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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.

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Fundamentals of Connection Design

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

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Owner, Davis Structural Engineering

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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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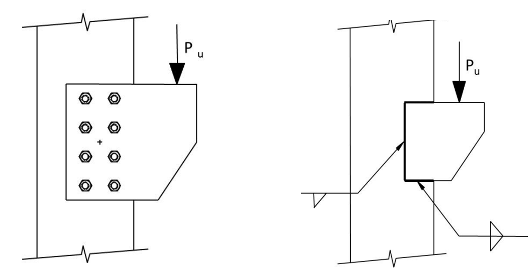
ECCENTRICALLY LOADED BOLTED AND WELDED CONNECTIONS



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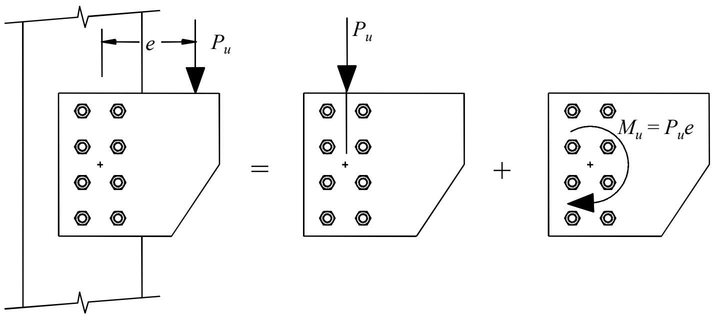
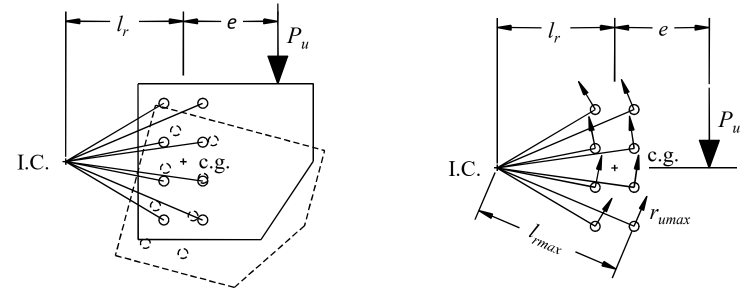
Bolts: Eccentric Connections



Elastic Method

Bolts: Eccentric Connections



Instantaneous Center of Rotation Method

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

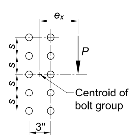
$$R_n = C r_n$$

or

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

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
3	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

$\phi P_n = C\,\phi r_{nv}$
$\phi = 0.75$

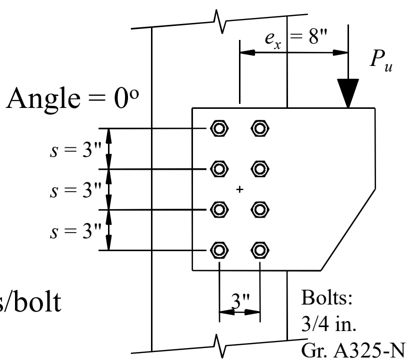
Example: Eccentric Bolted Connection

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 kips/bolt



ϕP_n = (2.93)(17.9) = 52.4 kips

Example: Eccentric Bolted Connection

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

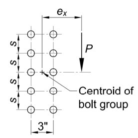
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$R_n = Cr_n$

or

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

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
3	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

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

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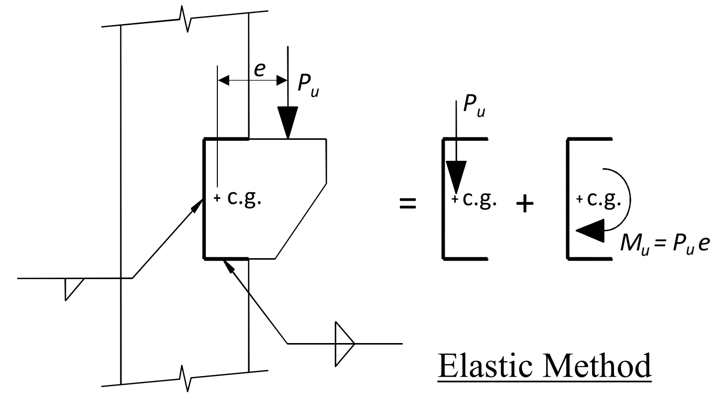
Welds: Eccentric Connections



Elastic Method

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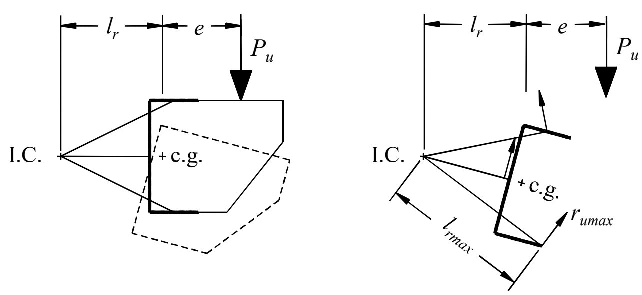
Welds: Eccentric Connections



Instantaneous Center of Rotation Method

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Welds: Eccentric Connections

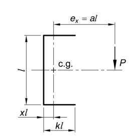
Table 8-8
Coefficients, C,
for Eccentrically Loaded Weld Groups
Angle = 0°

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		
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$	$D_{min} = \dfrac{P_u}{\phi CC_1 l}$	$l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$	$D_{min} = \dfrac{\Omega P_a}{CC_1 l}$	$l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, 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	5.86	6.36	7.37	8.39	9.42	10.5	11.5
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

