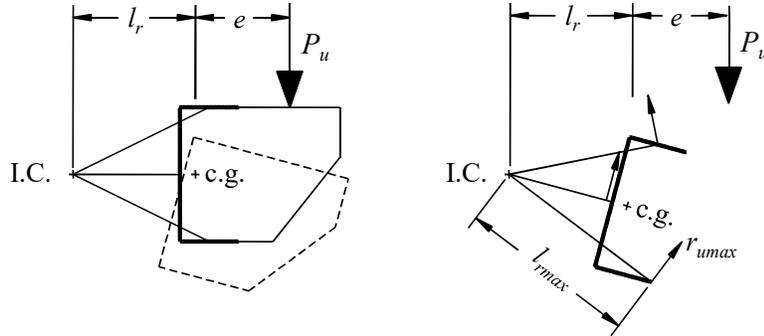
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Welds: Eccentric Connections



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Welds: Eccentric Connections



Instantaneous Center of Rotation Method



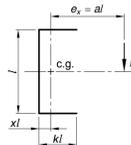
Welds: Eccentric Connections

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				
$C_{min} = \frac{P_u}{\phi C_1 D l}$	$D_{min} = \frac{P_u}{\phi C C_1 l}$	$l_{min} = \frac{P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_u}{C_1 D l}$	$D_{min} = \frac{\Omega P_u}{C C_1 l}$	$l_{min} = \frac{\Omega P_u}{C C_1 D}$				

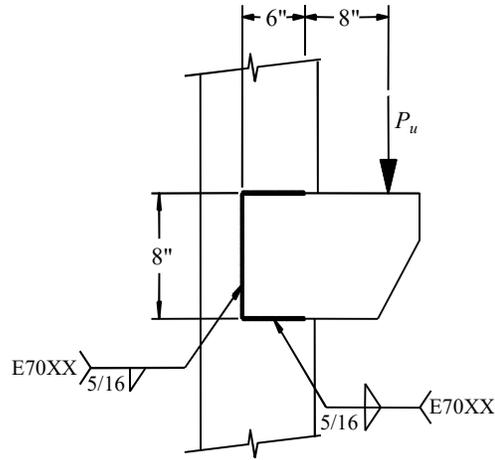
where
 P = required force, P_u or P_u/ϕ , kips
 D = number of sixteenths-of-an-inch in the fillet weld size
 l = characteristic length of weld group, in.
 $a = e_x/l$
 e_x = horizontal component of eccentricity of P
 with respect to centroid of weld group, in.
 C = coefficient tabulated below
 C_1 = 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 Section J2.4.

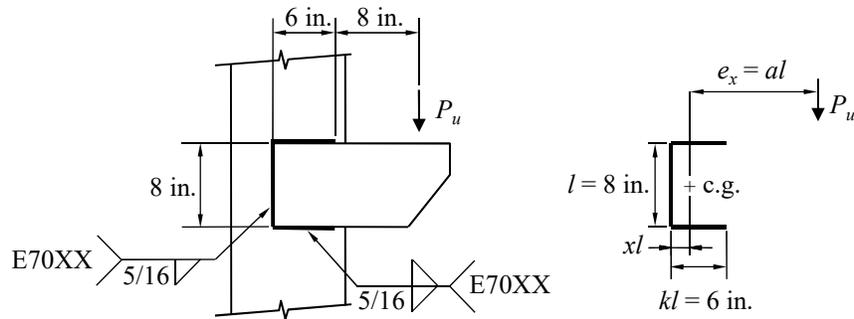
a	k																
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	8.26	9.37	10.5	11.6	12.7	
0.10	1.86	2.28	2.78	3.30	3.83	4.37	4.92	5.46	6.01	6.56	7.11	8.22	9.32	10.4	11.5	12.7	
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	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	5.88	6.38	7.37	8.39	9.42	10.5	11.5	
0.30	1.54	1.94	2.36	2.82	3.29	3.77	4.25	4.74	5.24	5.75	6.25	7.17	8.12	9.16	10.2	11.2	
0.35	1.41	1.80	2.20	2.66	3.13	3.61	4.09	4.58	5.08	5.59	6.09	6.94	7.82	8.86	9.91	10.9	
0.40	1.27	1.65	2.03	2.48	2.95	3.42	3.90	4.38	4.87	5.37	5.88	6.66	7.56	8.61	9.66	10.7	
0.45	1.13	1.49	1.86	2.29	2.75	3.21	3.68	4.15	4.63	5.11	5.60	6.31	7.23	8.28	9.33	10.4	
0.50	1.00	1.34	1.69	2.10	2.54	2.98	3.44	3.90	4.37	4.84	5.32	5.96	6.80	7.85	8.90	10.0	
0.55	0.88	1.19	1.52	1.90	2.31	2.73	3.17	3.61	4.06	4.51	4.97	5.63	6.48	7.53	8.58	9.63	
0.60	0.77	1.05	1.36	1.72	2.10	2.50	2.93	3.36	3.80	4.24	4.69	5.37	6.23	7.28	8.33	9.38	
0.65	0.67	0.92	1.20	1.54	1.89	2.26	2.65	3.04	3.43	3.82	4.27	4.97	5.84	6.89	7.94	8.99	
0.70	0.58	0.80	1.05	1.35	1.66	1.98	2.34	2.69	3.04	3.39	3.84	4.56	5.44	6.49	7.54	8.59	
0.75	0.50	0.69	0.91	1.16	1.42	1.68	1.99	2.29	2.58	2.87	3.31	4.04	4.92	5.97	7.02	8.07	
0.80	0.43	0.59	0.77	0.98	1.19	1.38	1.62	1.85	2.07	2.28	2.70	3.44	4.32	5.37	6.42	7.47	
0.85	0.37	0.50	0.64	0.82	0.99	1.14	1.33	1.51	1.68	1.84	2.24	2.98	3.86	4.91	5.96	7.01	
0.90	0.32	0.43	0.54	0.68	0.82	0.94	1.09	1.23	1.37	1.50	1.88	2.62	3.50	4.55	5.60	6.65	
0.95	0.27	0.36	0.44	0.55	0.65	0.74	0.85	0.94	1.03	1.11	1.48	2.22	3.10	4.15	5.20	6.25	
1.00	0.23	0.30	0.36	0.44	0.50	0.56	0.61	0.66	0.70	0.74	1.10	1.84	2.72	3.77	4.82	5.87	
1.05	0.20	0.26	0.31	0.36	0.40	0.44	0.47	0.50	0.52	0.54	0.90	1.64	2.52	3.57	4.62	5.67	
1.10	0.17	0.22	0.26	0.29	0.32	0.34	0.36	0.37	0.38	0.39	0.74	1.48	2.36	3.41	4.46	5.51	
1.15	0.15	0.19	0.22	0.24	0.25	0.26	0.27	0.27	0.28	0.28	0.69	1.42	2.30	3.35	4.40	5.45	
1.20	0.13	0.16	0.18	0.19	0.20	0.20	0.20	0.20	0.20	0.20	0.64	1.36	2.24	3.29	4.34	5.39	
1.25	0.11	0.14	0.15	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.60	1.30	2.18	3.23	4.28	5.33	
1.30	0.10	0.12	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.13	0.57	1.24	2.12	3.17	4.22	5.27	
1.35	0.09	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.54	1.18	2.06	3.11	4.16	5.21	
1.40	0.08	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.51	1.12	1.99	3.04	4.09	5.14	
1.45	0.07	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.48	1.06	1.93	2.98	4.03	5.08	
1.50	0.06	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.45	1.00	1.87	2.92	3.97	5.02	
1.55	0.05	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.42	0.94	1.81	2.86	3.91	4.96	
1.60	0.04	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.39	0.88	1.75	2.80	3.85	4.90	
1.65	0.03	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.36	0.82	1.67	2.72	3.77	4.82	
1.70	0.02	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.33	0.76	1.58	2.64	3.69	4.74	
1.75	0.01	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.30	0.70	1.49	2.56	3.61	4.66	
1.80	0.01	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.27	0.64	1.40	2.47	3.52	4.57	
1.85	0.01	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.24	0.58	1.31	2.38	3.43	4.48	
1.90	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.21	0.52	1.22	2.29	3.34	4.39	
1.95	0.01																

