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Course Description

Session 3: October 17, 2017 – Shear Connections Part I.

This live webinar provides an overview of various types of shear connections, including the advantages and disadvantages of each. Design considerations for shear connections, a review of limit states for block shear and flexural strength in coped beams are presented. Shear end-plate and double angle connection designs are also discussed. Design examples are presented to demonstrate the concepts.



Learning Objectives

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

- List several types of shear (framing) connections.
- List the limit states for framing connections.
- List the steps in designing shear end-plate connections.
- List the steps in designing double angle connections.



There's always a solution in steel.

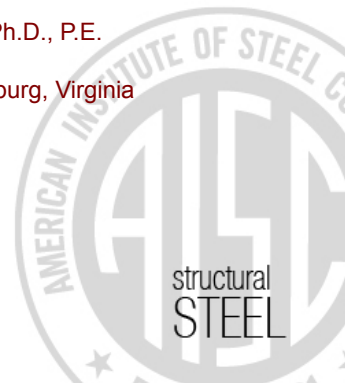
Fundamentals of Connection Design

Session 3: Shear Connections – Part 1

October 17, 2017



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



SCHEDULE

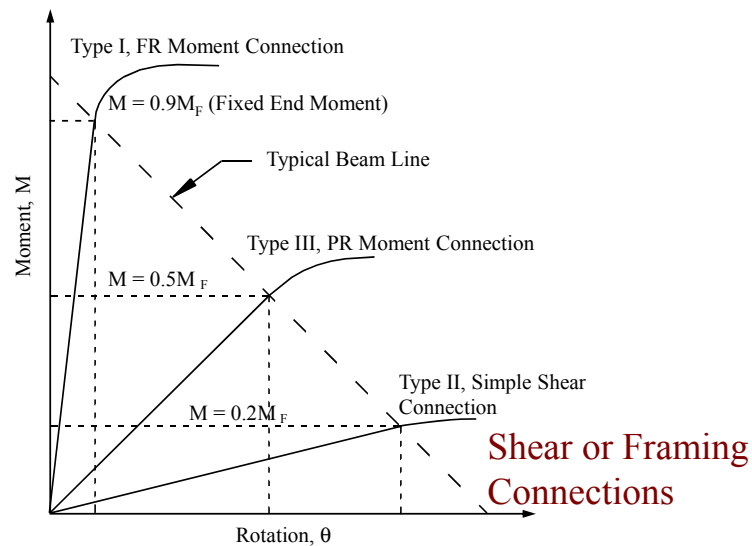
- **October 03, 2017** Fundamental Concepts Part I
- **October 10, 2017** Fundamental Concepts Part II
- **October 17, 2017** **Shear Connections Part I**
- **October 24, 2017** **Shear Connections Part II**
- **November 07, 2017** **Moment Connections Part I**
- **November 14, 2017** **Moment Connections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

SHEAR (FRAMING) CONNECTIONS PART I

TOPICS

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

Shear (Framing) Connections

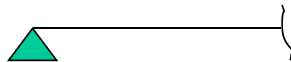


Types of Shear Connections

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

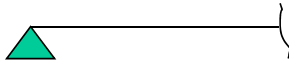
Design Considerations

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



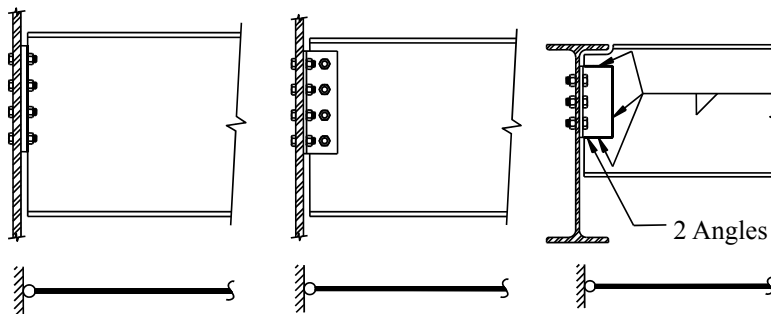
Design Considerations

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



Design Considerations

- Where is the pin?

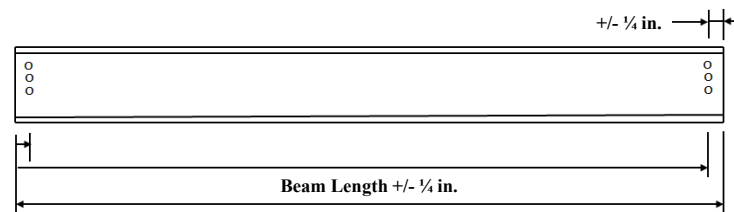


Design Considerations

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

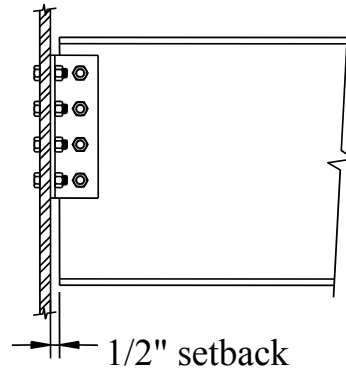
Design Considerations

- Beam Length Tolerance $\pm 1/4$ in.
For design:
Setbacks in calculations are usually $1/2$ in.
End edge distances are taken in calcs $1/4$ in. less than detailed.