$R_n = CC_1Dl$

$\phi = 0.75$

$k \rightarrow x$
$x \rightarrow a$
$k \;\&\; a \rightarrow C$

C_1 from Table 8-3.
$C_1 = 1.00$ for 70 ksi

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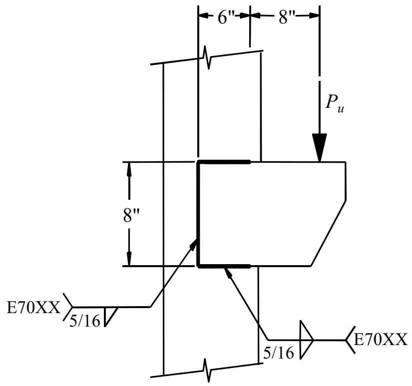
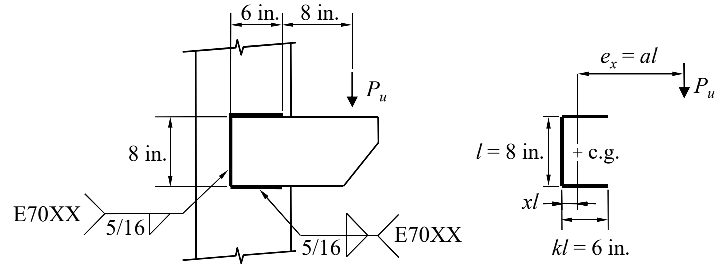
Example: Determine ϕP_n



Example: Determine ϕP_n



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

Manual Table 8-8: $x = 0.225$

xl = (0.225)(8 in.) = 1.8 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

$k = 0.75 \rightarrow x = 0.225$

Example: Determine ϕP_n

$e_x = al$

6 in. + 8 in. – 1.8 in. = (a)(8 in.)

$a = 1.53$

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

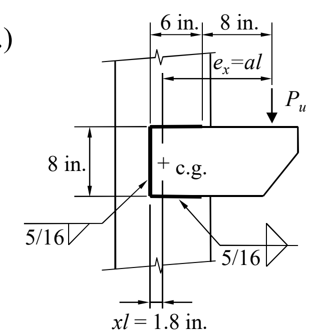
5/16 in. welds $\rightarrow$ $D = 5$

E70XX $\rightarrow$ $C_1 = 1.00$

$\phi P_n = \phi C C_1 D l$

= (0.75)(1.59)(1.00)(5)(8.0 in.)

= 47.7 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

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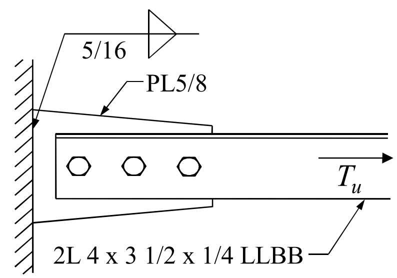
DIRECTLY LOADED CONNECTIONS



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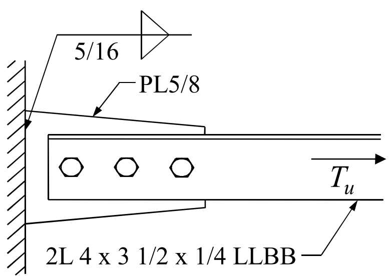
Applicable Limit States

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



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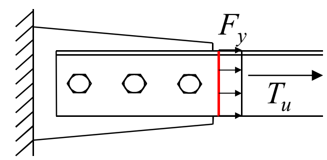
Limit State: Tensile Yielding

Specification D2 and J4.1



$\phi T_n = \phi F_y A_g$ (*Spec*. D2-1, J4-1)
$\phi = 0.9$

Note: Tensile yielding is a member limit state.

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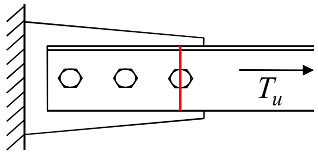
Limit State: Tensile Rupture

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 = UA_n

U = shear lag factor

A_n = net area

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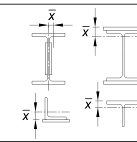
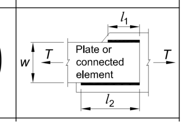
Limit State: Tensile Rupture

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

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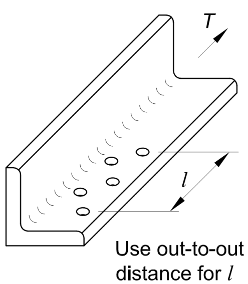
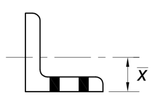
Limit State: Tensile Rupture

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





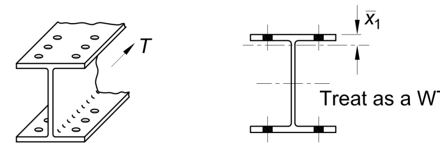
$$\boxed{U = 1 - \frac{\bar{x}}{l}}$$

Commentary Figure C-D3.2

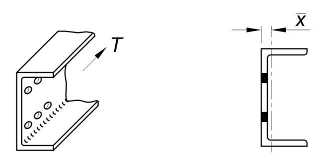
31

Limit State: Tensile Rupture

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



(a)



(b)

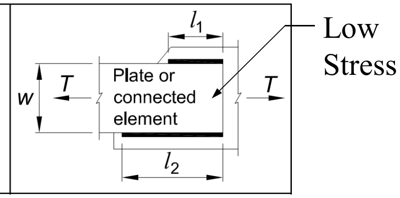
Commentary Figure C-D3.1

32

Limit State: Tensile Rupture

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

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



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

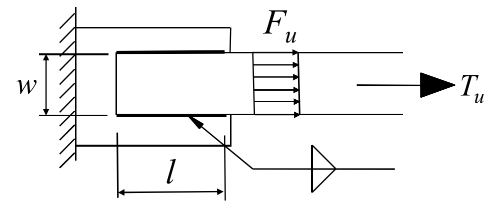
Note: l_1 and $l_2 \geq 4$ times fillet weld size.