Example: Determine ϕP_n



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Example: Determine ϕP_n



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

Manual Table 8-8: $x = 0.225$

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



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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

$k = 0.75 \rightarrow x = 0.225$



Example: Determine ϕP_n

$e_x = al$

$6 \text{ in.} + 8 \text{ in.} - 1.8 \text{ in.} = (a)(8 \text{ in.})$

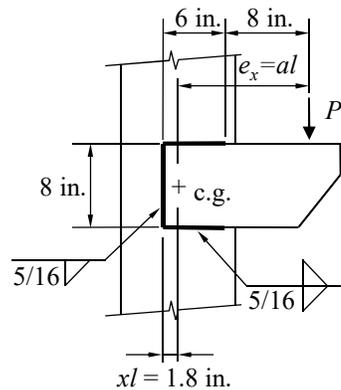
$a = 1.53$

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

5/16 in. welds $\rightarrow D = 5$

E70XX $\rightarrow C_1 = 1.00$

$\phi P_n = \phi C C_1 D l$
 $= (0.75)(1.59)(1.00)(5)(8.0 \text{ in.})$
 $= 47.7 \text{ kips}$



Example: Determine ϕP_n

Manual Table 8-8

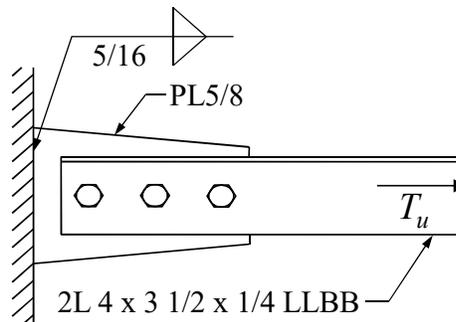
$k = 0.75$ and $a = 1.53 \rightarrow C = 1.59$

a	k								
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03
0.10	1.86	2.28	2.78	3.30	3.83	4.37	4.92	5.46	6.01
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87
0.20	1.76	2.18	2.63	3.11	3.60	4.11	4.61	5.13	5.64
0.25	1.66	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37
0.30	1.55	1.95	2.36	2.79	3.23	3.68	4.14	4.60	5.08
0.40	1.33	1.69	2.07	2.45	2.84	3.24	3.65	4.07	4.50
0.50	1.15	1.46	1.79	2.14	2.49	2.85	3.22	3.60	4.00
0.60	0.999	1.27	1.57	1.88	2.19	2.52	2.85	3.20	3.57
0.70	0.879	1.12	1.38	1.66	1.95	2.24	2.55	2.87	3.20
0.80	0.783	0.996	1.23	1.48	1.75	2.02	2.30	2.59	2.90
0.90	0.704	0.896	1.11	1.34	1.58	1.83	2.09	2.36	2.65
1.0	0.639	0.813	1.00	1.21	1.44	1.67	1.91	2.16	2.43
1.2	0.538	0.684	0.845	1.02	1.21	1.42	1.63	1.85	2.08
1.4	0.464	0.589	0.729	0.883	1.05	1.23	1.42	1.61	1.82
1.6	0.408	0.517	0.640	0.775	0.924	1.09	1.25	1.43	1.61
1.8	0.363	0.461	0.570	0.691	0.825	0.970	1.12	1.28	1.45



25

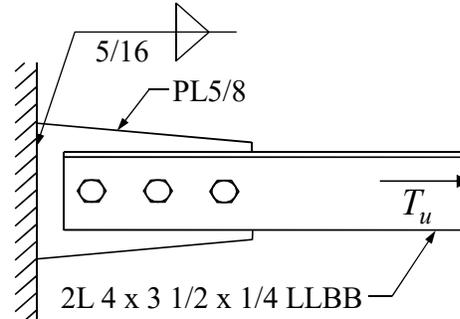
DIRECTLY LOADED CONNECTIONS



26

Applicable Limit States

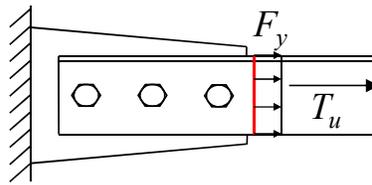
- Tensile Yielding
- Tensile Rupture
- Block Shear
- Shear Transfer
- Whitmore Section Considerations
- Weld Rupture
- Support Limit States
- Evaluation Criterion: $T_u \leq \phi T_n$



27

Limit State: Tensile Yielding

Specification D2 and J4.1



$$\phi T_n = \phi F_y A_g \quad (\text{Spec. D2-1, J4-1})$$

$$\phi = 0.9$$

Note: Tensile yielding is a member limit state.