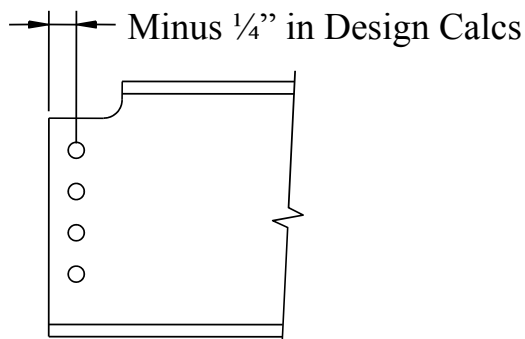
Design Considerations

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



Design Considerations

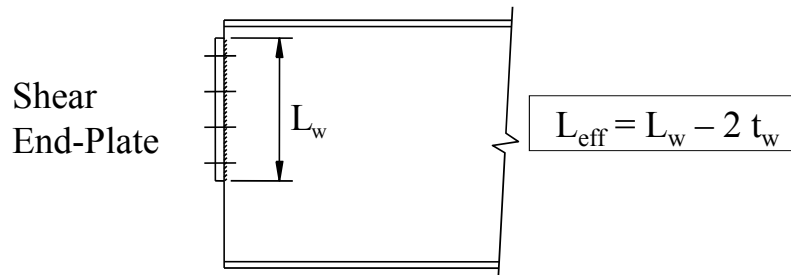
- Beam Length Tolerance $\pm 1/4$ in.



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.



New Limit States

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

Block Shear in Coped Beams

(a) Bolted Connections (b) Welded Connections

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Block Shear in Coped Beams

Block Shear Strength
Specification Section J4.3

$\phi = 0.75$

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

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

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

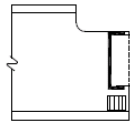
Equivalent to:

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

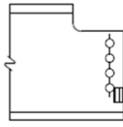
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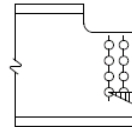
Block Shear in Coped Beams



Welded Angle



Single-Row Beam End Connections



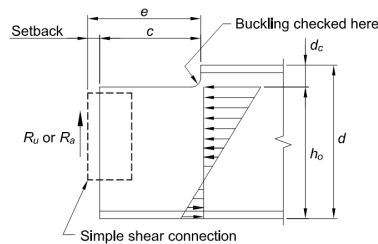
Multiple-Row Beam End Connections

$$\underline{U_{bs}} = 1.0$$

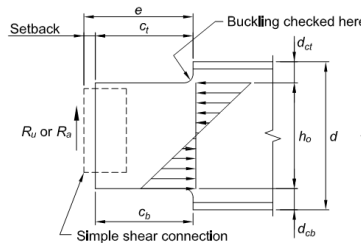
$$\underline{U_{bs}} = 0.5$$

See *Commentary Figure C-J4.2* for more examples.

Coped Beam Flexural Strength



Single Cope



Double Cope

Design Strength

$$\phi_b = 0.9$$

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

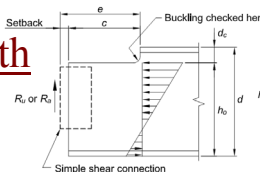
Limit States

- Flexural Yielding (C or T)
- Local Web Buckling

Coped Beam Flexural Strength

Single Coped Beam Flexural Strength

Manual pp. 9-6 and 9-9



$$\lambda \leq \lambda_p$$

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

(Eqn.9-6)

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

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

(Eqn.9-7)

$$\lambda > 2\lambda_p$$

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

(Eqn.9-8)

where

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

S_{net} = net elastic section modulus

Z_{net} = net plastic section modulus

Single Coped Beam Flexural Strength

Single Coped Beam Flexural Strength

λ = web slenderness = h_o/t_w

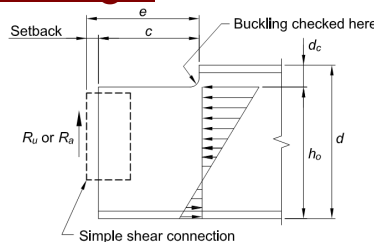
$$\lambda_p = (0.475) \sqrt{k_1 E / F_y}$$

$$k_1 = \max \left| \begin{array}{l} f k \\ 1.61 \end{array} \right.$$

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

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

$$f = \left| \begin{array}{ll} 2 \frac{c}{d} & \text{if } \frac{c}{d} \leq 1.0 \\ \min \left| \begin{array}{l} 1 + c/d \\ 3.0 \end{array} \right. & \text{if } \frac{c}{d} > 1.0 \end{array} \right.$$



Single Cope

Double Coped Beam Flexural Strength

Double Coped Beam Flexural Strength

Specification Section F11 (Modified Equations)