33

Limit State: Tensile Rupture

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

Plate with equal length welds.

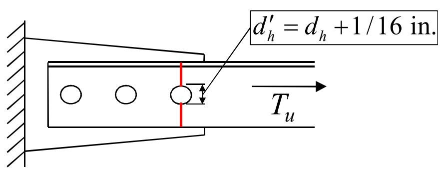


$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right) \Rightarrow U = \frac{3l^2}{3l^2 + w^2}$$

34

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

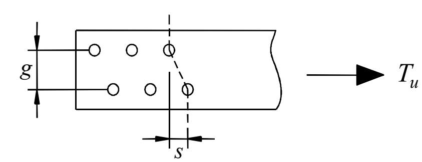
$= (t)(d_h + 1/16 \text{ in.})$ per Spec. B4.3b

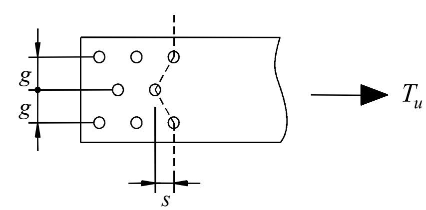
Add "Stagger" term(s) if applicable.

35

Limit State: Tensile Rupture

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





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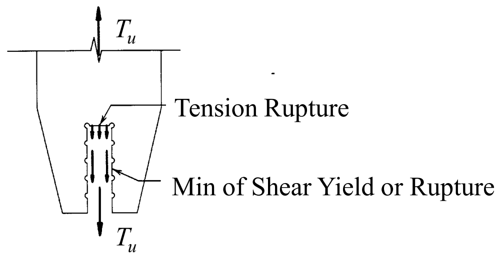
Limit State: Block Shear Strength

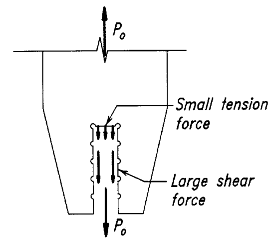
Specification J4.3

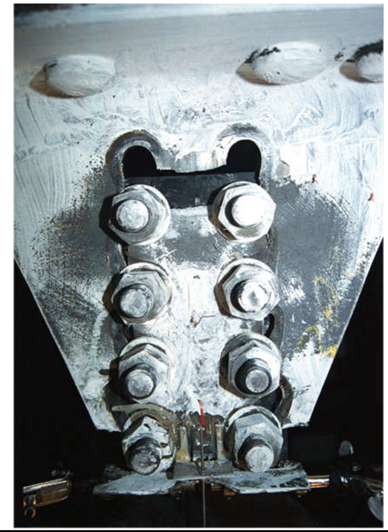


- Failure occurs when shear forces reach the smaller of shear yield and shear rupture.
- The tension area is at rupture when failure occurs.

37

Limit State: Block Shear





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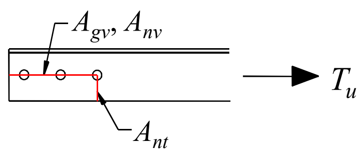
Limit State: Block Shear Strength

Specification J4.3



$\phi = 0.75$

$$R_n = [0.6F_uA_{nv} + U_{bs}F_uA_{nt}] \leq [0.6F_yA_{gv} + U_{bs}F_uA_{nt}] \quad \text{(J4-5)}$$

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

$$= \min \begin{vmatrix} 0.6F_uA_{nv} \\ 0.6F_yA_{gv} \end{vmatrix} + U_{bs}F_uA_{nt}$$

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Limit State: Block Shear Strength

Example

$A_{nv} = 2.53$ in.^{2} $A_{gv} = 3.63$ in.^{2}
$A_{nt} = 0.781$ in.^{2} $U_{bs} = 1.0$
A36 Steel: $F_y = 36$ ksi $F_u = 58$ ksi

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

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

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

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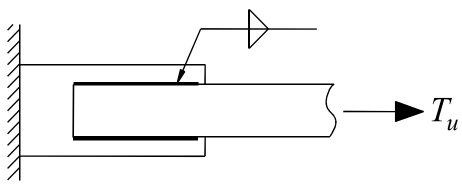
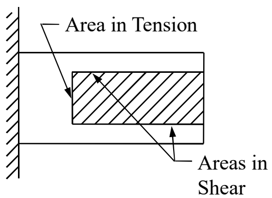
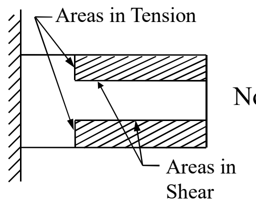
Limit State: Block Shear Strength

