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Fundamentals of Connection Design

Session 3: Shear Connections, Part I

November 6, 2019



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Course Description

Shear Connections, Part I
November 6, 2019

This session will provide an overview of a variety of shear connection types, addressing the advantages and disadvantages of each. The limit states for block shear and flexural strength in coped beams will be presented. Shear end-plate and double-angle connection designs will also be discussed. Design examples will be presented to demonstrate the concepts.



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Learning Objectives

- Identify several types of shear connections.
- Explain where the point of rotation is modeled for various shear connections.
- List the steps in designing shear end-plate connections.
- List the steps in designing double-angle connections.



Fundamentals of Connection Design

Session 3: Shear Connections, Part I
November 6, 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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SHEAR CONNECTIONS PART I



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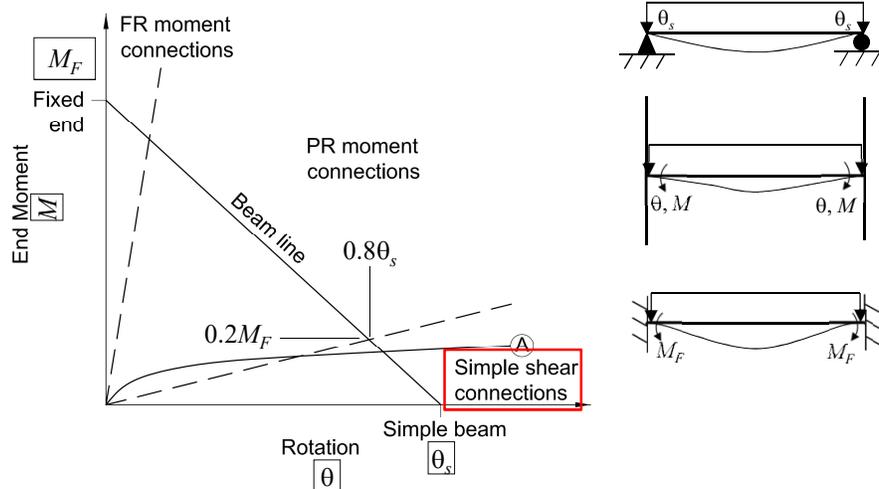
Topics

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



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Connection Classification



Manual Figure 10-1



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Types of Shear Connections

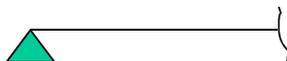
- Shear End-Plate
- Double-Angle
- Single-Angle
- Single-Plate or Shear Tab
- Tee Shear Connections
- Unstiffened Seated Connections
- Stiffened Seated Connections



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Design Considerations

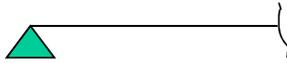
- Shear connection design assumes the connection is pinned.
- Where is the pin?



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Design Considerations

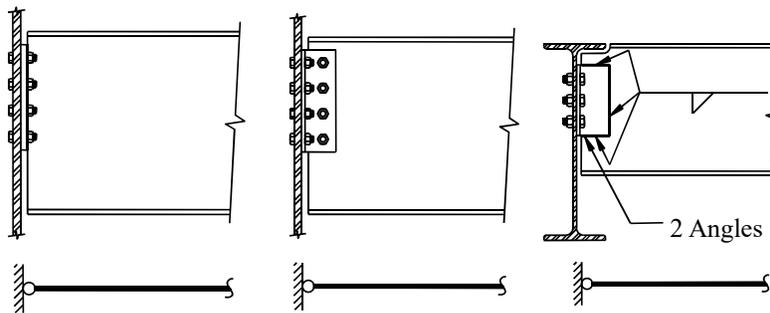
- Where is the pin?
Answer: At the most flexible side of the connection.



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Design Considerations

- Where is the pin?



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Design Considerations

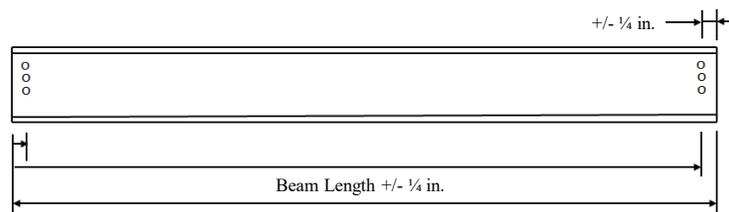
- Ductility Considerations
 - Angle or end plate thickness $\leq 5/8$ in.
 - Wide gage
 - Wide vertical weld spacing
 - Avoid welding along the top of angles or end plate
- Stability Consideration
 - Depth of Connection $\geq T / 2$
(T is clear distance between fillets – Tabulated in *Manual* Table 1-1)



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Design Considerations

- Beam Length Tolerance $\pm 1/4$ in.



Setbacks are usually $1/2$ in.

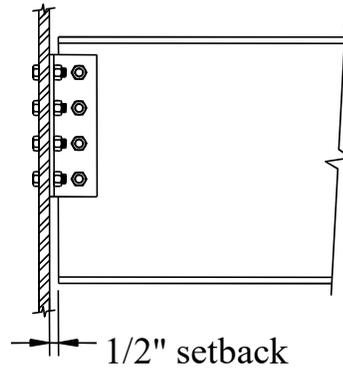
In calcs, end edge distances are taken as $1/4$ in. less than detailed.



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Design Considerations

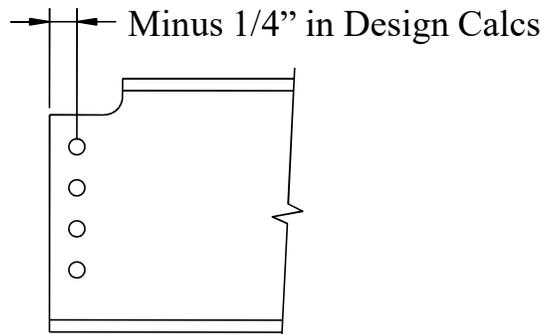
- Beam Length Tolerance + 1/4 in.



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Design Considerations

- Beam Length Tolerance + 1/4 in.

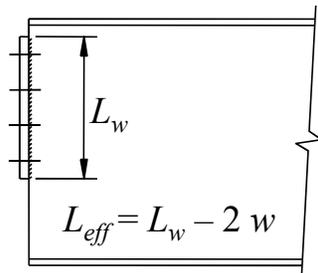


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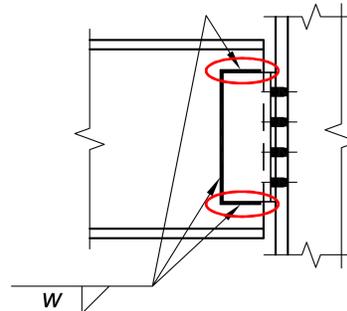
Design Considerations

- Effective Weld Length

When a weld terminates in the “air,” the dimensioned weld length is reduced by the weld size for calculations except for angles welded to a beam web.



Shear End-Plate



Double-Angle



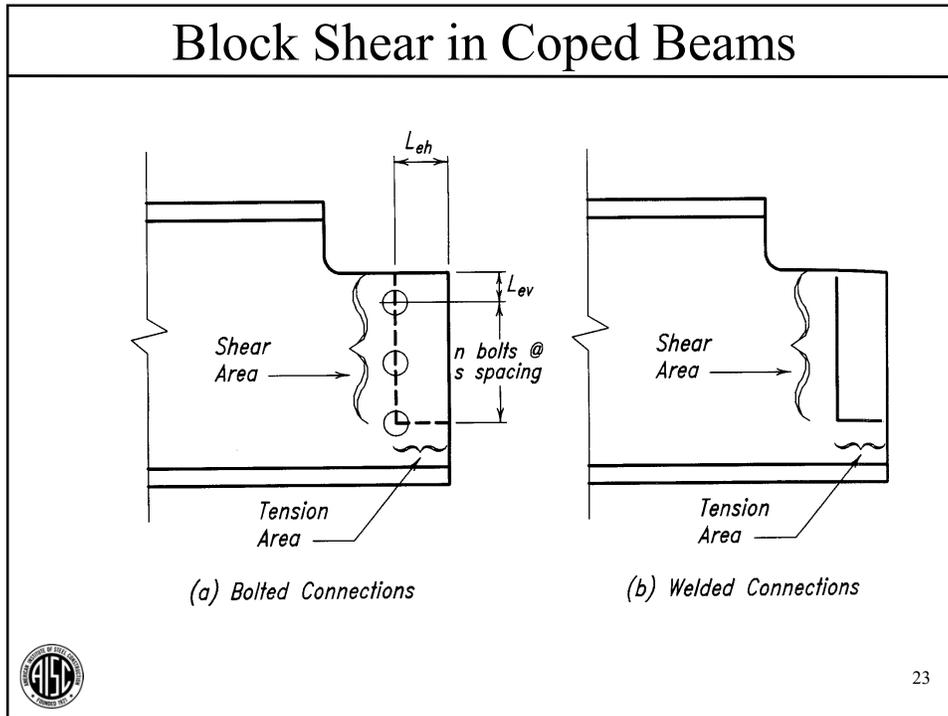
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Add'l Limit States for Shear Connections