$$\lambda \leq \lambda_p$$

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

$$\lambda_p < \lambda \leq \lambda_r$$

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

$$\lambda > \lambda_r$$

$$M_n = (1.9 E C_b / \lambda) S_x \leq M_p \quad (\text{Spec. Eqn. F11-4})$$

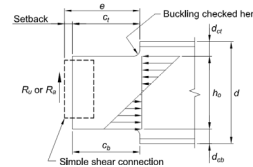
where

$$\lambda = c_t h_o / t_w^2$$

$$\lambda_p = 0.08 E / F_y$$

$$\lambda_r = 1.9 E / F_y$$

$$Z = t_w h_o^2 / 4$$



Double Coped Beam Flexural Strength

Local Web Buckling

Manual Page 9-9

If $c_b \geq c_t$

$$L_b = c_t$$

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

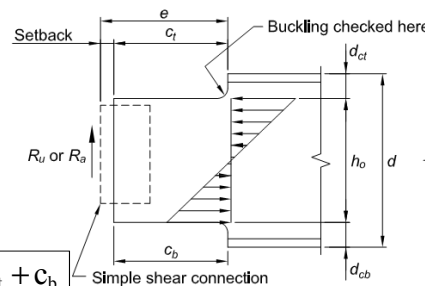
Otherwise

$$L_b = \frac{c_t + c_b}{2}$$

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

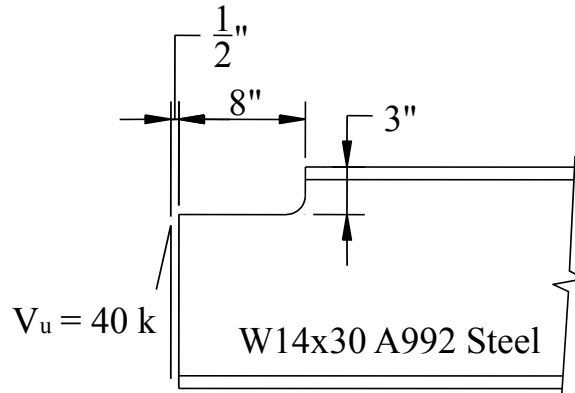
Note: Manual Page 9-9 shows \leq . Should be \geq as shown. Errata forthcoming.

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



Single Cope Flexural Strength Example

Example: Determine if Coped Beam Flexural Strength is Adequate for $V_u = 40$ k.



Coped Beam Flexural Strength Example

W14x30 A992

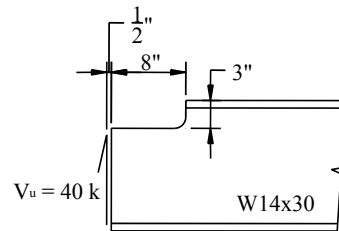
$$d = 13.8 \text{ in.} \quad t_w = 0.270 \text{ in.}$$

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

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

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

$$Z_{net} = 15.1 \text{ in}^3 \text{ from forthcoming AISC Design Ex.}$$

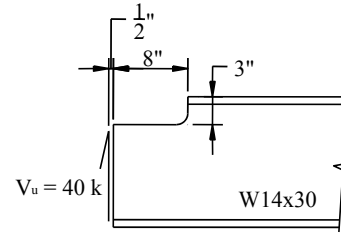


Note: The distance h_o above is not the same as h_o that is tabulated in *Manual Table 1-1 W-Shapes Dimensions*.

Coped Beam Flexural Strength Example

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

Shape	d _f , in.	t _f , in.	S _x , in. ³	S _o , in. ³	S _{net} , in. ³									
					d _c , 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				
×120	14.5	0.940	190	34.2	25.5	21.7	18.1	14.8	11.8	9.20				
×109	14.3	0.860	173	30.0	22.3	18.9	15.7	12.8	10.2	7.91				
×99	14.2	0.780	157	27.2	20.2	17.0	14.2	11.5	9.15	7.04				
×90	14.0	0.710	143	24.9	18.0	15.2	12.6	10.2	8.07	6.18				
W14x82	14.3	0.855	123	26.0	20.9	17.7	14.8	12.1	9.64	7.46				
×74	14.2	0.785	112	24.4	18.2	15.4	12.8	10.4	8.31	6.40				
×68	14.0	0.720	103	22.2	16.5	13.9	11.6	9.41	7.46	5.72				
×61	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					
×48	13.8	0.595	70.2	17.3	12.8	10.8	8.93	7.23	5.71					
×43	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				
×34	14.0	0.455	48.6	14.4	10.3	8.74	7.22	6.22	4.95					
×30	13.8	0.385	42.0	13.2	9.8	8.37	6.96	5.68	4.51					
W14x26	13.9	0.420	35.3	12.3	9.20	7.80	6.50	5.31	4.23					
×22	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			
×305	16.3	2.71	435	108	—	71.4	61.0	51.4	42.7	34.9	28.0			
×279	15.9	2.47	393	96.1	—	63.1	53.5	44.8	36.9	29.8				
×252	15.4	2.25	353	83.7	—	54.2	45.7	38.0	31.0	24.8				
×230	15.1	2.07	321	74.2	—	47.5	39.9	32.9	26.7	21.1				