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Limit State: Tensile Rupture

Specification D2 and J4.1

$$\phi T_n = \phi F_u A_e$$

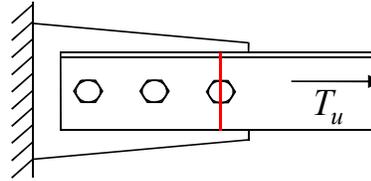
$$\phi = 0.75$$

F_u = tensile strength
 = 58 ksi for A36;
 65 ksi for A992, A529 Gr. 50, and A572 Gr. 50

A_e = effective net area = $U A_n$

U = shear lag factor

A_n = net area



Limit State: Tensile Rupture

Shear Lag Factor

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 = 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: Tensile Rupture

Example: *Specification* Table D3.1, Case 2

Use out-to-out distance for l

Commentary Figure C-D3.2



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Limit State: Tensile Rupture

(a)

(b)

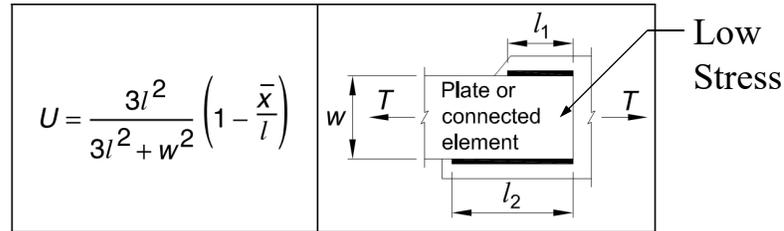
Commentary Figure C-D3.1

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


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Limit State: Tensile Rupture

Example: *Specification* Table D3.1 Case 4.



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

Note: l_1 and $l_2 \geq 4$ times fillet weld size.

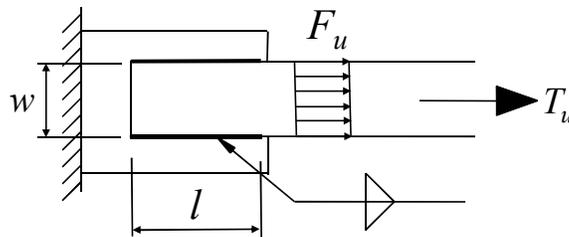


33

Limit State: Tensile Rupture

Example: *Specification* Table D3.1 Case 4.

Plate with equal length welds.



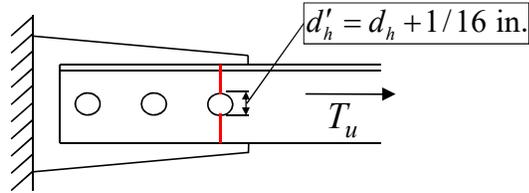
$$U = \frac{3l^2}{3l^2 + w^2} \left(1 - \frac{\bar{x}}{l}\right) \Rightarrow U = \frac{3l^2}{3l^2 + w^2}$$



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Limit State: Tensile Rupture

Net Area



$$A_n = A_g - \sum A'_h + \sum \text{Stagger}$$

A_g = gross area of cross-section

A'_h = effective area of hole

$$= (t)(d_h + 1/16 \text{ in.}) \text{ per Spec. B4.3b}$$

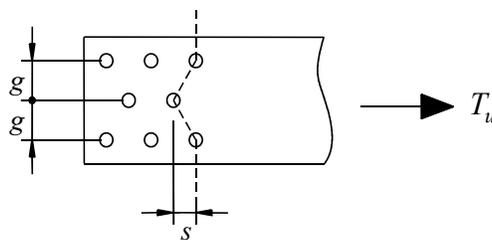
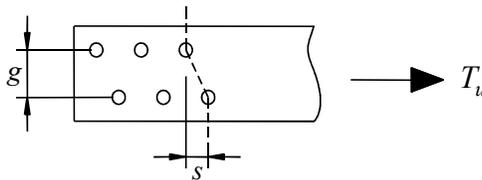
Add “Stagger” term(s) if applicable.



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Limit State: Tensile Rupture

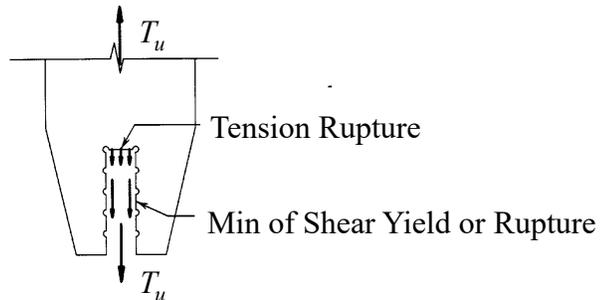
$$\text{Stagger Term} = (s^2/4g)t \text{ (Spec. B4.3b)}$$



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Limit State: Block Shear Strength

Specification J4.3

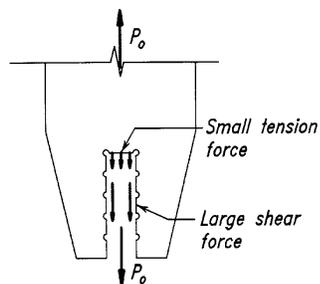


- Failure occurs when shear forces reach the smaller of shear yield and shear rupture.
- The tension area is at rupture when failure occurs.



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Limit State: Block Shear

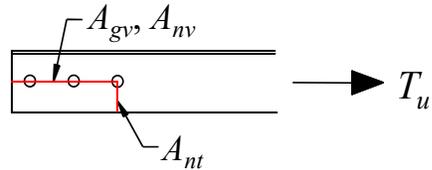


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Limit State: Block Shear Strength

Specification J4.3

$$\phi = 0.75$$



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

$$R_n = \min \begin{cases} \text{Shear Rupture} \\ \text{Shear Yielding} \end{cases} + \text{Tensile Rupture}$$

$$= \min \begin{cases} 0.6F_u A_{nv} \\ 0.6F_y A_{gv} \end{cases} + U_{bs} F_u A_{nt}$$



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Limit State: Block Shear Strength

Example

$$A_{nv} = 2.53 \text{ in.}^2 \quad A_{gv} = 3.63 \text{ in.}^2$$

$$A_{nt} = 0.781 \text{ in.}^2 \quad U_{bs} = 1.0$$

$$\text{A36 Steel: } F_y = 36 \text{ ksi } F_u = 58 \text{ ksi}$$

$$R_n = \min \begin{cases} 0.6(58)(2.53) = 88.0 \\ 0.6(36)(3.63) = 78.3 \end{cases} + (1.0)(58)(0.781)$$

$$= 78.3 + 45.3 = 124 \text{ kips}$$

$$\phi R_n = (0.75)(124) = 92.7 \text{ kips}$$



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Limit State: Block Shear Strength