Welded Connections





Case I



Case II

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

Limit State: Plate Compression

Specification Section J4.4

$P_u \le \phi P_n \qquad \phi = 0.90$

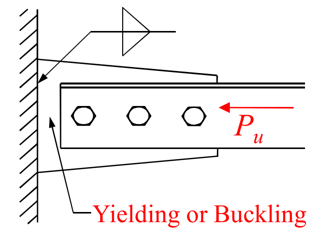
If $L_c/r \le 25$, then

$\phi P_n = \phi F_y A_g$

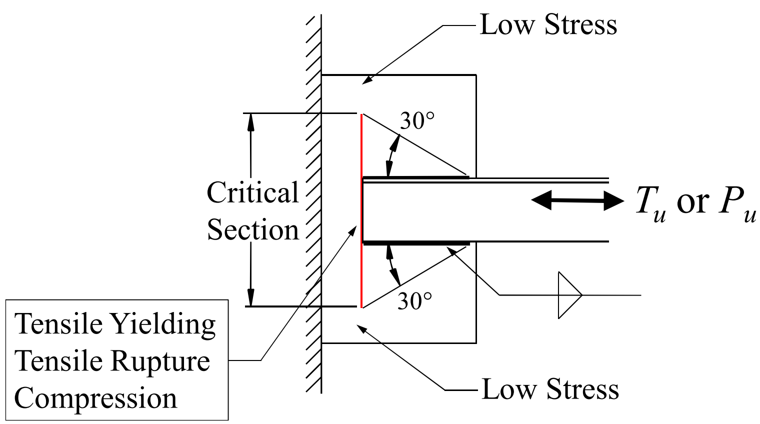
Otherwise

ϕP_n per Chapter E

$L_c = KL$ → See Design Guide 29

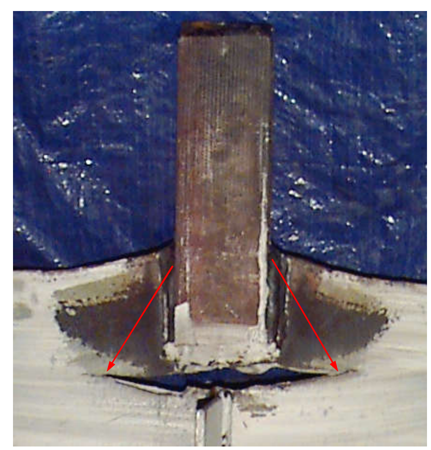


Whitmore Section

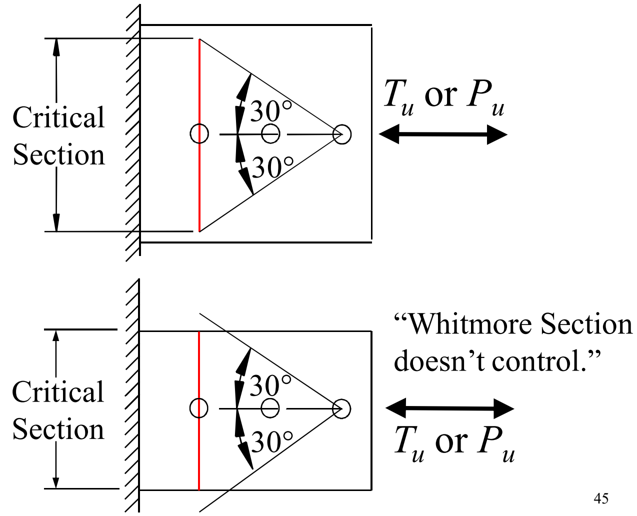


Whitmore Section on Page 9-4 in the *Manual*

Whitmore Section



Whitmore Section



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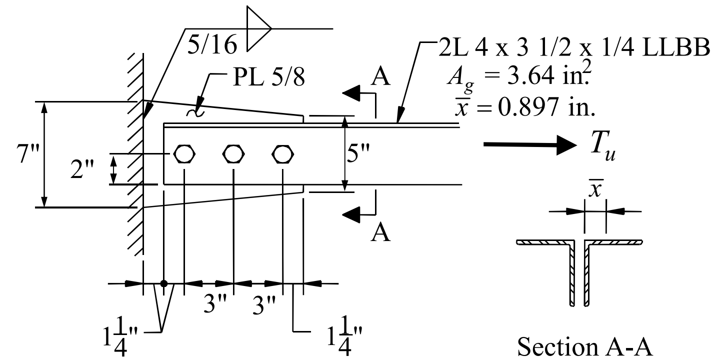
Light Bracing Connection

Example Determine ϕT_n.

A36 Steel 3/4 in. Gr. A325-N Bolts



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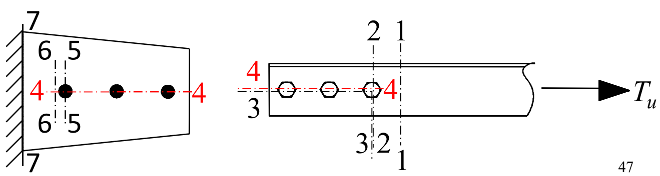
Light Bracing Connection

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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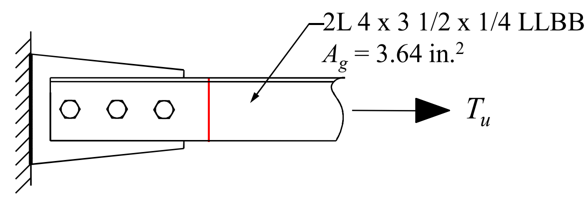
Light Bracing Connection

Angle Tensile Yielding



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

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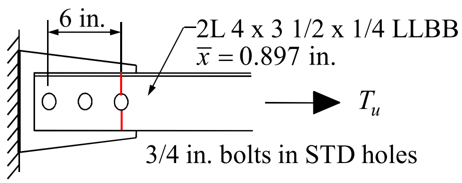
Light Bracing Connection