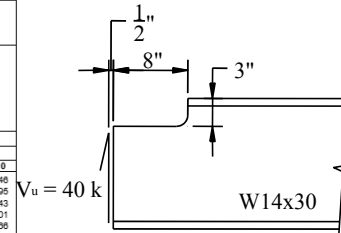


$S_{net} = 8.37 \text{ in}^3$

Coped Beam Flexural Strength Example

Table IV-11 (continued)
Plastic Section Modulus for Coped W-Shapes

Shape	d _f , in.	t _f , in.	Z _x , in. ³	Z _o , in. ³	Z _{net} , in. ³									
					d _c , in.									
					2	3	4	5	6	7	8	9	10	
W14x873	23.6	6.51	2030	916	—	—	—	—	—	532	480	432	387	348
×808	22.8	5.12	1830	817	—	—	—	—	—	483	419	372	332	295
×730	22.4	4.91	1860	869	—	—	—	—	—	421	380	341	308	273
×665	21.6	4.52	1480	579	—	—	—	—	—	388	321	287	259	227
×606	20.9	4.16	1320	503	—	—	—	—	—	308	272	242	214	186
×550	20.2	3.82	1180	434	—	—	—	—	—	289	258	229	203	179
×500	19.6	3.50	1050	379	—	—	—	—	—	248	221	195	172	150
×455	19.0	3.21	936	331	—	—	—	—	—	213	189	168	145	126
×428	18.7	3.04	869	301	—	—	—	—	—	193	170	149	130	113
×398	18.3	2.85	801	273	—	—	—	—	—	165	142	122	105	90.0
×370	17.9	2.66	736	246	—	—	—	—	—	174	154	134	117	101
×342	17.5	2.47	672	219	—	—	—	—	—	154	135	118	102	87.6
×311	17.1	2.28	603	193	—	—	—	—	—	134	117	102	87.5	74.6
×283	16.7	2.07	542	169	—	—	—	—	—	117	101	87.5	74.9	63.5
×257	16.4	1.89	487	150	117	102	88.8	78.3	64.9	54.7	46.6			
×233	16.0	1.72	436	130	101	87.9	75.9	64.8	54.8	45.9	37.9			
×211	15.7	1.56	390	115	88.9	77.1	68.2	58.3	47.3	39.3				
×193	15.5	1.44	355	103	78.8	68.1	58.3	48.4	41.4	34.2				
×178	15.2	1.31	320	92.2	70.3	60.8	51.8	43.5	36.2	29.7				
×159	15.0	1.19	287	81.0	61.5	52.8	44.9	37.0	31.2	25.4				
×145	14.8	1.09	260	72.2	54.5	46.7	39.8	33.1	27.2	22.1				
W14x53	13.9	0.660	87.1	34.2	25.1	21.1	17.4	14.2	11.2					
×48	13.8	0.595	78.4	31.1	22.7	19.1	15.7	12.7	10.1					
×43	13.7	0.530	69.6	27.6	20.1	16.9	13.9	11.2	8.83					
W14x38	14.1	0.515	61.5	29.1	21.7	18.3	15.1	12.3	9.77	7.84				
×34	14.0	0.455	54.6	26.3	19.7	16.5	13.7	11.1	8.78	6.75				
×30	13.8	0.385	47.3	23.8	17.6	15.1	12.5	10.1	7.91					



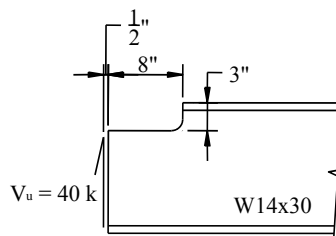
$Z_{net} = 15.1 \text{ in}^3$

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 Compact Web:

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

Plate Buckling Coefficient, k:

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

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

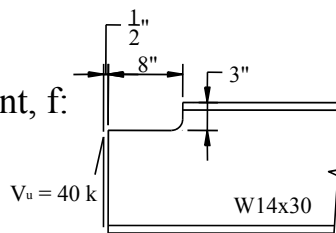
Coped Beam Flexural Strength Example

Local Web Flexural Strength

Modified Plate Buckling Coefficient, 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 :

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

Coped Beam Flexural Strength Example

Local Web Nominal Flexural Strength

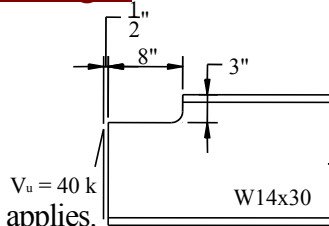
$$\begin{aligned}\lambda_p &= (0.475)\sqrt{k_1 E / F_y} \\ &= (0.475)\sqrt{(4.19)(29,000 / 50)} \\ &= 23.4\end{aligned}$$

$$\lambda = 40 < 2\lambda_p = 46.8 \text{ so Manual Eqn. 9-7 applies.}$$

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

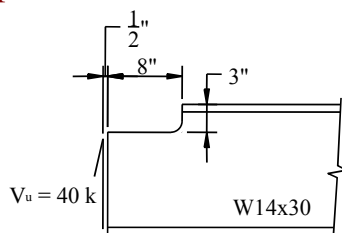
$$M_y = F_y S_{net} = (50)(8.37) = 419 \text{ kip-in.}$$

$$\begin{aligned}M_n &= M_p - (M_p - M_y)(\lambda / \lambda_p - 1) \leq 1.6M_y \quad (\text{Manual Eqn. 9-7}) \\ &= 755 - (755 - 419)(40 / 24.8 - 1) \\ &= 549 \text{ kip-in.}\end{aligned}$$



Coped Beam Flexural Strength Example

Single Coped Beam Flexural Strength



$$\phi M_n = 0.9(549 \text{ kip-in.}) = 494 \text{ kip-in.}$$

$$M_u = (40 \text{ kips})(8.5 \text{ in.})$$

$$= 340 \text{ kip-in.} \leq \phi M_n = 494 \text{ kip-in.} \quad \text{Adequate}$$

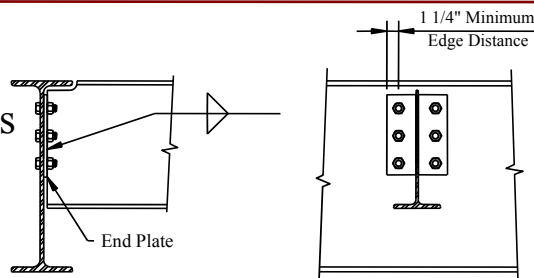
SHEAR END-PLATE CONNECTIONS



Shear End-Plate Connections

Advantages:

- Simple – Few Parts
- No Holes in Beam



Disadvantages:

- Requires Beam to be Cut to Exact Length

Note : End Plate Thickness Range is 1/4" to 3/8"