- Block Shear in Coped Beams
 - Bolted at Web
 - Welded at Web
- Coped Beam Flexural Strength



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Block Shear in Coped Beams

Specification Section J4.3

$\phi = 0.75$

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

Equivalent to:

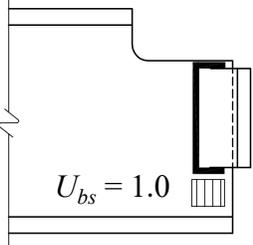
$$R_n = \min \left\{ \begin{array}{l} \text{Shear Rupture} \\ \text{Shear Yielding} \end{array} \right\} + \text{Tensile Rupture}$$

$$= \min \left\{ \begin{array}{l} 0.6F_u A_{nv} \\ 0.6F_y A_{gv} \end{array} \right\} + U_{bs} F_u A_{nt}$$

$U_{bs} = 1.0$ when tensile stress is uniform
 $= 0.5$ otherwise

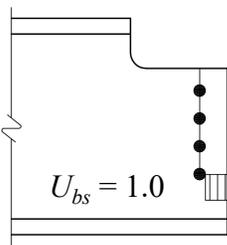
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Block Shear in Coped Beams



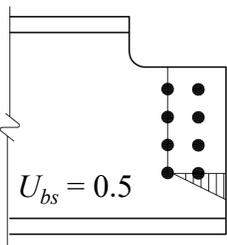
$U_{bs} = 1.0$

Welded Angle



$U_{bs} = 1.0$

Single-Row Beam End Connections



$U_{bs} = 0.5$

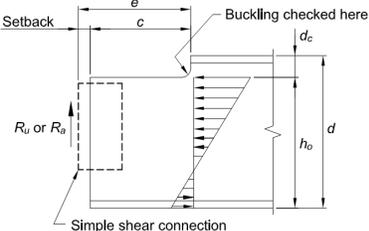
Multiple-Row Beam-End Connections

More Examples in *Commentary* Figure C-J4.2

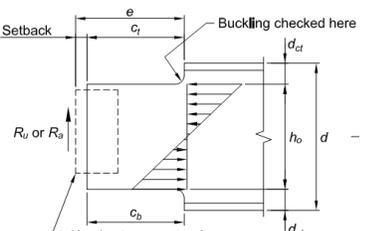


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Coped Beam Flexural Strength



Single Cope



Double Cope

Evaluation Criterion

$$M_u = R_u e \leq \phi_b M_n$$

$$\phi_b = 0.9$$

Limit States

- Flexural Yielding (C or T)
- Web Flexural Local Buckling (Single Cope)
- Web LTB (Double Cope)



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Coped Beam Flexural Strength

Single Coped Beam Flexural Strength

Manual Pages 9-6 through 9-9

$$\lambda \leq \lambda_p$$

$$M_n = M_p = F_y Z_{net}$$

$$\lambda_p < \lambda \leq 2\lambda_p$$

$$M_n = M_p - (M_p - M_y)(\lambda/\lambda_p - 1)$$

$$\lambda > 2\lambda_p$$

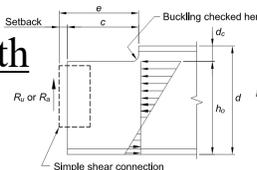
$$M_n = F_{cr} S_{net}$$

where

$$F_{cr} = 0.903 E k_1 / \lambda^2$$

S_{net} = net elastic section modulus at the cope, in.³

Z_{net} = net plastic section modulus at the cope, in.³



(Manual 9-6)

(Manual 9-7)

(Manual 9-8)



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Single Coped Beam Flexural Strength

Single Coped Beam Flexural Strength

$$\lambda = \text{web slenderness} = h_o / t_w$$

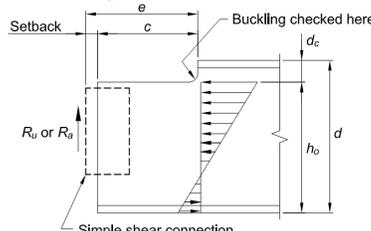
$$\lambda_p = 0.475 \sqrt{k_1 E / F_y}$$

$$k_1 = \max \begin{cases} f_k \\ 1.61 \end{cases}$$

$$k = 2.2 \left(\frac{h_o}{c} \right)^{1.65} \quad \text{if } \frac{c}{h_o} \leq 1.0$$

$$k = 2.2 \frac{h_o}{c} \quad \text{if } \frac{c}{h_o} > 1.0$$

$$f = \begin{cases} 2 \frac{c}{d} & \text{if } \frac{c}{d} \leq 1.0 \\ \min \begin{cases} 1 + c/d \\ 3.0 \end{cases} & \text{if } \frac{c}{d} > 1.0 \end{cases}$$



Single Cope



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Double Coped Beam Flexural Strength

Web Lateral-Torsional Buckling

Manual Page 9-9 and *Specification* Section F11

$$\lambda \leq \lambda_p \quad M_n = M_p = F_y Z \leq 1.6 F_y S \quad (\text{Spec. F11-1})$$

$$\lambda_p < \lambda \leq \lambda_r \quad M_n = C_b [1.52 - 0.274 \lambda (F_y/E)] M_y \leq M_p \quad (\text{Spec. F11-2})$$

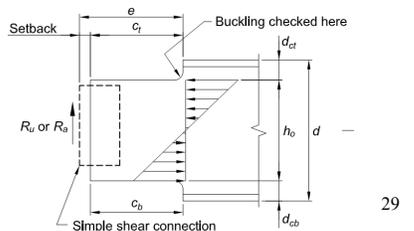
$$\lambda > \lambda_r \quad M_n = (1.9 E C_b / \lambda) S_x \leq M_p \quad (\text{Spec. F11-3})$$

where

$$\lambda = L_b h_o / t_w^2 \quad Z = t_w h_o^2 / 4$$

$$\lambda_p = 0.08 E / F_y \quad S = t_w h_o^2 / 6$$

$$\lambda_r = 1.9 E / F_y$$



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Double Coped Beam Flexural Strength

Web Lateral-Torsional Buckling, *Manual* Page 9-9

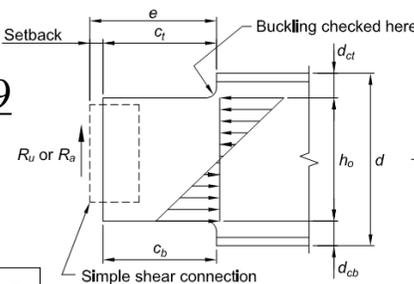
If $c_b \geq c_t$

$$C_b = \left(3 + \ln \frac{L_b}{d} \right) \left(1 - \frac{d_{ct}}{d} \right) \geq 1.84$$

Otherwise

$$C_b = \frac{c_b}{c_t} \left(3 + \ln \frac{L_b}{d} \right) \left(1 - \frac{d_{ct}}{d} \right) \geq 1.84$$

Note: When $c_b > c_t$ flexural tensile yielding must be checked at the bottom cope.

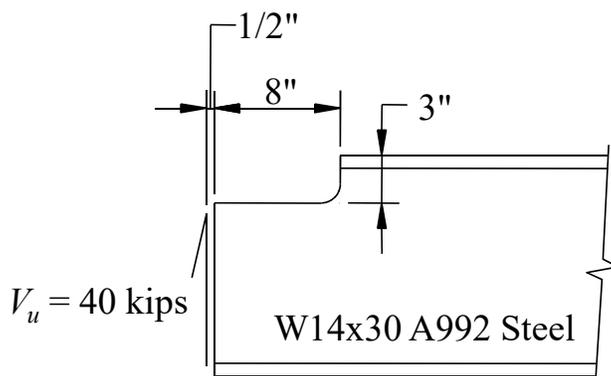


Note: First printing of the *Manual* Page 9-9 shows \leq . Should be \geq as shown here.