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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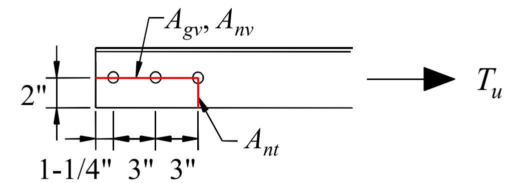
Light Bracing Connection

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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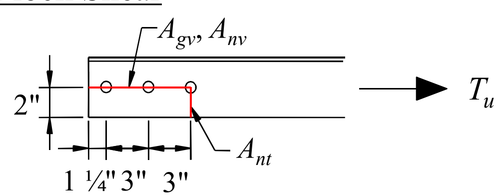
Light Bracing Connection

Angle Block Shear



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

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

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

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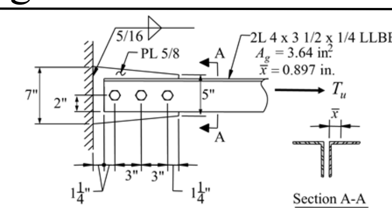
Light Bracing Connection

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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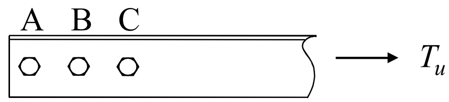
Light Bracing Connection

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

Light Bracing Connection

Shear Transfer

Bolt Shear Rupture

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

Table 7-1: ϕr_n = 35.8 kips → r_n = 47.7 kips

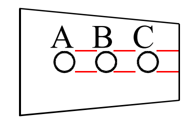
Plate Bearing

r_n = 65.3 kips

Plate Tearout

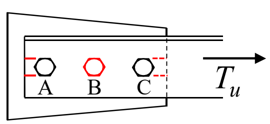
Bolts A and B: r_n = 95.1 kips

Bolt C: r_n = 36.8 kips



Light Bracing Connection

Shear Transfer



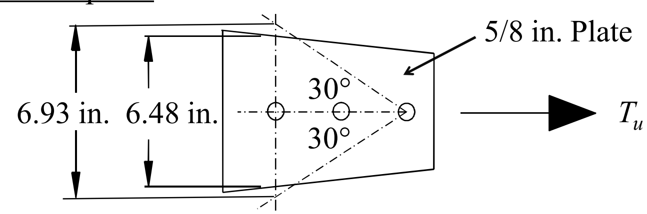
$$r_n = \min \begin{cases} \text{Bolt Rupture} \\ \text{Angle Bearing} \\ \text{Angle Tearout} \\ \text{Plate Bearing} \\ \text{Plate Tearout} \end{cases}$$

$$r_{nA} = \min \begin{cases} 47.7 \text{ kips} \\ 52.2 \text{ kips} \\ \boxed{29.4 \text{ kips}} \\ 65.3 \text{ kips} \\ 95.1 \text{ kips} \end{cases} \quad r_{nB} = \min \begin{cases} \boxed{47.7 \text{ kips}} \\ 52.2 \text{ kips} \\ 76.1 \text{ kips} \\ 65.3 \text{ kips} \\ 95.1 \text{ kips} \end{cases} \quad r_{nC} = \min \begin{cases} 47.7 \text{ kips} \\ 52.2 \text{ kips} \\ 76.1 \text{ kips} \\ 65.3 \text{ kips} \\ \boxed{36.8 \text{ kips}} \end{cases}$$

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

Light Bracing Connection

Plate Rupture



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

Whitmore Section doesn't control.

$A_n = (6.48 - 7/8)(5/8)$

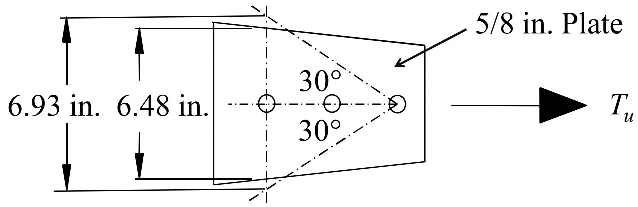
$= 3.50 \text{ in.}^2$

$\phi T_n = \phi F_u A_e$

$= (0.75)(58)(1.0)(3.50) = 152$ kips

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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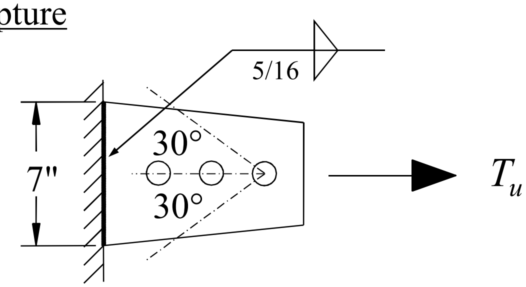
Light Bracing Connection

Weld Rupture



Whitmore 30° results in > 7 in. at the weld, so use $L_{weld} = 7$ 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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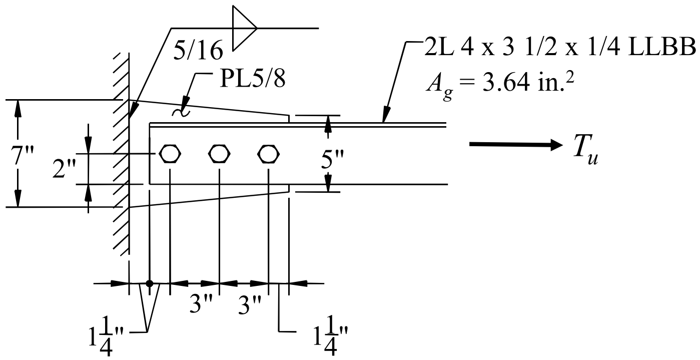
Light Bracing Connection