Welded Connections

Case I
Case II

Note: $A_{nv} = A_{gv}$

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Limit State: Plate Compression

Specification Section J4.4

$$P_u \leq \phi P_n \quad \phi = 0.90$$

If $L_c/r \leq 25$, then

$$\phi P_n = \phi F_y A_g$$

Otherwise

$$\phi P_n \text{ per Chapter E}$$

$$L_c = KL \rightarrow \text{See Design Guide 29}$$

Yielding or Buckling

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Whitmore Section

The diagram illustrates a Whitmore section, which is a tapered steel section used in connections. It shows a vertical section of a steel member with a 30-degree taper. The section is divided into a 'Critical Section' (the narrowest part) and a wider section. The taper is labeled with '30°' at both the top and bottom. The wider section is labeled 'Low Stress'. The critical section is labeled 'Critical Section'. The force applied to the section is labeled T_u or P_u . A box on the left lists the failure modes: 'Tensile Yielding', 'Tensile Rupture', and 'Compression'. The AISC logo is in the bottom left corner.

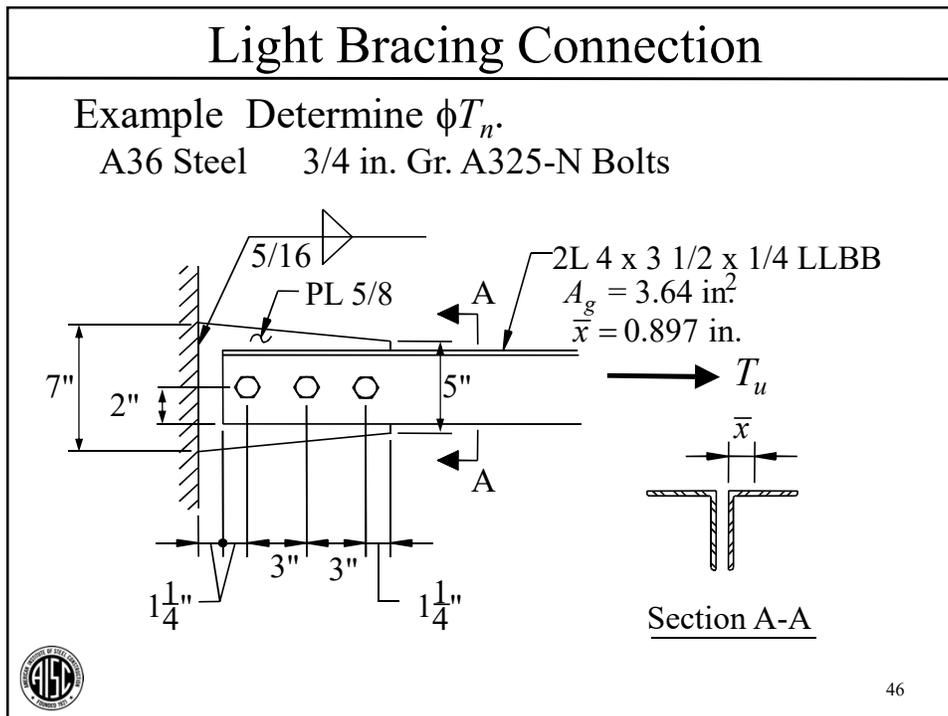
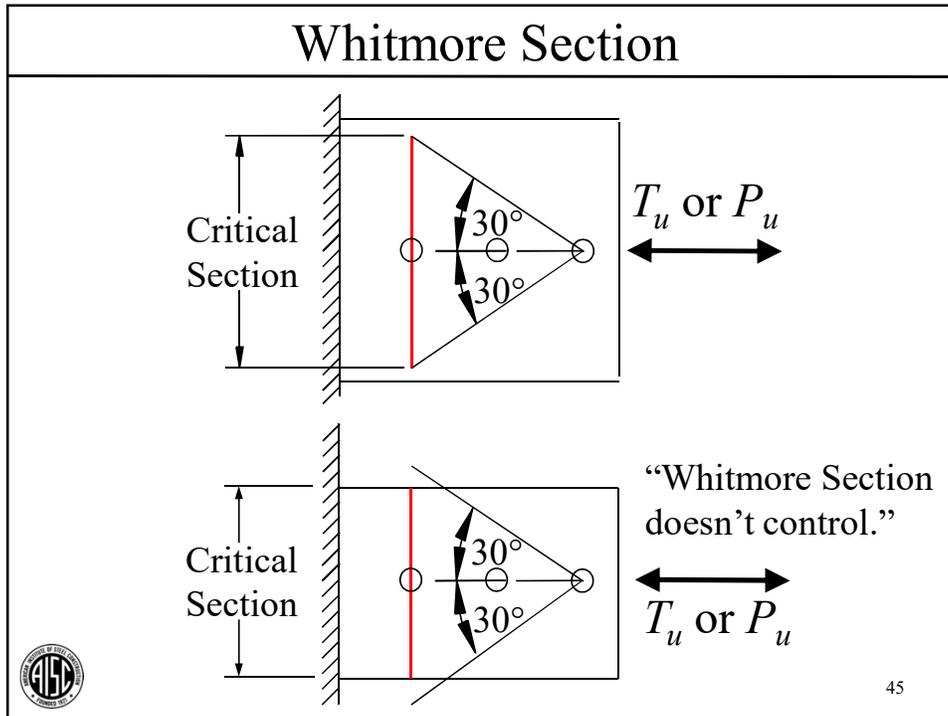
Whitmore Section on Page 9-4 in the *Manual*

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Whitmore Section

The left photograph shows a full Whitmore section with a red arrow pointing to the critical section. The right photograph is a close-up of the critical section, showing the edges of the section with red arrows pointing to the edges. The AISC logo is in the bottom left corner.