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Single Cope Flexural Strength Example

Example: Evaluate coped beam flexural strength.



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Coped Beam Flexural Strength Example

W14x30

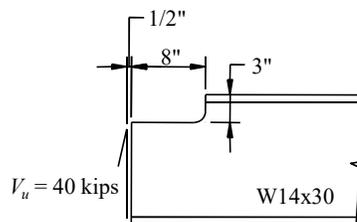
$$d = 13.8 \text{ in.} \quad t_w = 0.270 \text{ in.}$$

$$b_f = 6.73 \text{ in.} \quad t_f = 0.385 \text{ in.}$$

$$h_o = 13.8 - 3.0 = 10.8 \text{ in.}$$

$$S_{net} = 8.37 \text{ in.}^3 \text{ from } Manual \text{ Table 9-2}$$

$$Z_{net} = 15.1 \text{ in.}^3 \text{ from } Companion \text{ to the AISC Steel} \\ \text{Construction Manual, Volume 2, Table 9-A.}$$



Note: The distance h_o above $\neq h_o$ that is tabulated in
Manual Table 1-1 W-Shapes Dimensions.

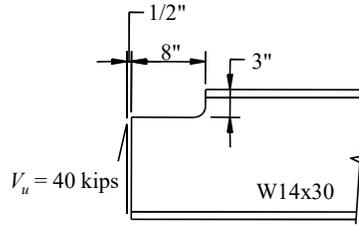


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Coped Beam Flexural Strength Example

Table 9-2 (continued)
 Elastic Section Modulus for Coped W-Shapes

Shape	d, in.	tf, in.	Sx, in. ³	So, in. ³	Snet, in. ³									
					dco, in.									
					2	3	4	5	6	7	8	9	10	
W14x132	14.7	1.03	209	38.1	28.6	24.3	20.3	16.7	13.4	10.5				
x120	14.5	0.940	190	34.2	25.5	21.7	18.1	14.8	11.8	9.20				
x109	14.3	0.860	173	30.0	22.3	18.9	15.7	12.8	10.2	7.91				
x99	14.2	0.780	157	27.2	20.2	17.0	14.2	11.5	9.15	7.04				
x90	14.0	0.710	143	24.3	18.0	15.2	12.6	10.2	8.07	6.18				
W14x82	14.3	0.855	123	28.0	20.9	17.7	14.8	12.1	9.64	7.46				
x74	14.2	0.785	112	24.4	18.2	15.4	12.8	10.4	8.31	6.40				
x68	14.0	0.720	103	22.2	16.5	13.9	11.6	9.41	7.46	5.72				
x61	13.9	0.645	92.1	19.7	14.6	12.3	10.2	8.28	6.54					
W14x53	13.9	0.660	77.8	19.1	14.2	12.0	9.93	8.07	6.39					
x48	13.8	0.595	70.2	17.3	12.8	10.8	8.93	7.23	5.71					
x43	13.7	0.530	62.6	15.3	11.3	9.49	7.84	6.34	4.99					
W14x38	14.1	0.515	54.6	16.0	12.0	10.2	8.48	6.94	5.54	4.28				
x34	14.0	0.455	48.6	14.4	10.8	9.14	7.62	6.22	4.95					
x30	13.8	0.385	42.0	13.2	9.8	8.37	7.06	5.68	4.51					
W14x26	13.9	0.420	35.3	12.3	9.20	7.80	6.50	5.31	4.23					
x22	13.7	0.335	29.0	10.7	7.97	6.75	5.62	4.58	3.64					
W12x336	16.8	2.96	483	123		83.1	71.4	60.6	50.8	41.9	34.1			
x305	16.3	2.71	435	108		71.4	61.0	51.4	42.7	34.9	28.0			
x279	15.9	2.47	393	96.1		63.1	53.5	44.8	36.9	29.8				
x252	15.4	2.25	353	83.7		54.2	45.7	38.0	31.0	24.8				
x230	15.1	2.07	321	74.2		47.5	39.9	32.9	26.7	21.1				



$$S_{net} = 8.37 \text{ in.}^3$$

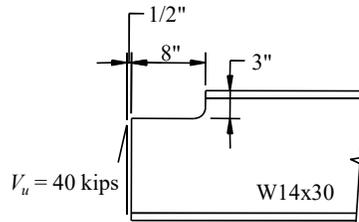


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Coped Beam Flexural Strength Example

Table 9-A (continued)
 Plastic Section Modulus for Coped W-Shapes

Shape	d, in.	tf, in.	Zx, in. ³	Zo, in. ³	Znet, in. ³									
					dco, in.									
					2	3	4	5	6	7	8	9	10	
W14x873	23.6	5.51	2030	916					532	480	432	387	346	
x808	22.8	5.12	1830	817					463	416	372	332	295	
x730	22.4	4.91	1660	669					421	380	341	306	273	243
x665	21.6	4.52	1480	579					358	321	287	256	227	201
x605	20.9	4.16	1320	503					305	272	242	214	189	166
x550	20.2	3.82	1180	434					289	258	229	203	179	157
x500	19.6	3.50	1050	379					248	221	195	172	150	131
x455	19.0	3.21	936	331					213	189	166	145	126	109
x420	18.7	3.04	869	301					193	170	149	130	113	97.0
x398	18.3	2.85	801	273					195	172	152	132	115	99.0
x370	17.9	2.66	736	246					174	154	134	117	101	86.5
x342	17.5	2.47	672	219					154	135	118	102	87.6	74.7
x311	17.1	2.26	603	193					134	117	102	87.5	74.6	63.1
x283	16.7	2.07	542	169					117	101	87.5	74.9	63.5	53.3
x257	16.4	1.89	487	150					102	88.8	76.3	64.9	54.7	45.6
x233	16.0	1.72	436	130					101	87.9	75.9	64.8	45.8	37.8
x211	15.7	1.56	390	115					88.9	77.1	66.2	56.3	47.3	39.3
x193	15.5	1.44	355	103					88.1	78.8	68.1	58.3	49.4	41.4
x176	15.2	1.31	320	92.2					80.6	71.6	61.6	51.6	43.5	36.2
x159	15.0	1.19	287	81.0					61.5	52.8	44.9	37.6	31.2	25.4
x145	14.8	1.09	260	72.2					54.5	46.7	39.6	33.1	27.2	22.1
W14x38	14.1	0.515	61.5	29.1	21.7	18.3	15.1	12.3	9.77	7.54				
x34	14.0	0.455	54.6	26.3	19.7	16.5	13.7	11.1	8.78	6.75				
x30	13.8	0.385	47.3	23.8	17.8	15.1	12.5	10.1	7.91					



$$Z_{net} = 15.1 \text{ in.}^3$$



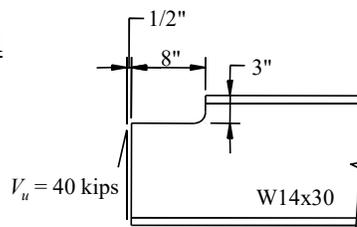
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Coped Beam Flexural Strength Example

Local Web Flexural Strength

Slenderness:

$$\lambda = \frac{h_o}{t_w} = \frac{10.8 \text{ in.}}{0.270 \text{ in.}} = 40$$



Limiting Slenderness for Yielding:

$$\lambda_p = 0.475 \sqrt{k_1 E / F_y} \quad \text{Need } k_1 = f k \geq 1.61$$

Plate Buckling Coefficient, k :

$$\frac{c}{h_o} = \frac{8 \text{ in.}}{10.8 \text{ in.}} \leq 1.0, \text{ so}$$

$$k = 2.2 \left(\frac{h_o}{c} \right)^{1.65} = 2.2 \left(\frac{10.8 \text{ in.}}{8 \text{ in.}} \right)^{1.65} = 3.61$$



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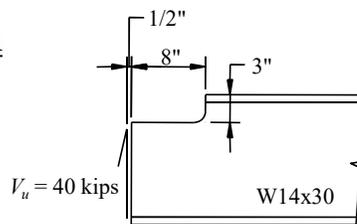
Coped Beam Flexural Strength Example