Connection Design Strength



$\phi T_n = 85.4$ kips (Shear Transfer)

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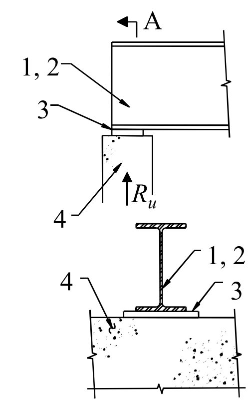
BEAM BEARING PLATES

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)



Section A

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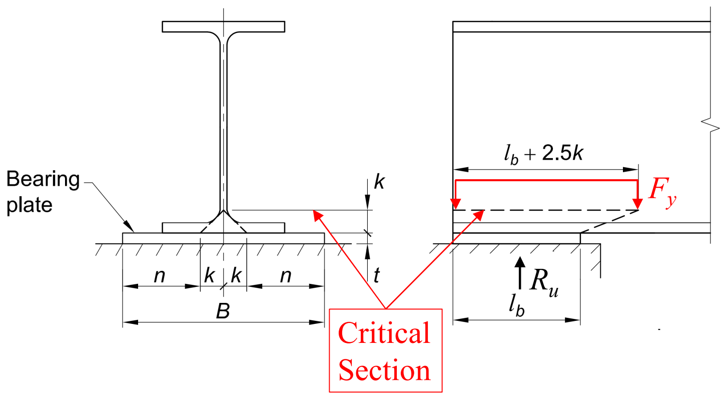
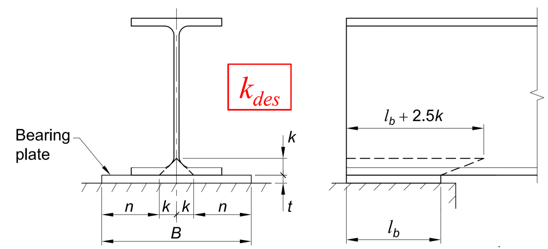
Beam Web Local Yielding



Manual Figure 14-1

Beam Web Local Yielding

Specification J10.2



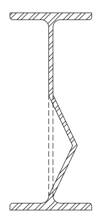
$R_n = F_{yw}t_w(2.5k + l_b)$ if load $\leq d$ from end

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

$\phi = 1.0$

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.

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 $l_b/d > 0.2$

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

Concrete Crushing

Specification J8

$\phi_c = 0.65$

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

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

(b) Otherwise:

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

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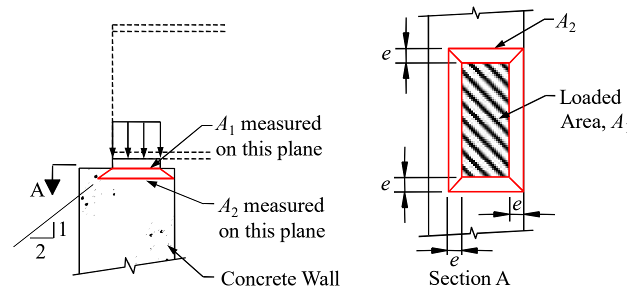
Beam Bearing: Concrete Crushing



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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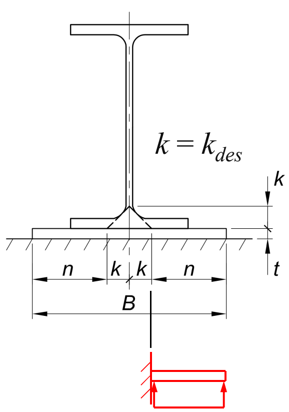
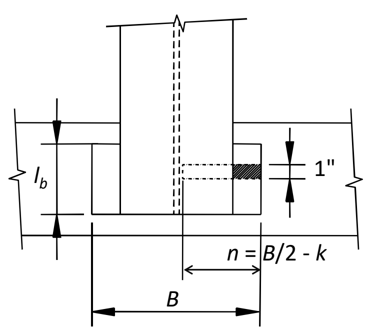
Beam Bearing: Plate Bending





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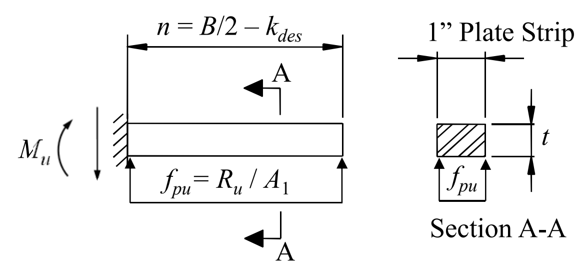
Beam Bearing: Plate Bending



$M_u \le \phi M_n \qquad \phi = 0.9$

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

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

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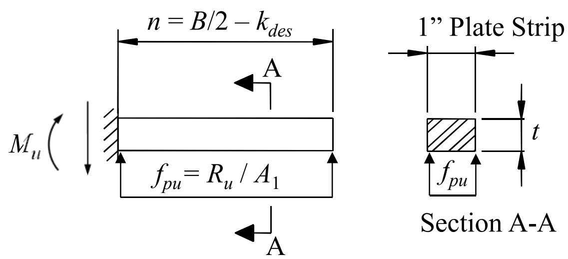
Beam Bearing Plate Bending



$$M_u \le \phi M_n$$

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

$$\phi = 0.90$$

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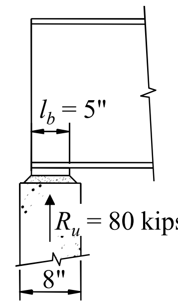
Beam Bearing Plate Example