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Light Bracing Connection

Limit States

- 1-1 Angle Tensile Yielding
- 2-2 Angle Tensile Rupture
- 3-3 Angle Block Shear
- 4-4 Shear Transfer Between Angles and Plate**
- 5-5 Plate Tensile Rupture
- 6-6 Plate Tensile Yielding
- 7-7 Weld Rupture

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Light Bracing Connection

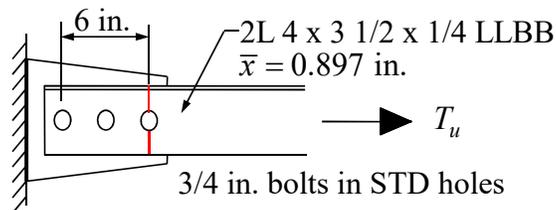
Angle Tensile Yielding

$\phi T_n = \phi F_y A_g$
 $= (0.9)(36)(3.64) = 118 \text{ kips}$

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Light Bracing Connection

Angle Tensile Rupture



$$\phi T_n = \phi F_u A_e = \phi F_u U A_n$$

$$A_n = A_g - A'_h = 3.64 - (1/4)(3/4 + 1/16 + 1/16)(2 \text{ angles}) = 3.20 \text{ in.}^2$$

$$U = 1 - \bar{x} / l = 1 - 0.897 / 6 = 0.850$$

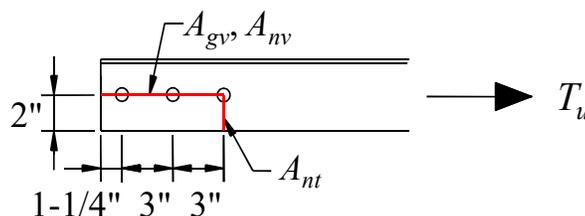


$$\phi T_n = (0.75)(58)(0.850)(3.20) = 118 \text{ kips}$$

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Light Bracing Connection

Angle Block Shear



$$A_{gv} = (7.25)(1/4)(2 \text{ angles}) = 3.63 \text{ in.}^2$$

$$A_{nv} = [7.25 - (2.5)(7/8)](1/4)(2 \text{ angles}) = 2.53 \text{ in.}^2$$

$$A_{nt} = [2.0 - (0.5)(7/8)](1/4)(2 \text{ angles}) = 0.781 \text{ in.}^2$$



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Light Bracing Connection

Angle Block Shear

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

$$= \min \left\{ \begin{array}{l} (0.6)(58)(2.53) = 88.0 \\ (0.6)(36)(3.63) = \underline{78.3} \end{array} \right. + (1.0)(58)(0.781)$$

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


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Light Bracing Connection

Shear Transfer

Specification Section
 J3.6 User Note

Effective Strength of each bolt is the minimum of:

- Angle Bearing
- Angle Tearout
- Bolt Shear Rupture
- Plate Bearing
- Plate Tearout



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Light Bracing Connection

Shear Transfer



Angle Bearing
 Use the bolt diameter, $d = 3/4$ in.
 $r_n = 52.2$ kips

Angle Tearout
 Use the nominal hole diameter, $d_h = 13/16$ in.
 Bolt A: $r_n = 29.4$ kips
 Bolts B and C: $r_n = 76.1$ kips



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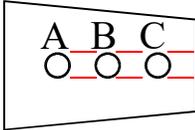
Light Bracing Connection

Shear Transfer

Bolt Shear Rupture
 3/4 in. A325-N Bolts in Double Shear
 Table 7-1: $\phi r_n = 35.8$ kips $\rightarrow r_n = 47.7$ kips

Plate Bearing
 $r_n = 65.3$ kips

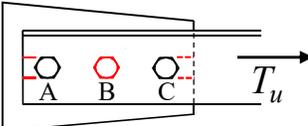
Plate Tearout
 Bolts A and B: $r_n = 95.1$ kips
 Bolt C: $r_n = 36.8$ kips




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Light Bracing Connection

Shear Transfer



$r_n = \min$

- Bolt Rupture
- Angle Bearing
- Angle Tearout
- Plate Bearing
- Plate Tearout

	47.7 kips	
	52.2 kips	
$r_{nA} = \min$	29.4 kips	$r_{nB} = \min$
	65.3 kips	76.1 kips
	95.1 kips	95.1 kips
		$r_{nC} = \min$
		76.1 kips
		65.3 kips
		36.8 kips

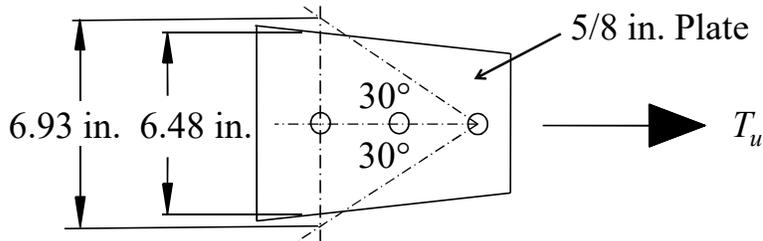


$\phi T_n = 0.75(29.4 + 47.7 + 36.8) = 85.4 \text{ kips}$

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Light Bracing Connection

Plate Rupture



$A_e = UA_n \quad U = 1.0 \text{ (Table D3.1 Case 1)}$

Whitmore Section doesn't control.

 $A_n = (6.48 - 7/8)(5/8)$
 $\quad = 3.50 \text{ in.}^2$
 $\phi T_n = \phi F_u A_e$
 $\quad = (0.75)(58)(1.0)(3.50) = 152 \text{ kips}$



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Light Bracing Connection

Plate Yielding

Whitmore Section doesn't control.