Local Web Flexural Strength

Buckling Adjustment Factor, f :

$$\frac{c}{d} = \frac{8 \text{ in.}}{13.8 \text{ in.}} = 0.580 \leq 1.0, \text{ so}$$

$$f = 2 \frac{c}{d} = 2(0.580) = 1.16$$



Modified Plate Bending Coefficient, k_1 :

$$k_1 = \max \begin{cases} f k = (1.16)(3.61) = 4.19 \\ 1.61 \end{cases} = 4.19$$



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Coped Beam Flexural Strength Example

Local Web Flexural Strength

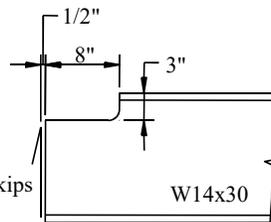
$$\begin{aligned}\lambda_p &= (0.475)\sqrt{k_1 E / F_y} \\ &= (0.475)\sqrt{(4.19)(29,000 / 50)} \\ &= 23.4\end{aligned}$$

$$\lambda_p < (\lambda = 40) < 2\lambda_p \Rightarrow \text{I.B.}$$

$$M_p = F_y Z_{net} = (50)(15.1) = 755 \text{ kip-in.}$$

$$M_y = F_y S_{net} = (50)(8.37) = 419 \text{ kip-in.}$$

$$\begin{aligned}M_n &= M_p - (M_p - M_y)(\lambda / \lambda_p - 1) \\ &= 755 - (755 - 419)(40 / 23.4 - 1) \\ &= 517 \text{ kip-in.}\end{aligned}$$



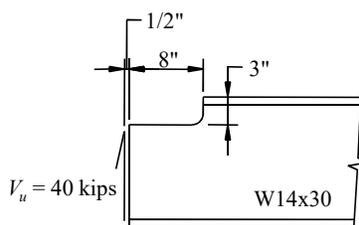
(Manual Eq. 9-7)



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Coped Beam Flexural Strength Example

Single Coped Beam Flexural Strength



$$\phi M_n = (0.9)(517 \text{ kip-in.}) = 465 \text{ kip-in.}$$

$$M_u = (40 \text{ kips})(8.5 \text{ in.}) = 340 \text{ kip-in.} < \phi M_n, \text{ OK}$$



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Shear End-Plate Connections



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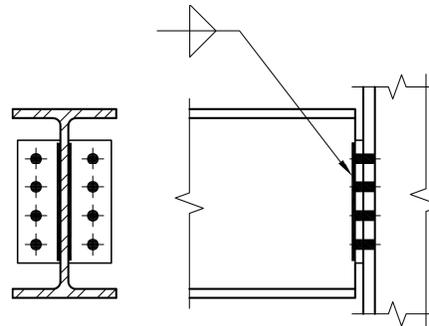
Shear End-Plate Connections

Advantages

- Simple – Few Parts
- No Holes in Beam

Disadvantage

- Requires Beam to be Cut to Exact Length



Manual Figure 10-6



40

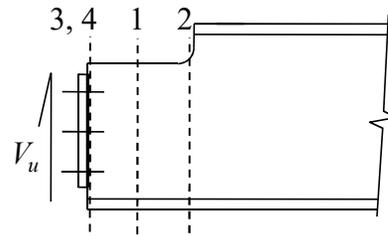
Shear End-Plate Connection Limit States

Beam

1. Shear Yielding
2. Coped Beam Flexural Strength
3. Web Base Metal Strength at Weld

Weld

4. Weld Rupture Strength

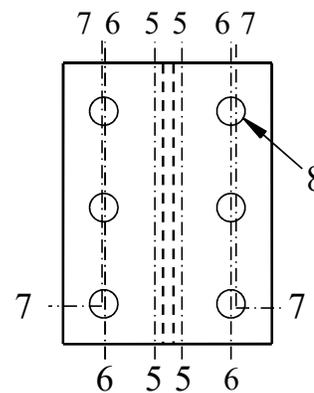


41

Shear End-Plate Connection Limit States

Plate

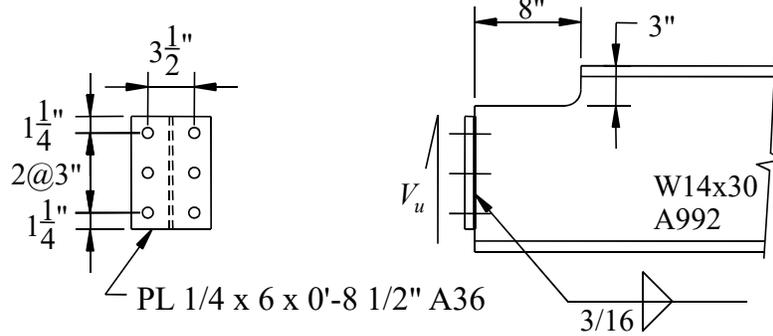
5. Shear Yielding
6. Shear Rupture
7. Block Shear
8. Shear Transfer Between Plate and Support



42

Shear End-Plate Connection Example

Example: Determine the design strength, ϕV_n .
 3/4 in. Gr. A325-N Bolts, E70XX Electrode



Girder: A992 steel
 $t_w = 0.5$ in.

W14x30 $F_y = 50$ ksi $F_u = 65$ ksi
 $d = 13.8$ in. $t_w = 0.27$ in.



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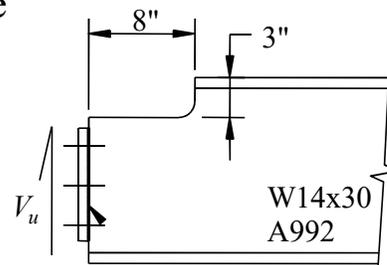
Shear End-Plate Connection Example

1. Shear Yielding at Cope

$$d = 13.8 \text{ in.}$$

$$d_c = 3.0 \text{ in.}$$

Spec. J4-3:



$$\begin{aligned} \phi V_n &= \phi(0.6 F_y)(d-d_c)t_w \\ &= (1.0)(0.6)(50)(13.8-3.0)(0.27) \\ &= 87.5 \text{ kips} \end{aligned}$$



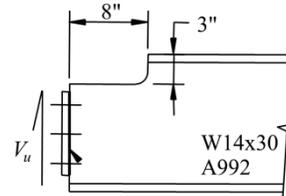
44

Shear End-Plate Connection Example

2. Coped Beam Flexural Strength

From previous example

$$\phi M_n = 465 \text{ kip-in.}$$



With $e = \text{cope length} + \text{plate thickness}$
 $= 8.0 + 0.25 = 8.25 \text{ in.}$

$$\begin{aligned} \phi V_n &= 465 / 8.25 \\ &= 56.4 \text{ kips} \end{aligned}$$



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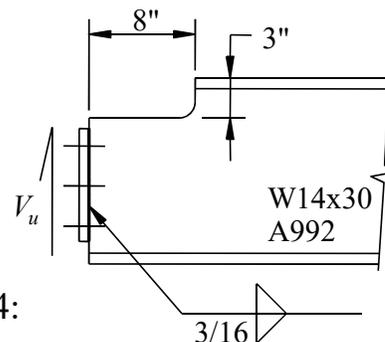
Shear End-Plate Connection Example

3. Web Shear Rupture Strength at Weld

Plate $L = 8.5 \text{ in.}$

$t_{\text{weld}} = 3/16 \text{ in.}$

Beam Web $t_w = 0.270 \text{ in.}$



Spec. Table J2.5 and Eq. J4-4:

$$\begin{aligned} \phi V_n &= \phi(0.6F_u)(L - 2t_{\text{weld}})t_w \\ &= 0.75(0.6)(65)[8.5 - (2)(3/16)](0.270) \\ &= 64.2 \text{ kips} \end{aligned}$$

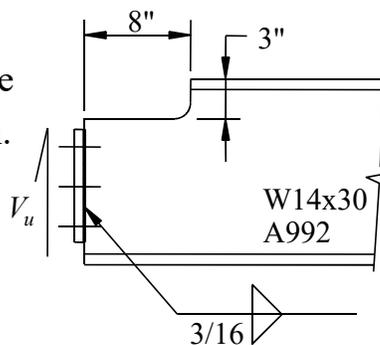


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Shear End-Plate Connection Example