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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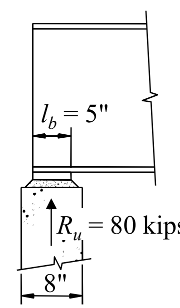
Beam Bearing Plate Example

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 $\le d/2$:

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

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

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

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

5 in. bearing length is adequate

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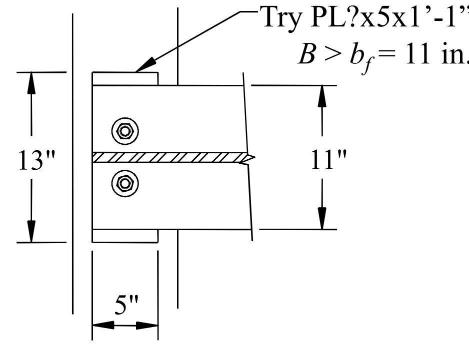
Beam Bearing Plate Example

Plate Width

Trial based on flange width.



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Beam Bearing Plate Example

Concrete Crushing

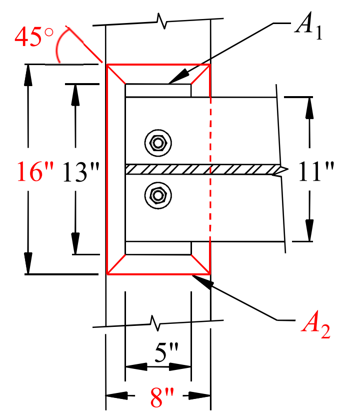
$A_1 = (5)(13) = 65.0 \text{ in.}^2$

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

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

$$= (0.65)(0.85)(3)(65)\sqrt{\frac{128}{65}}$$

$$= 151 \text{ kips} > 80 \text{ kips, OK}$$



Use 5 in. x 13 in. plan dimensions.

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Beam Bearing Plate Example

Plate Bending

$F_y = 36$ ksi

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

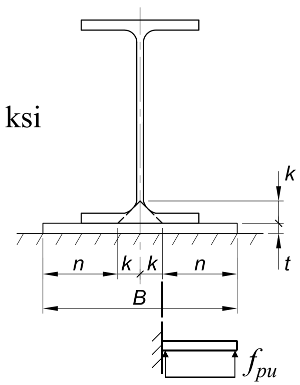
$n = B/2 - k_{des} = 13/2 - 1.08$

$= 5.42$ in.

$$t_{min} = \sqrt{\frac{2 f_{pu} n^2}{\phi F_y}}$$

$$= \sqrt{\frac{2(1.23)(5.42^2)}{(0.9)(36)}}$$

$$= 1.49 \text{ in.}$$



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.

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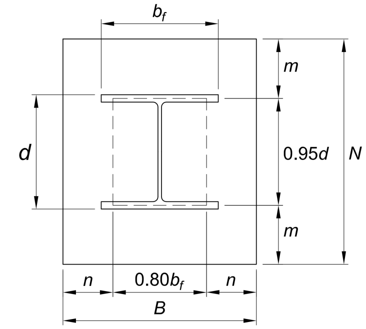
Column Base Plate Design

Required Base Plate Thickness

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

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

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



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

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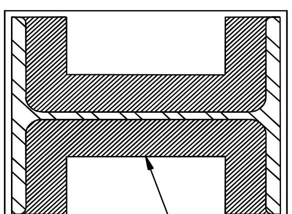
Column Base Plate Design

Lightly Loaded Base Plates



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Column Base Plate Design

Lightly Loaded Base Plates

For lightly loaded base plates, replace m' with

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

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

and

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

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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

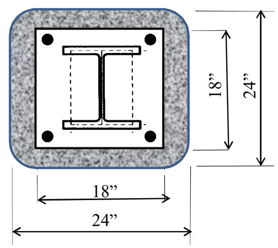
Base Plate: PL 1-1/2 x 18 x 1'-6" A36

Concrete Pedestal:

24 in. by 24 in.

$f'_c = 3$ ksi

Plate centered on the pedestal.



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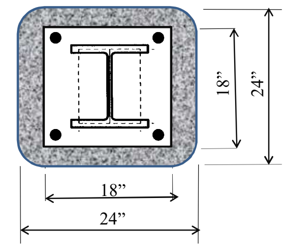
Column Base Plate Design Example

Concrete Crushing

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

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



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

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

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

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

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Column Base Plate Design Example

Plate Bending

$m = 4.38$ in. and $n = 5.82$ in

$$n' = \sqrt{d b_f} / 4$$

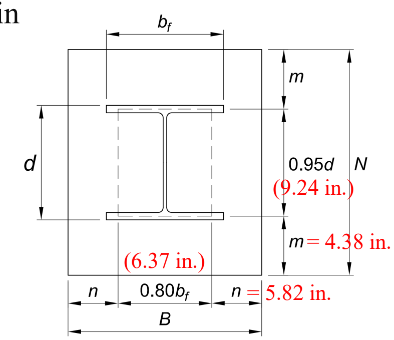
$$= \sqrt{(9.73)(7.96)} / 4$$

$$= 2.20 \text{ in.}$$

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

$$= \frac{4(9.73)(7.96)}{(9.73 + 7.96)^2} \frac{250}{716}$$

$$= 0.346$$



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Column Base Plate Design Example

Plate Bending

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} = \frac{2\sqrt{0.346}}{1 + \sqrt{1 - 0.346}}$$

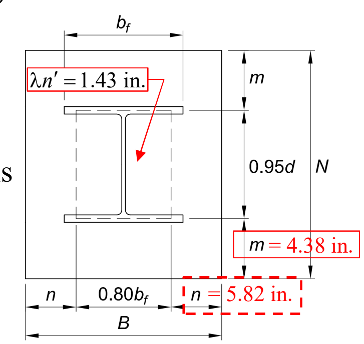
$$= 0.650 \le 1.0$$

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

$$l = \max \left| \begin{array}{l} m = 4.38 \text{ in.} \\ n = 5.82 \text{ in.} \Leftarrow \text{ controls} \\ \lambda n' = 1.43 \text{ in.} \end{array} \right.$$

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

$$= 0.772 \text{ ksi}$$



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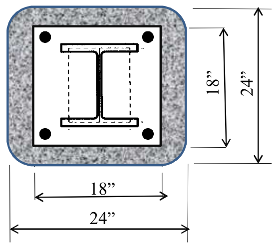
Column Base Plate Design Example

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?