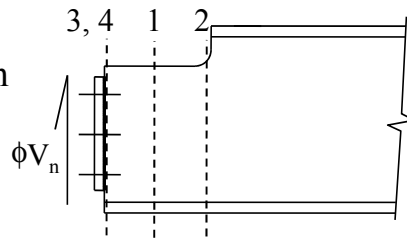
Shear End-Plate Connection Limit States

Beam:

1. Gross Shear Yielding
2. Coped Beam Flexural Strength
3. Web Shear Rupture Strength at Weld

Weld:

4. Weld Rupture Strength



Shear End-Plate Connection Limit States

Plate:

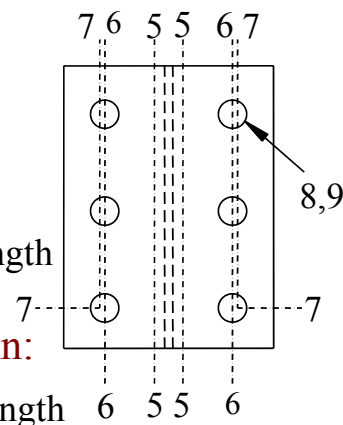
5. Gross Shear Yielding
6. Net Shear Rupture
7. Block Shear Strength

Shear Transfer at the Elements

8. Bearing and Tear-Out Strength
7. Bolt Shear Rupture

Supporting Girder or Column:

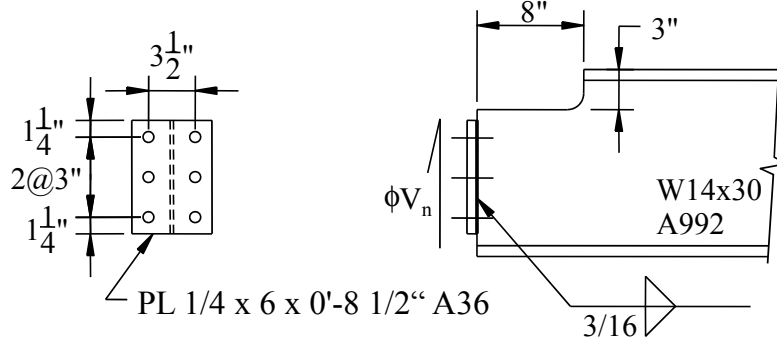
9. Bearing and Tear-Out Strength



Shear End-Plate Connection Example

Example: Determine the design strength, ϕV_n .

3/4" A325-N Bolts, E70XX Electrode



Assume thickness of supporting girder web = 0.5 in. W14x30 $F_y = 50$ ksi $F_u = 65$ ksi
 $d = 13.8$ in. $t_w = 0.27$ in.

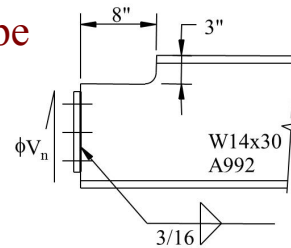
Shear End-Plate Connection Example

1. Gross Shear Yielding at Cope

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

$$d_{ct} = 3.0 \text{ in.}$$

$$\begin{aligned} \phi V_n &= 1.0 (0.6 F_y) (d - d_{ct}) t_w \\ &= 1.0 (0.6 \times 50) (13.8 - 3.0) (0.27) \\ &= \underline{87.5 \text{ k}} \end{aligned}$$

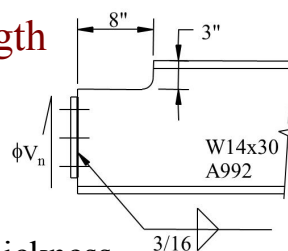


Shear End-Plate Connection Example

2. Coped Beam Flexural Strength

From previous example

$$\phi M_n = 494 \text{ in.-kips}$$



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

$$\begin{aligned} \phi V_n &= 494 / 8.25 \\ &= \underline{59.9 \text{ k}} \end{aligned}$$

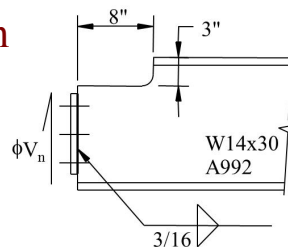
Shear End-Plate Connection Example

3. Web Shear Rupture Strength at Weld

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

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

Beam Web $t_w = 0.27 \text{ in.}$

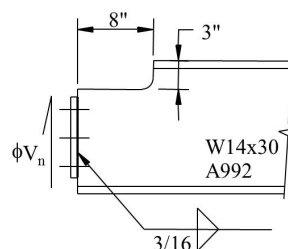


$$\begin{aligned} \phi V_n &= 0.75 (0.6 F_u) (L - 2 t_{\text{weld}}) t_w \\ &= 0.75 (0.6 \times 65) [8.5 - (2 \times 3/16)] (0.27) \\ &= \underline{64.2 \text{ k}} \end{aligned}$$

Shear End-Plate Connection Example

4. Weld Rupture Strength

Minimum Weld Size = 1/8"
 from *Manual* Table J3.4. OK
 E70xx Electrode



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

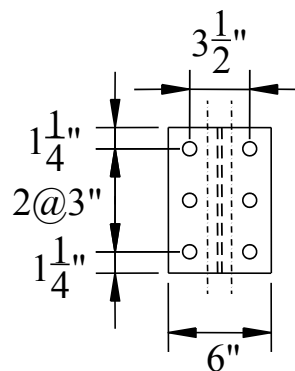
Shear End-Plate Connection Example

Plate Limit States:

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

A36 Steel:

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



5. Gross Shear Yielding

$$\begin{aligned}\phi V_n &= 1.0 (0.6 F_y) (2 L t_p) \\ &= 1.0 (0.6 \times 36) (2 \times 8.5 \times 1/4) \\ &= \underline{91.8 \text{ k}}\end{aligned}$$

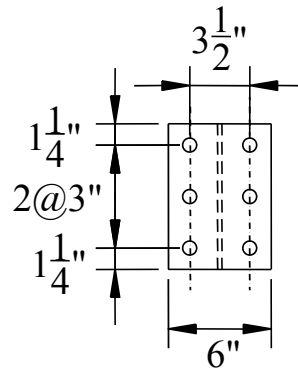
Shear End-Plate Connection Example

6. Net Shear Rupture

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

$$A_n = (8.5 - 3 \times 7/8) (1/4)(2) \\ = 2.94 \text{ in.}^2$$

$$\phi V_n = 0.75 (0.6 F_u) (A_n) \\ = 0.75 (0.6 \times 58) (2.94) \\ = \underline{76.7 \text{ k}}$$



Shear End-Plate Connection Example

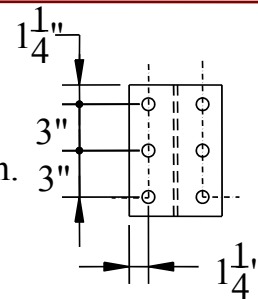
7. Block Shear Strength

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

$$d = 3/4 \text{ in. } d_h = 13/16 \text{ in. } d_h' = 7/8 \text{ in.}$$

$$\phi = 0.75$$

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



Shear End-Plate Connection Example

7. Plate Block Shear

- Shear Yielding

$$0.6F_y A_{gv} = (0.6 \times 36)(2 \times 0.25 \times 7.25) = 78.3 \text{ k}$$

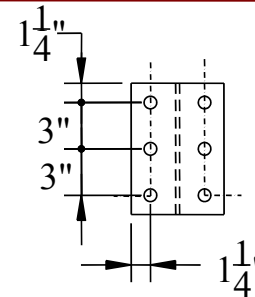
- Shear Rupture

$$0.6F_u A_{nv} = (0.6 \times 58)(7.25 - 2.5 \times 7/8)(2 \times 1/4) = 88.1 \text{ k}$$

Shear Yielding Controls

- Tension Rupture

$$F_u A_{nt} = 58(1.25 - 0.5 \times 7/8)(2 \times 1/4) = 23.6 \text{ k}$$



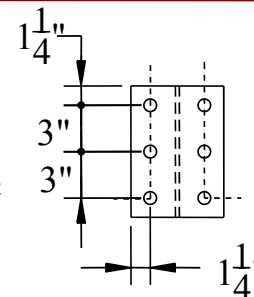
Shear End-Plate Connection Example

7. Plate Block Shear

$$R_n = \min \begin{cases} \text{Shear Yield} \\ \text{Shear Rupture} \end{cases} + U_{bs} F_u A_{nt}$$

$$= \min \begin{cases} 78.3 \\ 88.1 \end{cases} + 1.0 \times 23.6 = 101.9 \text{ k}$$

$$\phi V_n = 0.75 \times 101.9 = \underline{76.4 \text{ k}}$$



Shear End-Plate Connection Example

8. Shear Transfer Between Elements

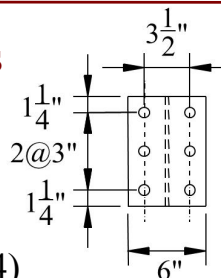
- Bearing/Tear Out

Since the girder web $t_w = 0.5$ in., 0.25 in. plate Bearing/Tear Out will control.

$$\begin{aligned} \text{Brg: } 2.4 F_u d_b t &= (2.4 \times 58) (3/4 \times 1/4) \\ &= 26.1 \text{ k} \end{aligned}$$

$$\begin{aligned} \text{Edge: } 1.2 F_u L_c t &= (1.2 \times 58) (1.25 - 13/32) (1/4) \\ &= \underline{14.7 \text{ k}} < 26.1 \text{ k} \end{aligned}$$

$$\begin{aligned} \text{Other: } 1.2 F_u L_c t &= (1.2 \times 58) (3 - 13/16) (1/4) \\ &= 38.1 \text{ k} > \underline{26.1 \text{ k}} \end{aligned}$$



Shear End-Plate Connection Example

8. Shear Transfer Between Elements

- Bolt Shear Rupture

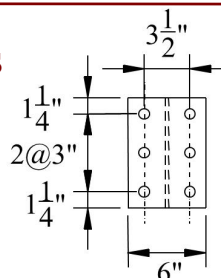
$$3/4'' \text{ A325-N } F_{nv} = 54 \text{ ksi}$$