4. Weld Rupture Strength

From *Manual* Table J2.4, the minimum weld size is 1/8 in.
 3/16 in. weld OK so far.



$$\begin{aligned}\phi V_n &= (1.392)(D)(L - 2t_{weld}) \\ &= (1.392)(3)[8.5 - (2)(3/16)](2 \text{ welds}) \\ &= 67.9 \text{ kips}\end{aligned}$$



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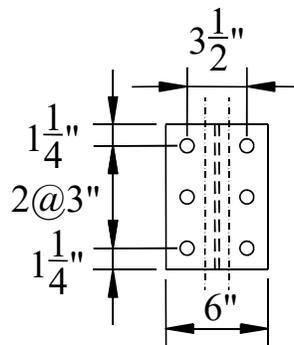
Shear End-Plate Connection Example

Plate Limit States

$$t_p = 1/4 \text{ in.}$$

A36 Steel:

$$F_y = 36 \text{ ksi} \quad F_u = 58 \text{ ksi}$$



5. Shear Yielding

$$\begin{aligned}\phi V_n &= \phi(0.6F_y)(2 L t_p) \quad (\text{Spec. J4-3}) \\ &= 1.0 (0.6 \times 36) (2 \times 8.5 \times 1/4) \\ &= 91.8 \text{ kips}\end{aligned}$$



48

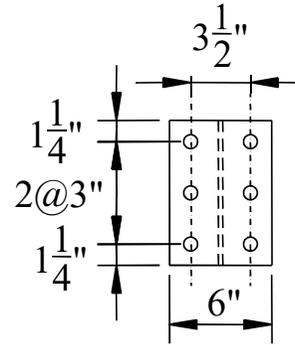
Shear End-Plate Connection Example

6. Shear Rupture

$$d'_h = 3/4 + 1/16 + 1/16 = 7/8 \text{ in.}$$

$$A_{nv} = (1/4)[8.5 - 3(7/8)](2) = 2.94 \text{ in.}^2$$

$$\begin{aligned} \phi V_n &= \phi 0.6 F_u A_{nv} \\ &= (0.75)(0.6)(58)(2.94) \\ &= 76.7 \text{ kips} \end{aligned}$$



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Shear End-Plate Connection Example

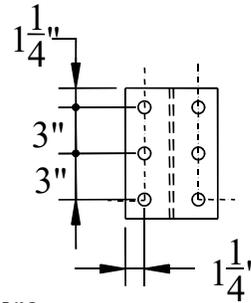
7. Block Shear Strength

PL 1/4 x 6 x 0'-8 1/2"

$\phi = 0.75$

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

$$= \min \begin{cases} 0.6 F_u A_{nv} \\ 0.6 F_y A_{gv} \end{cases} + U_{bs} F_u A_{nt} \quad (\text{Spec. J4-5})$$



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Shear End-Plate Connection Example

7. Plate Block Shear

- Shear Rupture

$$0.6F_u A_{nv} = (0.6)(58)(1/4)[7.25 - (2.5)(7/8)](2)$$

$$= 88.1 \text{ kips}$$

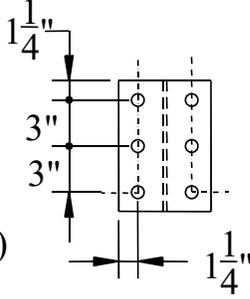
- Shear Yielding

$$0.6F_y A_{gv} = (0.6)(36)(1/4)(7.25)(2)$$

$$= 78.3 \text{ kips}$$

Tensile Rupture

$$F_u A_{nt} = 58(1/4)[1.25 - (0.5)(7/8)](2)$$

$$= 23.6 \text{ kips}$$


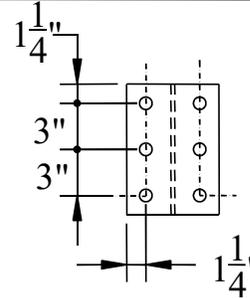

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Shear End-Plate Connection Example

7. Plate Block Shear

$$R_n = \min \left\{ \begin{array}{l} \text{Shear Rupture} \\ \text{Shear Yielding} \end{array} \right. + \text{Tensile Rupture}$$

$$= \min \left\{ \begin{array}{l} 0.6F_u A_{nv} \\ 0.6F_y A_{gv} \end{array} \right. + U_{bs} F_u A_{nt}$$

$$= \min \left\{ \begin{array}{l} 88.1 \text{ kips} \\ 78.3 \text{ kips} \end{array} \right. + (1.0)(23.6 \text{ kips}) = 102 \text{ kips}$$


$$\phi V_n = (0.75)(102) = 76.4 \text{ kips}$$


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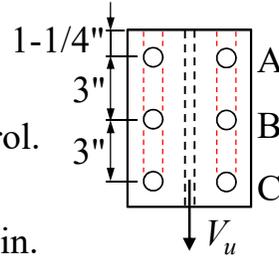
Shear End-Plate Connection Example

8. Shear Transfer Between Plate and Girder

A992 girder with $t_w = 0.5$ in.

A36 plate with $t = 0.25$ in.

Therefore, the plate **won't** will control.



Bearing: $r_n = 26.1$ kips with $d = 3/4$ in.

Tearout with $d_h = 13/16$ in.

At A: $r_n = 14.7$ kips

At B and C: $r_n = 38.1$ kips



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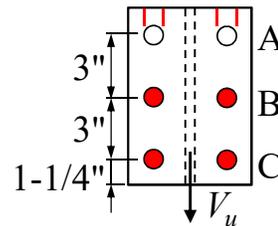
Shear End-Plate Connection Example

8. Shear Transfer Between Plate and Girder

Bolt Shear Rupture

$$r_n = 23.9 \text{ kips}$$

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



$$r_{nA} = \min \begin{cases} 23.9 \text{ kips} \\ 26.1 \text{ kips} \\ 14.7 \text{ kips} \leftarrow \end{cases} \quad r_{nB} = r_{nC} = \min \begin{cases} 23.9 \text{ kips} \leftarrow \\ 26.1 \text{ kips} \\ 38.1 \text{ kips} \end{cases}$$

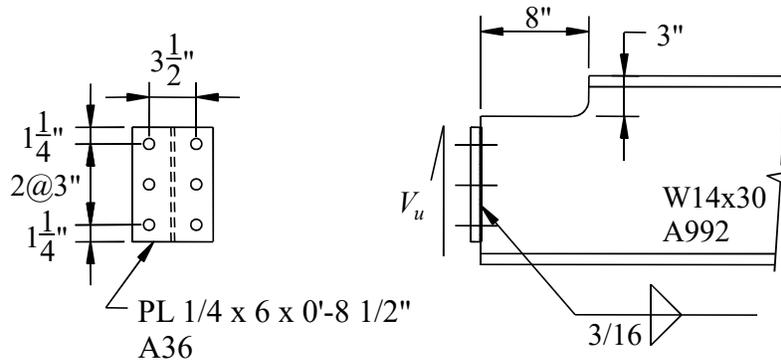


$$\phi R_n = (0.75)[(14.7)(2) + (23.9)(4)] = 93.8 \text{ kips}$$

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Shear End-Plate Connection Example

Connection Design Strength



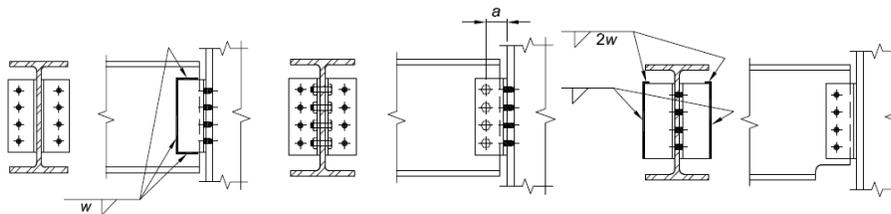
Coped Beam Flexural Strength Controls

$$\phi V_n = 56.4 \text{ kips}$$



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Double-Angle Connections



Welded/Bolted

Bolted/Bolted

Bolted/Welded

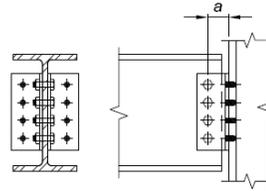


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Double-Angle Connections

Advantages

- Beam length can vary.
- Weld or bolt to beam.
- Strong



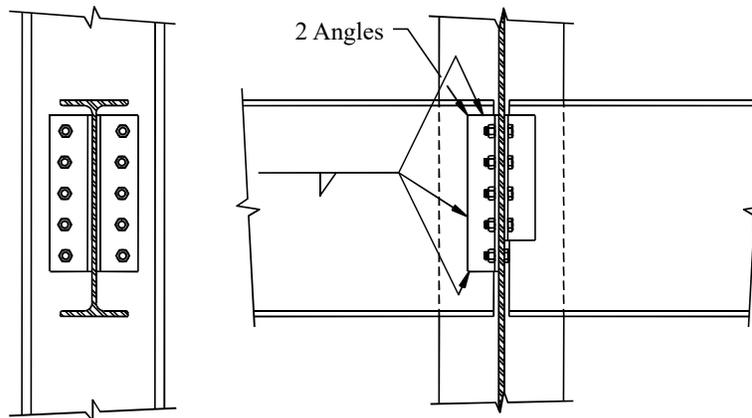
Disadvantage

- For double-sided connections at a column or girder web, *shared bolts* cause an erection safety issue.