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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!

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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.

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4-Session Registrants

CEU / PDH Certificates

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

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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.

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4-Session Registrants

Course Resources

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

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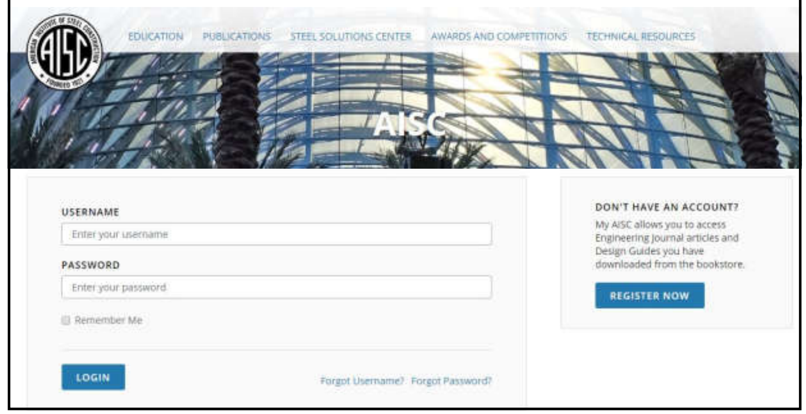
4-Session Registrants

Course Resources

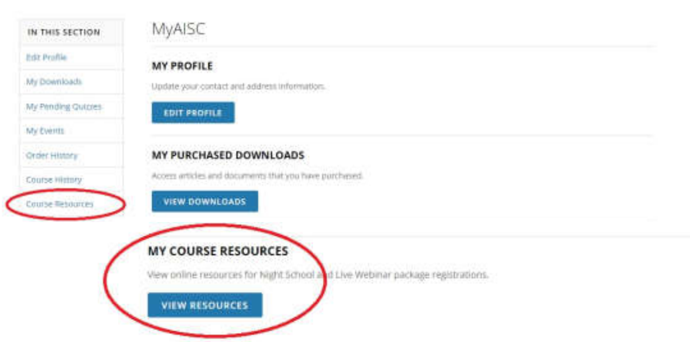
Go to www.aisc.org and sign in.



4-Session Registrants

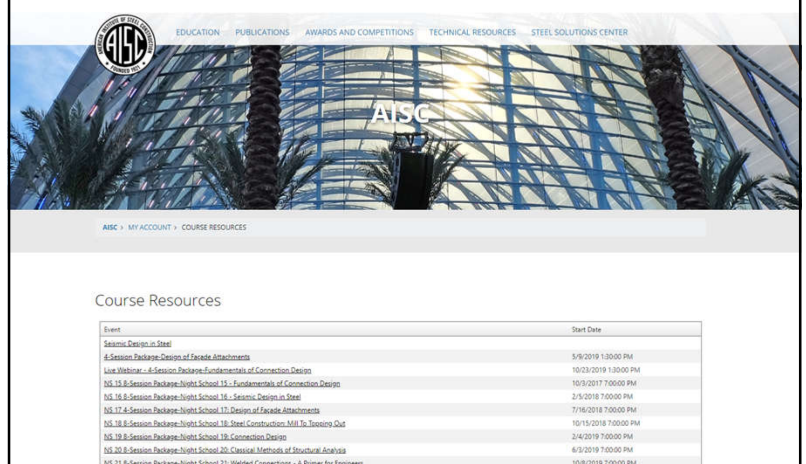
Course Resources

Go to www.aisc.org and sign in.



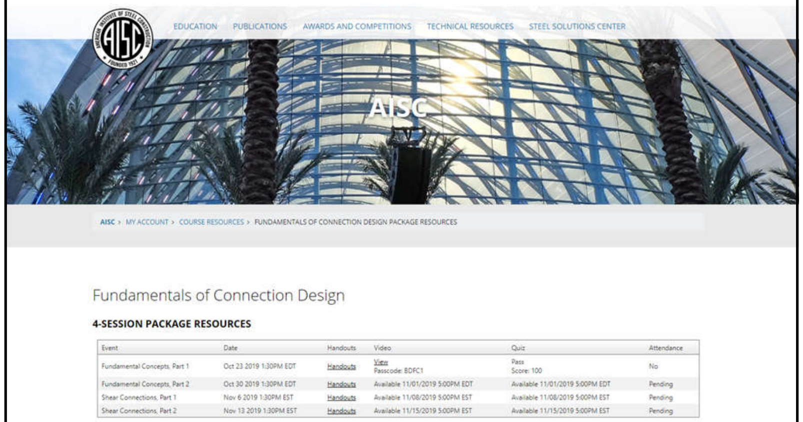
4-Session Registrants

Course Resources



4-Session Registrants

Course 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	View Passcode: BDFC1	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.

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