$$A_g = (6.48)(5/8) = 4.05 \text{ in.}^2$$

$$\phi T_n = \phi F_y A_g = (0.9)(36)(4.05) = 131 \text{ kips}$$


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Light Bracing Connection

Weld Rupture

Whitmore 30° results in > 7 in. at the weld, so use
 $L_{weld} = 7 \text{ in.}$

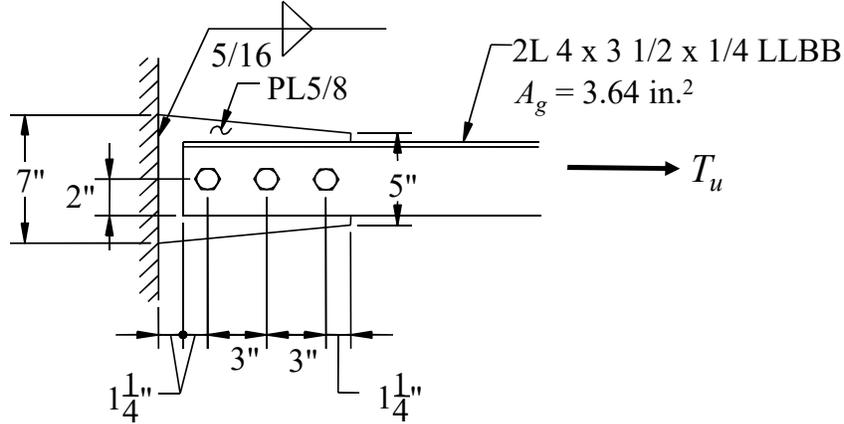
$$\phi T_n = 1.392(1.5)DL_{weld}$$

$$= 1.392(1.5)(5)(7 \text{ in.})(2 \text{ welds}) = 146 \text{ kips}$$


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Light Bracing Connection

Connection Design Strength



$$\phi T_n = 85.4 \text{ kips (Shear Transfer)}$$

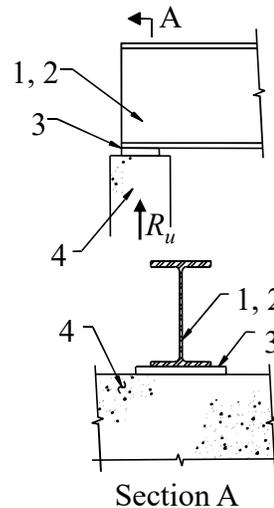


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BEAM BEARING PLATES

Limit States

1. Beam Web Local Yielding (*Specification J10.2*)
2. Beam Web Local Crippling (*Specification J10.3*)
3. Bearing Plate Bending (*Specification J4.5*)
4. Concrete Crushing (*Specification J8*)



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Beam Web Local Yielding

Critical Section

Manual Figure 14-1



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Beam Web Local Yielding

Specification J10.2

k_{des}

$$R_n = F_{yw} t_w (2.5k + l_b) \text{ if load } \leq d \text{ from end}$$

$$R_n = F_{yw} t_w (5.0k + l_b) \text{ if load } > d \text{ from end}$$

$$\phi = 1.0$$

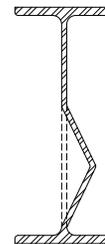

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Beam Web Local Crippling

Specification J10.3

$$\phi = 0.75$$

If load is applied $\geq d / 2$ from end:



$$R_n = 0.80t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f \quad (\text{Spec. J10-4})$$

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



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Beam Web Local Crippling

Specification J10.3

If load is applied $< d / 2$ from end:

If $l_b / d \leq 0.2$

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

If $l_b / d > 0.2$

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



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Concrete Crushing

Specification J8

$$\phi_c = 0.65$$

(a) On the full area of a concrete support:

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

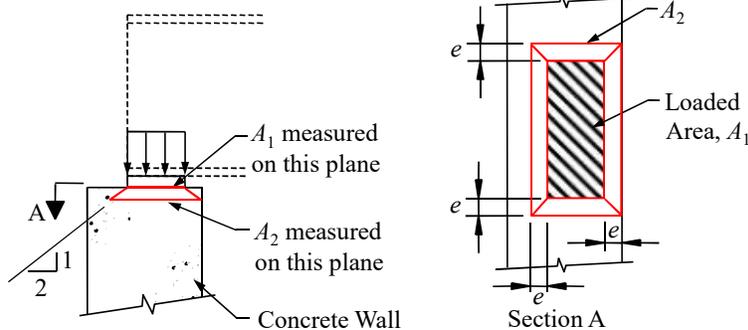
(b) Otherwise:

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



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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



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Beam Bearing: Plate Bending

$k = k_{des}$

$n = B/2 - k$

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Beam Bearing: Plate Bending

$n = B/2 - k_{des}$

$f_{pu} = R_u / A_1$

1" Plate Strip

Section A-A

$$M_u \leq \phi M_n \quad \phi = 0.9$$

$$M_u = w_u n^2 / 2 = f_{pu} (1 \text{ in.}) n^2 / 2$$

$$\phi M_n = \phi F_y Z_x = \phi F_y (1 \text{ in.}) t^2 / 4$$

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Beam Bearing Plate Bending

$$M_u \leq \phi M_n$$

$$M_u = (\phi M_n)_{min} \Rightarrow t_{min} = \sqrt{\frac{2 f_{pu} n^2}{\phi F_y}}$$

$$\phi = 0.90$$


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Beam Bearing Plate Example

Example

- Beam bearing on a concrete wall.
- Bearing plate centered on the wall.
- A36 Plate
- $f'_c = 3$ ksi
- Determine whether or not the 5 in. dimension is adequate.
- Determine the required plate width.
- Determine the required plate thickness.



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Beam Bearing Plate Example

Beam: W18x76 A992

$b_f = 11.0$ in.

$t_f = 0.680$ in.

$d = 18.2$ in.

$t_w = 0.425$ in.

$k_{des} = 1.08$ in.

$F_y = 50$ ksi



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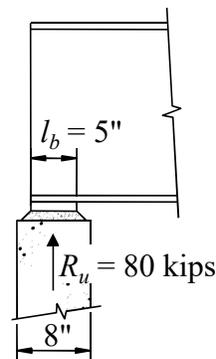
Beam Bearing Plate Example

Web Local Yielding

$F_{yw} = 50$ ksi

$k_{des} = 1.08$ in.

$t_w = 0.425$ in.



$$\begin{aligned} \phi R_n &= \phi F_{yw} t_w (2.5k + l_b) \\ &= (1.00)(50)(0.425)[(2.5)(1.08) + 5.0] \\ &= 164 \text{ kips} > R_u = 80 \text{ kips, OK} \end{aligned}$$



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Beam Bearing Plate Example

Web Local Crippling

At ≤ d/2:

$$l_b/d = 5.0 / 18.2 = 0.270 > 0.2$$

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

$$= (0.75)(0.4)(0.425^2) \left(1 + [4(0.270) - 0.2] \left(\frac{0.425}{0.680} \right)^{1.5} \right) \sqrt{\frac{29000(50)(0.680)}{0.425}}$$

$$= 118 \text{ kips} > R_u = 80 \text{ kips, OK}$$

5 in. bearing length is adequate

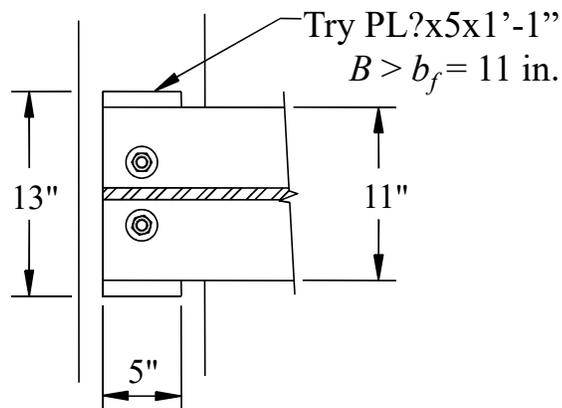


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Beam Bearing Plate Example

Plate Width

Trial based on flange width.