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Solution of Erection Safety Issue

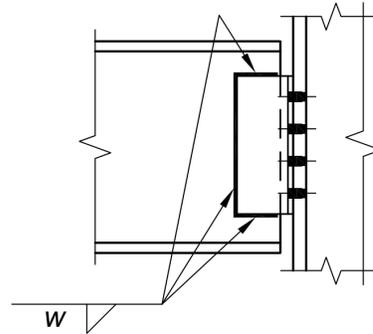


Double-Sided Connection into Column Web



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Welded/Bolted Double-Angle Connections

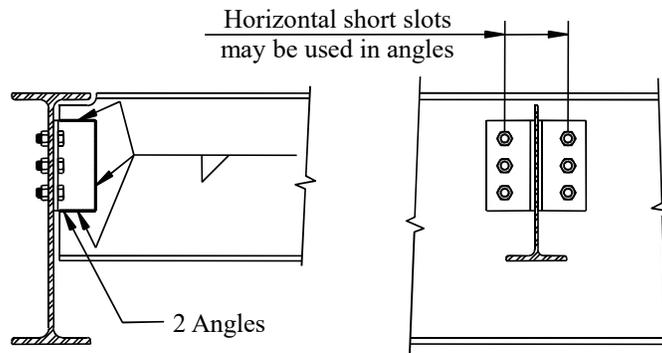


Manual Figure 10-4(b)



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Welded/Bolted Double-Angle Connections



Pin is at face of supporting element

Beam web weld is subjected to eccentric shear.



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Welded/Bolted Double-Angle Connections

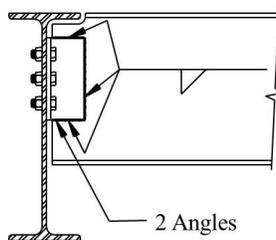
Limit States

Beam

- Shear Yielding
- Coped Beam Flexural Strength
- Block Shear
- Web Base Metal Strength at Weld

Weld

- Rupture of Eccentrically Loaded Weld Group



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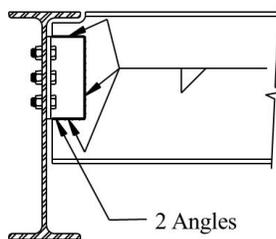
Welded/Bolted Double-Angle Connections

Angles

- Base Metal Strength at Weld
- Yielding
- Shear Rupture
- Block Shear

Shear Transfer

- Angle Bearing / Tearout
- Bolt Shear Rupture
- Supporting Element Bearing / Tearout

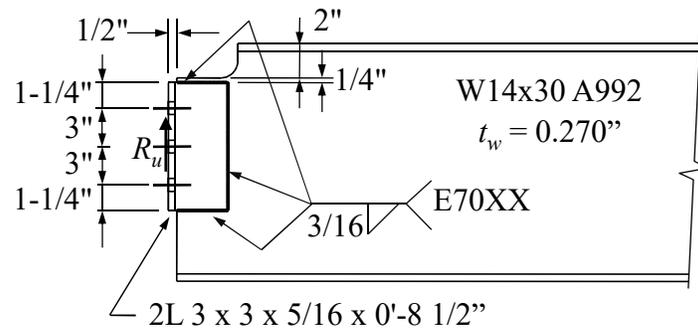


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Welded/Bolted Double-Angle Example

Example: Determine ϕR_n for

1. Beam Web Block Shear
2. Weld Rupture due to Eccentric Shear
3. Beam Web Strength at Weld



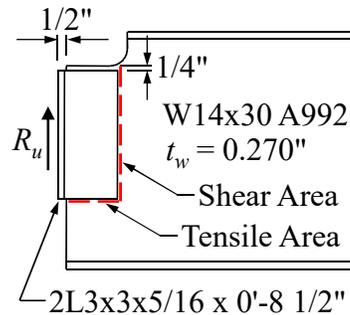
63

Welded/Bolted Double-Angle Example

1. Beam Web Block Shear

$$R_n = \min \left[\begin{array}{l} 0.6F_u A_{nv} \\ 0.6F_y A_{gv} \end{array} + U_{bs} F_u A_{nt} \right]$$

- $A_{nv} = A_{gv}$ (no holes)
- $0.6F_u A_{nv} > 0.6F_y A_{gv}$
- Shear yielding controls



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

$$= (0.6)(50)(0.270)(8.5 + 1/4) + (1.0)(65)(0.270)(3 - 1/2 - 1/4)$$

$$= 110 \text{ kips}$$

$$\phi R_n = (0.75)(110) = 82.8 \text{ kips}$$

Setback

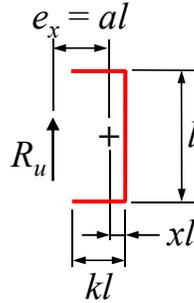
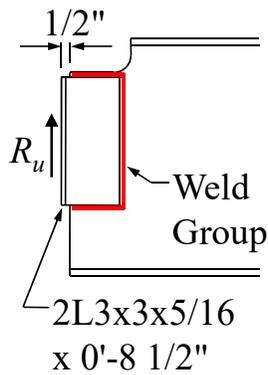
Beam
Length
Tolerance



64

Welded/Bolted Double-Angle Example

2. Weld Rupture Due to Eccentric Shear



Instantaneous Center
of Rotation Method

Design Aid: Table 8-8

$$R_n = CC_1 D l$$

$$\phi = 0.75$$



65

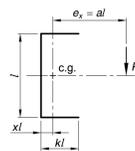
Welded/Bolted Double-Angle Example

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
 $R_n = CC_1 D l$ ($\phi = 0.75$, $\Omega = 2.00$)