$$r_n = (54 \text{ ksi})(0.4418 \text{ in}^2) = 23.8 \text{ k}$$

- Design Shear Transfer Strength

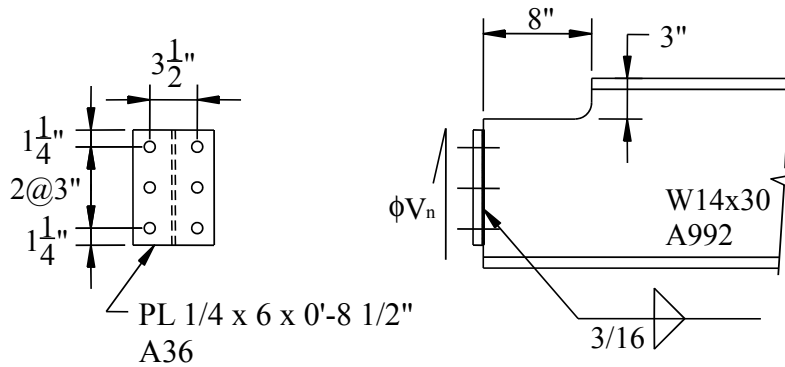
$$\begin{aligned} V_n &= 2[\text{min. (Edge B/T.O., Bolt Shear Rupture)}] \\ &\quad 4[\text{min. (Other B/T.O., Bolt Shear Rupture)}] \\ &= 2[\text{min. (14.7, 23.8)}] + 4[\text{min. (26.1, 23.8)}] \\ &= 125 \text{ k} \end{aligned}$$

$$\phi V_n = 0.75 \times 125 = \underline{93.4 \text{ k}}$$



Shear End-Plate Connection Example

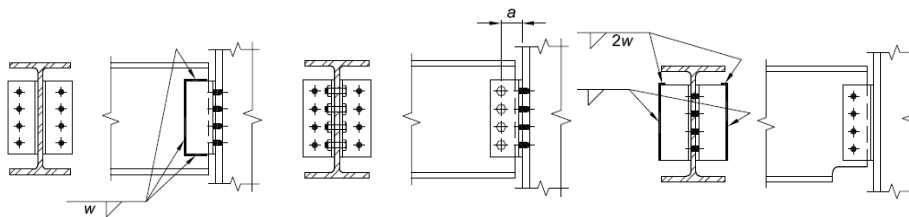
CONNECTION DESIGN STRENGTH



Coped Beam Flexural Strength Controls

$$\phi V_n = 59.9 \text{ k}$$

DOUBLE ANGLE CONNECTIONS



Welded/Bolted

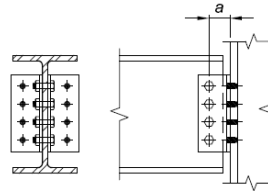
Bolted/Bolted

Bolted/Welded

Double Angle Connections

Advantages:

- Beam Length can Vary
- Weld or Bolt to Beam



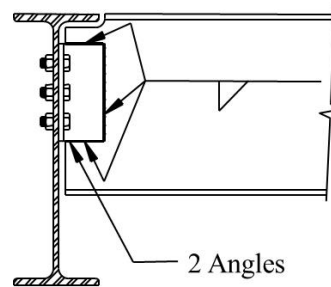
Disadvantages:

- Double Sided Connections into Column Webs are an Erection Problem
- “Shared” Bolts are an Erection Safety Issue

Welded/Bolted Double Angle Connections

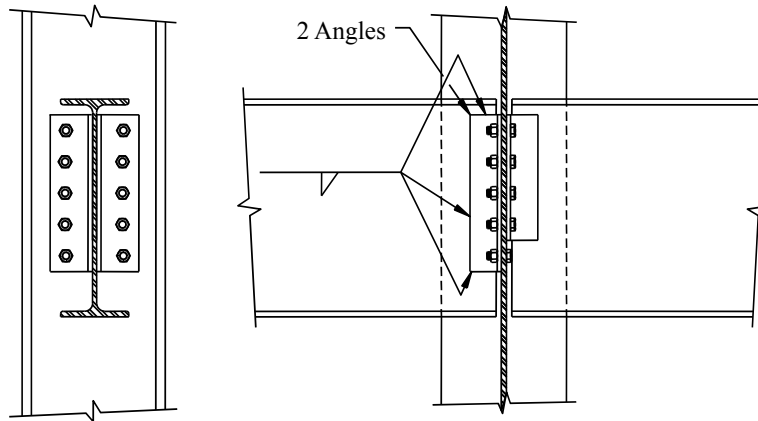


Beam Dropped for Joist Seat



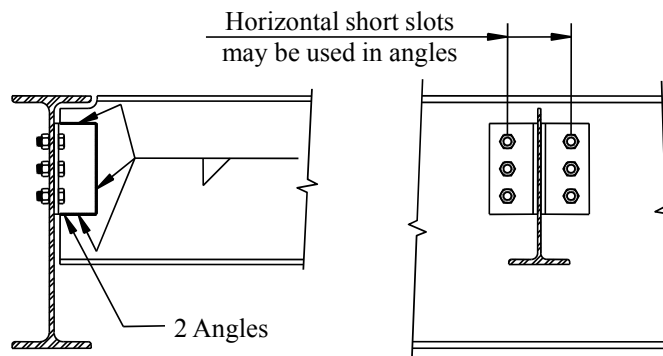
Coped Beam

Welded/Bolted Double Angle Connections



Double Sided Connection into Column Web

Welded/Bolted Double Angle Connections



Pin is at face of supporting element

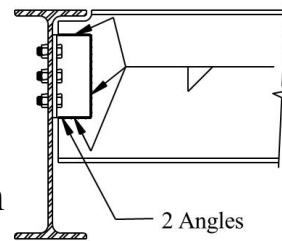
Beam web weld is subjected to eccentric shear.