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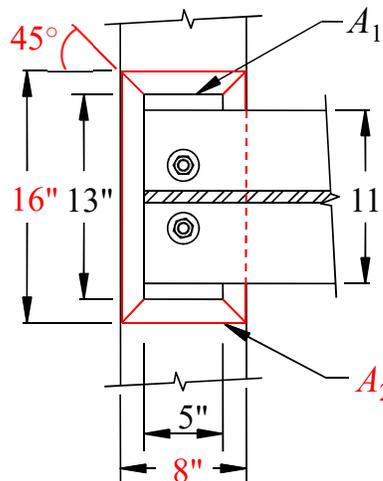
Beam Bearing Plate Example

Concrete Crushing

$$A_1 = (5)(13) = 65.0 \text{ in.}^2$$

$$A_2 = (8)(16) = 128 \text{ in.}^2$$

$$\begin{aligned} \phi P_p &= \phi 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \\ &= (0.65)(0.85)(3)(65) \sqrt{\frac{128}{65}} \\ &= 151 \text{ kips} > 80 \text{ kips, OK} \end{aligned}$$



Use 5 in. x 13 in. plan dimensions.

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Beam Bearing Plate Example

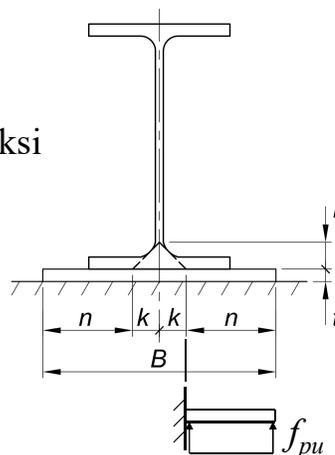
Plate Bending

$$F_y = 36 \text{ ksi}$$

$$f_{pu} = R_u / A_1 = 80 / [(5)(13)] = 1.23 \text{ ksi}$$

$$\begin{aligned} n &= B/2 - k_{des} = 13/2 - 1.08 \\ &= 5.42 \text{ in.} \end{aligned}$$

$$\begin{aligned} t_{min} &= \sqrt{\frac{2 f_{pu} n^2}{\phi F_y}} \\ &= \sqrt{\frac{2(1.23)(5.42^2)}{(0.9)(36)}} \\ &= 1.49 \text{ in.} \end{aligned}$$



Use PL1-1/2 x 5 x 1'-1" A36

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Column Base Plate Design

Concrete Crushing

Same as for beam bearing plates.



77

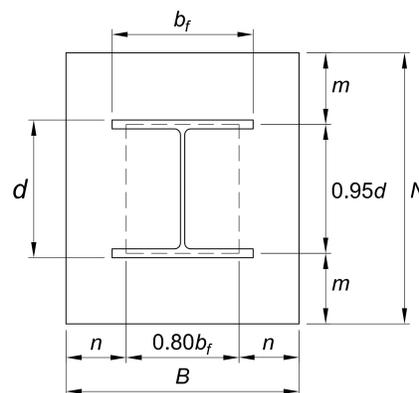
Column Base Plate Design

Required Base Plate Thickness

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

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

$$t_{min} = \sqrt{\frac{2f_{pu}(m')^2}{\phi F_y}}$$



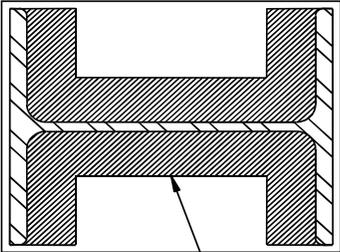
What is t_{min} if m and n are very small?



78

Column Base Plate Design

Lightly Loaded Base Plates



Bearing Area


79

Column Base Plate Design

Lightly Loaded Base Plates

For lightly loaded base plates, replace m' with

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

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1.0 \qquad n' = \frac{1}{4}\sqrt{db_f}$$

and

$$X = \left(\frac{4db_f}{(d + b_f)^2} \right) \frac{P_u}{\phi P_p} \qquad t_{min} = \sqrt{\frac{2f_{pu}l^2}{\phi F_y}}$$


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Column Base Plate Design Example

Example: Evaluate the base plate.

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

$P_u = 250$ kips

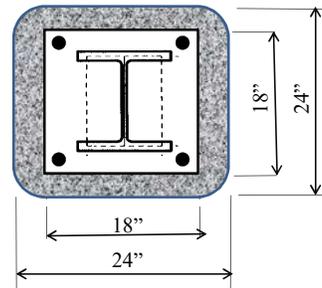
Base Plate: PL 1-1/2 x 18 x 1'-6" A36

Concrete Pedestal:

24 in. by 24 in.

$f'_c = 3$ ksi

Plate centered on the pedestal.



81

Column Base Plate Design Example

Concrete Crushing

$$A_1 = (18)(18) = 324 \text{ in.}^2$$

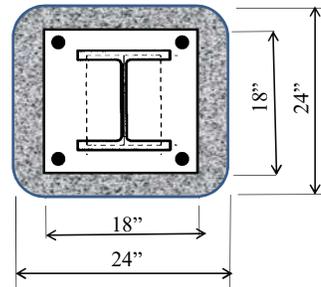
$$A_2 = (24)(24) = 576 \text{ in.}^2$$

$$\phi P_p = \phi 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \leq \phi 1.7 f'_c A_1$$

$$= (0.65)(0.85)(3)(324) \sqrt{\frac{576}{324}} \leq (0.65)(1.7)(\cancel{0.85})(3)(324)$$

$$= 716 \text{ kips} < \cancel{913 \text{ kips}} \ 1074 \text{ kips}$$

$$= 716 \text{ kips} > P_u = 250 \text{ kips, OK}$$



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Column Base Plate Design Example

Plate Bending

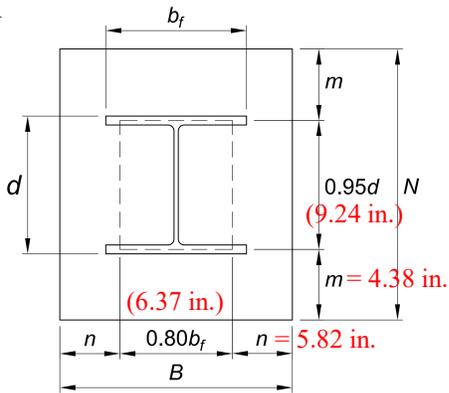
$$m = 4.38 \text{ in. and } n = 5.82 \text{ in.}$$

$$\begin{aligned} n' &= \sqrt{db_f} / 4 \\ &= \sqrt{(9.73)(7.96)} / 4 \\ &= 2.20 \text{ in.} \end{aligned}$$

$$\begin{aligned} X &= \frac{4db_f}{(d + b_f)^2} \frac{P_u}{\phi P_p} \\ &= \frac{4(9.73)(7.96)}{(9.73 + 7.96)^2} \frac{250}{716} \end{aligned}$$



$$= 0.346$$



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Column Base Plate Design Example