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_s , 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	1.68	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37	5.86	6.36	7.37	8.40	9.42	10.5	11.5	
0.30	1.60	1.97	2.39	2.81	3.25	3.70	4.16	4.63	5.10	5.58	6.06	7.04	8.04	9.04	10.0	11.0	
0.35	1.52	1.87	2.27	2.67	3.10	3.54	4.00	4.46	4.93	5.40	5.88	6.83	7.81	8.78	9.73	10.6	
0.40	1.44	1.77	2.15	2.53	2.94	3.36	3.81	4.26	4.72	5.18	5.64	6.56	7.51	8.46	9.39	10.2	
0.45	1.36	1.67	2.03	2.39	2.78	3.18	3.61	4.04	4.48	4.92	5.37	6.26	7.19	8.11	9.01	9.88	
0.50	1.28	1.57	1.91	2.25	2.62	2.99	3.40	3.81	4.23	4.65	5.08	5.94	6.84	7.72	8.58	9.42	
0.55	1.20	1.47	1.78	2.10	2.45	2.80	3.18	3.57	3.96	4.35	4.74	5.57	6.43	7.28	8.11	8.92	
0.60	1.12	1.38	1.66	1.95	2.28	2.60	2.95	3.31	3.67	4.03	4.39	5.18	6.00	6.81	7.61	8.38	
0.65	1.04	1.28	1.53	1.79	2.08	2.36	2.68	2.99	3.30	3.60	3.90	4.64	5.41	6.18	6.93	7.66	
0.70	0.96	1.18	1.40	1.63	1.88	2.13	2.41	2.68	2.94	3.20	3.46	4.15	4.87	5.59	6.30	6.98	
0.75	0.88	1.08	1.27	1.47	1.68	1.89	2.12	2.34	2.56	2.77	2.98	3.62	4.29	4.95	5.62	6.25	
0.80	0.80	0.98	1.14	1.31	1.48	1.63	1.82	2.00	2.18	2.34	2.50	3.08	3.69	4.29	4.91	5.48	
0.85	0.72	0.88	1.02	1.16	1.29	1.41	1.56	1.70	1.83	1.95	2.07	2.59	3.13	3.65	4.21	4.73	
0.90	0.64	0.78	0.90	1.01	1.11	1.19	1.30	1.40	1.49	1.57	1.64	2.11	2.58	3.05	3.54	4.01	
0.95	0.56	0.68	0.78	0.86	0.93	0.99	1.06	1.12	1.17	1.21	1.24	1.64	2.03	2.42	2.83	3.24	
1.00	0.48	0.58	0.66	0.72	0.77	0.81	0.84	0.87	0.89	0.91	0.92	1.26	1.57	1.89	2.23	2.58	
1.05	0.40	0.49	0.55	0.59	0.62	0.64	0.66	0.67	0.68	0.69	0.69	0.97	1.21	1.46	1.72	1.98	
1.10	0.32	0.39	0.43	0.46	0.48	0.49	0.50	0.50	0.50	0.50	0.50	0.71	0.87	1.04	1.21	1.38	
1.15	0.24	0.29	0.31	0.32	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.46	0.55	0.64	0.73	0.81	
1.20	0.16	0.19	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.26	0.31	0.35	0.39	0.43	
1.25	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.11	0.13	0.14	0.15	0.15	
1.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

$$R_n = CC_1 D l$$

$$k \rightarrow x$$

$$x \rightarrow a$$

$$k \ \& \ a \rightarrow C$$

C_1 from Table 8-3.

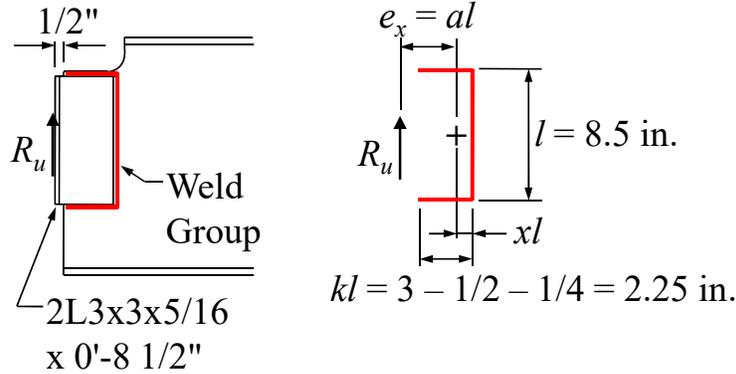
$C_1 = 1.00$ for 70 ksi



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Welded/Bolted Double-Angle Example

Determine C from Table 8-8:



$$k = 2.25 / 8.5 = 0.265$$



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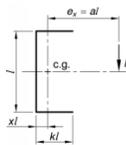
Welded/Bolted Double-Angle Example

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_s , 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
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	0.346	0.421	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

$$k = 0.265 \rightarrow x = 0.0465$$

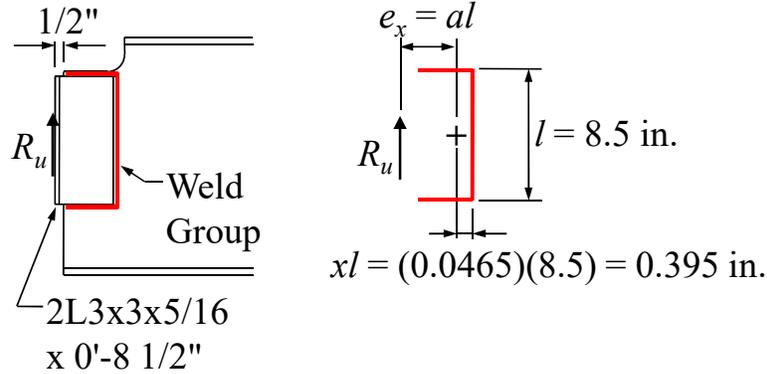


68



Welded/Bolted Double-Angle Example

Determine C from Table 8-8:



69

Welded/Bolted Double-Angle Example

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
 $R_n = C C_1 D$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD				ASD														
$C_{min} = \frac{P_u}{\phi C_1 D}$	$D_{min} = \frac{P_u}{\phi C C_1 l}$	$l_{min} = \frac{P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_u}{C_1 D}$	$D_{min} = \frac{\Omega P_u}{C C_1 l}$	$l_{min} = \frac{\Omega P_u}{C C_1 D}$													
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	7.71	8.26	8.82	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	7.66	8.21	8.76	9.31	10.4	11.5	12.6
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.41	6.94	7.47	8.00	8.53	9.06	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	7.20	7.72	8.24	8.76	9.83	10.9	12.0
0.25	1.66	2.07	2.51	2.98	3.42	3.90	4.38	4.87	5.37	5.86	6.36	6.86	7.37	7.87	8.38	9.42	10.5	11.5
0.30	1.55	1.95	2.36	2.79	3.23	3.68	4.14	4.60	5.08	5.55	6.03	6.51	7.01	7.50	8.00	9.00	10.0	11.0
0.40	1.33	1.69	2.07	2.45	2.84	3.24	3.65	4.07	4.50	4.94	5.39	5.84	6.30	6.74	7.24	8.19	9.16	10.1

where
 P = required force, P_u or P_s , kips
 D = number of sixteenths-of-an-inch in the fillet weld size
 l = characteristic length of weld group, in.
 $a = e_y/l$
 e_y = 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, J2.4(a), J2.4(b) and J2.4(c).

$k = 0.265$
 $a = 0.307$
 $k \& a \Rightarrow C = 2.62$



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Welded/Bolted Double-Angle Example

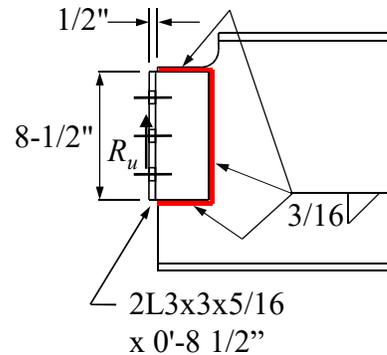
$$C = 2.62$$

$$3/16 \text{ in. welds} \rightarrow D = 3$$

$$E70XX \rightarrow C_1 = 1.0$$

Strength of eccentrically loaded weld group:

$$\begin{aligned} \phi R_n &= \phi C C_1 D l \\ &= (0.75)(2.62)(1.0)(3)(8.5)(2) \\ &= 100 \text{ kips} \end{aligned}$$



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Welded/Bolted Double-Angle Example

3. Beam Web Strength at Weld

Proposed Rational Approach

$$\begin{aligned} \phi R_n &= (\phi R_n)_{Weld} \frac{\text{Web Shear Rupture Strength / in.}}{\text{Weld Rupture Strength / in.}} \\ &= (\phi R_n)_{Weld} \frac{\phi 0.6 F_u t_w (1.0 \text{ in.})}{(1.392)(D)(1.0 \text{ in.})(2 \text{ welds})} \\ &= (100) \frac{(0.75)(0.6)(65)(0.270)(1.0)}{(1.392)(3)(1.0)(2)} \\ &= 94.6 \text{ kips} \end{aligned}$$

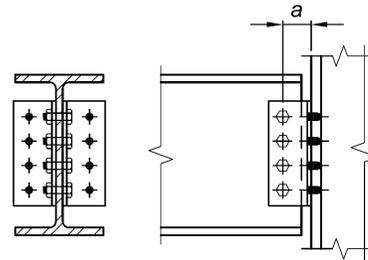


72

Bolted/Bolted Double-Angle Connections



Beam to Girder



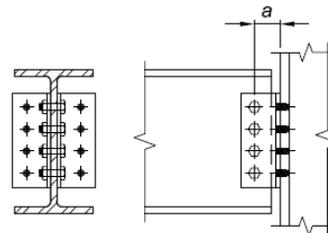
Beam to Column Flange



73

Bolted/Bolted Double-Angle Connections

Bolt eccentricity ignored
in bolted/bolted double-
angle connections.



Shear transfer between angles and beam web and
angles and supporting element as previous:

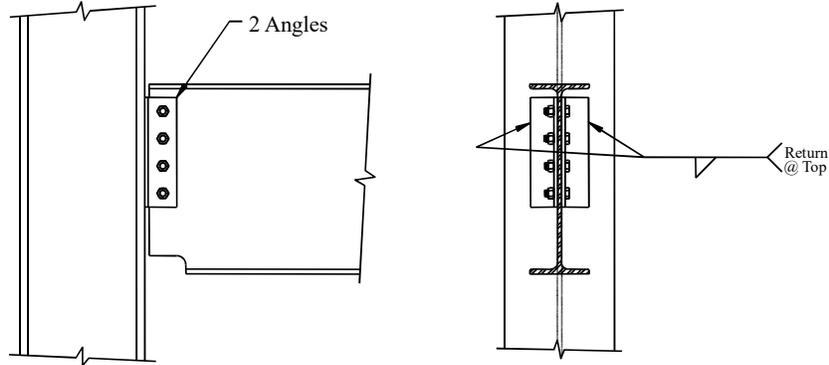
Min of Bearing, Tearout & Bolt Shear Rupture
at each hole/bolt.

No Additional Limit States



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Bolted/Welded Double-Angle Connections

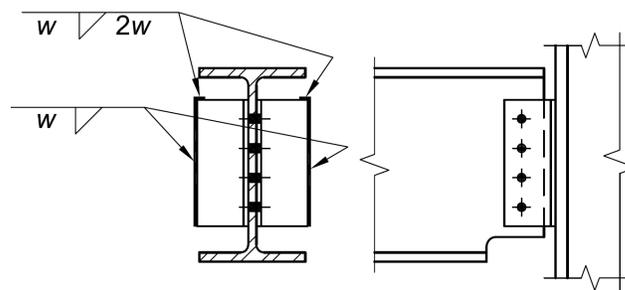


Bolted/Welded Double-Angle Knife Connection



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Double-Angle Knife Connections



Weld returns per *Specification* Section J2.2b User Note.

Bolted/Welded to Column Flange



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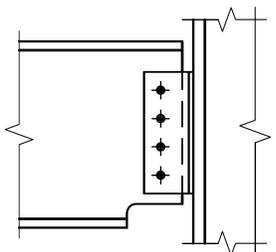
Bolted/Welded Double-Angle

Bolted to Beam / Welded to Column Flange

- Referred to as a “Knife” Connection
- Bottom Cope to Permit Erection

Additional Limit States

- Coped Beam Web Strength at Tension Flange
- Weld Strength on Outstanding Legs (OSLs) – angle-to-column flange connection.




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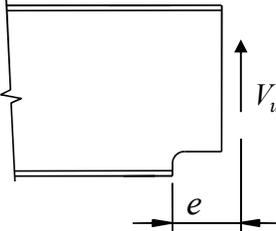
Bolted/Welded Double-Angle

Coped Beam Web Strength at Tension Flange

$\phi_b = 0.9$
 $V_n = M_n / e$
 where

$$M_n = \min \begin{cases} M_p = F_y Z_{net} \\ 1.6M_y = 1.6F_y S_{net} \end{cases}$$

S_{net} = elastic section modulus from *Manual* Table 9-2
 Z_{net} = plastic section modulus from *AISC Companion to the AISC Steel Construction Manual* Table 9-A.

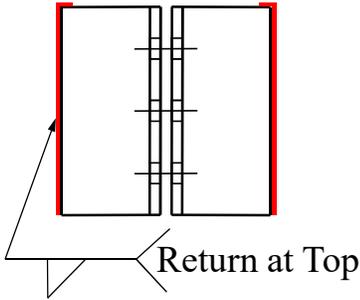



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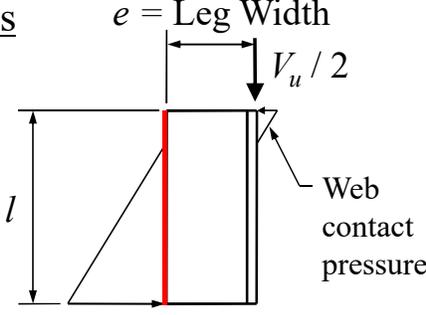
Bolted/Welded Double-Angle

Weld Strength on OSLs



Return at Top

$e = \text{Leg Width}$



Web contact pressure

Elastic Vector Method

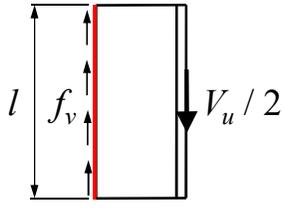
- Uniform vertical weld force.
- Linearly varying horizontal weld force.
- Neglect returns.


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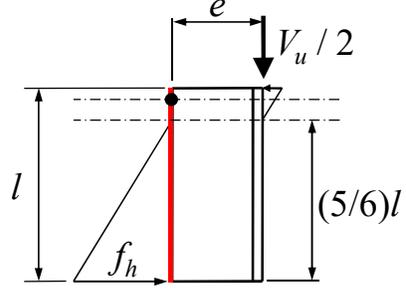
Bolted/Welded Double-Angle

Vertical Weld Force

$$f_v = \frac{V_u / 2}{l}$$



Max Horizontal Weld Force



f_v and f_h
in
kips/in.

$$\sum M. = 0 \Rightarrow f_h = 1.8 \frac{V_u e}{l^2}$$


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Bolted/Welded Double-Angle

Maximum Weld Force as a function of V_u

$$f_u = \sqrt{f_h^2 + f_v^2} = \frac{V_u}{2l^2} \sqrt{l^2 + 12.96e^2} \quad (\text{kips/in.})$$

Design Weld Force (kips/in.) with $\theta = 0^\circ$

$$\phi f_n = 1.392D$$

Weld Group Design Shear Strength, ϕV_n

$$\phi f_n = \frac{\phi V_n}{2l^2} \sqrt{l^2 + 12.96e^2}$$

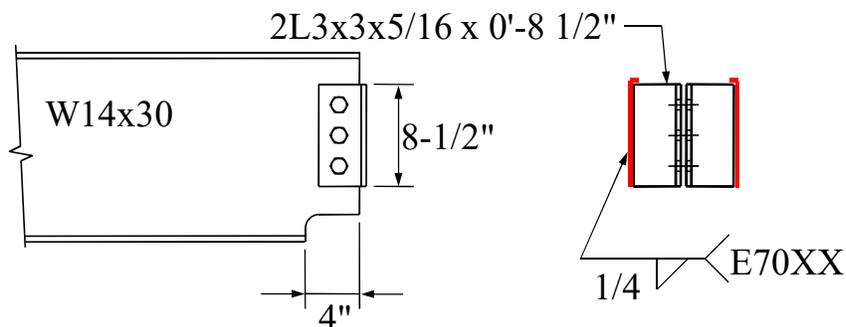
$$\phi V_n = \frac{2(1.392Dl)}{\sqrt{1 + 12.96e^2/l^2}} \quad (\text{Manual 10-1a})$$



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Bolted/Welded Double-Angle Example

Example: Calculate the weld design rupture strength at OSLs, ϕV_n .



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Bolted/Welded Double-Angle Example

Weld design rupture strength at OSLs

$$\begin{aligned}\phi V_n &= \frac{2(1.392Dl)}{\sqrt{1+12.96e^2/l^2}} \\ &= \frac{(2)(1.392)(4)(8.5)}{\sqrt{1+12.96(3^2)/(8.5^2)}} \\ &= 58.5 \text{ kips}\end{aligned}$$



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End of Session 3

Thank You for
Attending

Next Up



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Next Session

- November 13, 2019 Shear Connections Part II

TOPICS

- Single-Angle Connections
- Single-Plate (Shear Tab) Connections
- Unstiffened Seated Connections
- Stiffened Seated 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.

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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 bar with links for EDUCATION, PUBLICATIONS, AWARDS AND COMPETITIONS, TECHNICAL RESOURCES, and STEEL SOLUTIONS CENTER. Below the navigation bar is a large image of a modern building with a glass facade and palm trees, with the AISC logo overlaid. Below the image is a breadcrumb trail: 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

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: BDPC1	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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