Plate Bending

$$\begin{aligned} \lambda &= \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} = \frac{2\sqrt{0.346}}{1 + \sqrt{1 - 0.346}} \\ &= 0.650 \leq 1.0 \end{aligned}$$

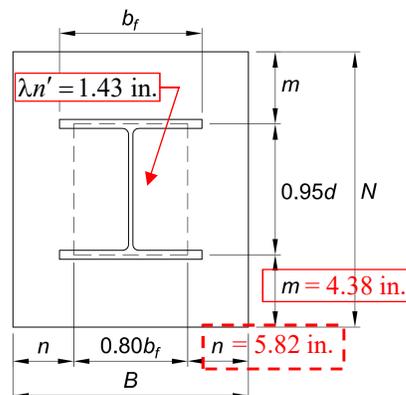
$$\lambda n' = (0.650)(2.20) = 1.43 \text{ in.}$$

$$l = \max \begin{cases} m = 4.38 \text{ in.} \\ n = 5.82 \text{ in.} \leftarrow \text{controls} \\ \lambda n' = 1.43 \text{ in.} \end{cases}$$

$$f_{pu} = \frac{P_u}{BN} = \frac{250}{(18)(18)}$$



$$= 0.772 \text{ ksi}$$



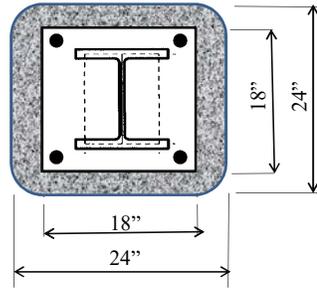
84

Column Base Plate Design Example

Plate Bending

$$t_{min} = \sqrt{\frac{2 f_{pu} l^2}{\phi F_y}} = \sqrt{\frac{(2)(0.772)(5.82^2)}{(0.9)(36)}}$$
$$= 1.27 \text{ in.} < 1.5 \text{ in., OK}$$

PL 1-1/2 x 18 x 1'-6" A36
is adequate.



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End of Session 2

Thank You for
Attending

Next Up



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Next Session

- November 6, 2019 Shear Connections Part I

TOPICS

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



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AISC | Questions?



Single-Session Registrants

CEU / PDH Certificates

- 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!



Single-Session Registrants

CEU / PDH Certificates

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



4-Session Registrants

CEU / PDH Certificates

One certificate will be issued at the conclusion of the course.



4-Session Registrants

Attendance and PDH Certificates

- You have two options to receive credit for a given session.
 - Option 1: Watch the live session. Credit for live attendance will be displayed on the Course Resources table within two days of the session.
 - Option 2: Watch the recording and pass the associated quiz.

Videos and Quizzes

- For each session, find access by the end of the day, Friday, after the live air date. (An email will be sent from webinars@aisc.org.)
- Each video recording and quiz will be available for four weeks.
- Quiz scores are displayed in the Course Resources table.

Distribution of Certificates

All certificates will be issued after the course is completed (the week of December 16). Only the registrant will receive a certificate for the course.



4-Session Registrants

Course Resources

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



4-Session Registrants

Course Resources

Go to www.aisc.org and sign in.



4-Session Registrants

Course Resources

Go to www.aisc.org and sign in.

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

4-Session Registrants

Course Resources

The screenshot shows the AISC website's 'Course Resources' page. At the top, there is a navigation menu with links for EDUCATION, PUBLICATIONS, AWARDS AND COMPETITIONS, TECHNICAL RESOURCES, and STEEL SOLUTIONS CENTER. Below the menu is a large image of a modern building with a glass facade and palm trees, with the AISC logo overlaid. Below the image, the breadcrumb trail reads 'AISC > MY ACCOUNT > COURSE RESOURCES'. The main content area is titled 'Course Resources' and contains a table with two columns: 'Event' and 'Start Date'.

Event	Start Date
Seismic Design in Steel	
4-Session Package-Design of Facade Attachments	5/9/2019 1:30:00 PM
Live Webinar - 4-Session Package-Fundamentals of Connection Design	10/23/2019 1:30:00 PM
NS-15 8-Session Package-Night School 15 - Fundamentals of Connection Design	10/3/2017 7:00:00 PM
NS-16 8-Session Package-Night School 16 - Seismic Design in Steel	2/5/2018 7:00:00 PM
NS-17 4-Session Package-Night School 17: Design of Facade Attachments	7/16/2018 7:00:00 PM
NS-18 8-Session Package-Night School 18: Steel Construction Mill To Topping Out	10/15/2018 7:00:00 PM
NS-19 8-Session Package-Night School 19: Connection Design	2/4/2019 7:00:00 PM
NS-20 8-Session Package-Night School 20: Classical Methods of Structural Analysis	6/3/2019 7:00:00 PM
NS-21 8-Session Package-Night School 21: Welded Connections - A Primer for Engineers	10/8/2019 7:00:00 PM



4-Session Registrants

Course Resources



[EDUCATION](#) | [PUBLICATIONS](#) | [AWARDS AND COMPETITIONS](#) | [TECHNICAL RESOURCES](#) | [STEEL SOLUTIONS CENTER](#)



[AISC](#) > [MY ACCOUNT](#) > [COURSE RESOURCES](#) > [FUNDAMENTALS OF CONNECTION DESIGN PACKAGE RESOURCES](#)

Fundamentals of Connection Design

4-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
Fundamental Concepts, Part 1	Oct 23 2019 1:30PM EDT	Handouts	Video Passcode: BD4C1	Pass Score: 100	No
Fundamental Concepts, Part 2	Oct 30 2019 1:30PM EDT	Handouts	Available 11/01/2019 5:00PM EDT	Available 11/01/2019 5:00PM EDT	Pending
Shear Connections, Part 1	Nov 6 2019 1:30PM EST	Handouts	Available 11/08/2019 5:00PM EST	Available 11/08/2019 5:00PM EST	Pending
Shear Connections, Part 2	Nov 13 2019 1:30PM EST	Handouts	Available 11/15/2019 5:00PM EST	Available 11/15/2019 5:00PM EST	Pending



AISC | Thank you.



Smarter.
Stronger.
Steel.