Welded/Bolted Double Angle Connections

Limit States

Beam:

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



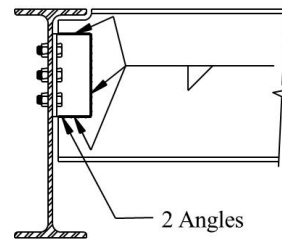
Weld:

- Weld Rupture Due to Shear Plus Torsion

Welded/Bolted Double Angle Connections

Angles:

- Gross Shear
- Net Shear
- Block Shear
- Angle Strength at Weld



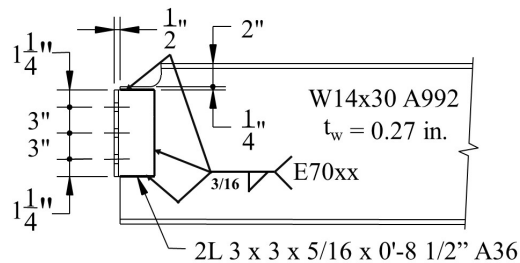
Shear Transfer Between Elements:

- Angle Bearing/Tear Out
- Bolt Shear Rupture
- Supporting Element Bearing/Tear Out

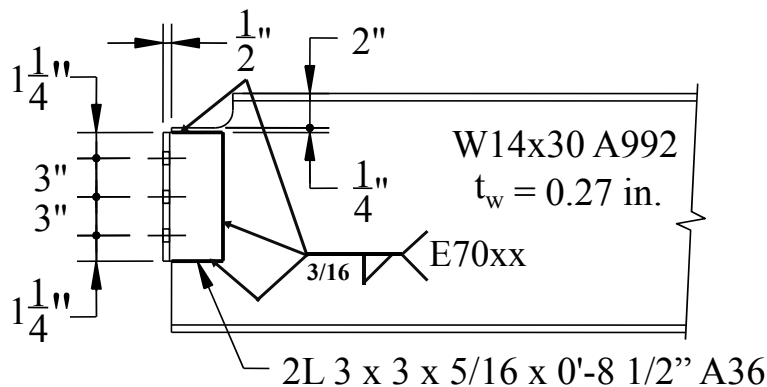
Welded/Bolted Double Angles Example

Example: Determine ϕV_n for the limit states of:

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



Welded/Bolted Double Angles Example



A992	$F_y = 50$ ksi	$F_u = 65$ ksi
A36	$F_y = 36$ ksi	$F_u = 58$ ksi

Welded/Bolted Double Angles Example

1. Beam Web Block Shear

- **Shear Rupture**

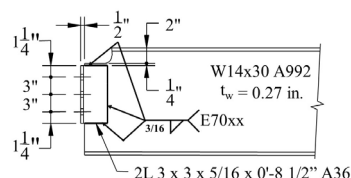
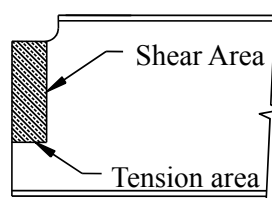
Never controls when angles are welded to the beam web.

- **Shear Yielding**

$$0.6 F_y A_{gv} = 0.6 (50) (8.5 + 0.25)(0.27) = 70.9 \text{ k}$$

- **Tension Rupture**

$$F_u A_{nt} = (65) (3 - 1/2 - 1/4) (0.27) = 39.5 \text{ k}$$

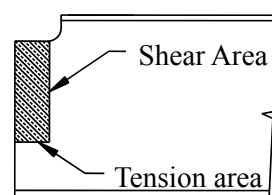


Welded/Bolted Double Angles Example

1. Beam Web Block Shear

- **Beam Web Block Shear Strength**

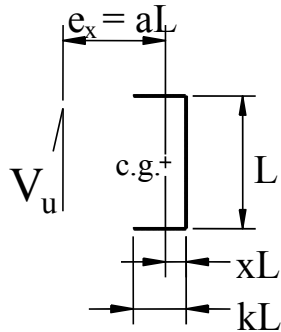
$$\begin{aligned} \phi V_n &= 0.75 (\text{min shear} + U_{bs} \text{ tension rupture}) \\ &= 0.75 (70.9 + 1.0 \times 39.5) \\ &= \underline{82.8 \text{ k}} \end{aligned}$$



$$\phi V_n = \underline{82.8 \text{ k}}$$

Welded/Bolted Double Angles Example

2. Weld Rupture Due to Eccentric Shear



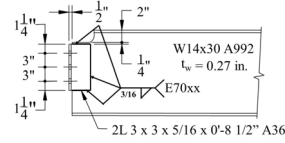
$$\phi = 0.75$$

$$V_n = C C_1 D l$$

C = effective weld coefficient
 from *Manual* Table 8-8

$$C_1 = F_u \text{ of weld metal} / 70$$

D = number of 1/16 ths



Needed for Table 8-8:
 k, x, a

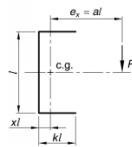
Welded/Bolted Double Angles Example

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

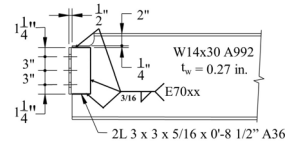
LRFD				ASD			
$C_{min} = \frac{P_u}{\phi C_1 D l}$	$D_{min} = \frac{P_u}{\phi C C_1 l}$	$l_{min} = \frac{P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_u}{C_1 D l}$	$D_{min} = \frac{\Omega P_u}{C C_1 l}$	$l_{min} = \frac{\Omega P_u}{C C_1 D}$		

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
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



$\phi = 0.75$
 $R_n = C C_1 D l$
 Parameters:
 $C_1 = E_{xx}/70$
 $k \Rightarrow x$
 $x \Rightarrow a$
 $x \ \& \ a \Rightarrow C$



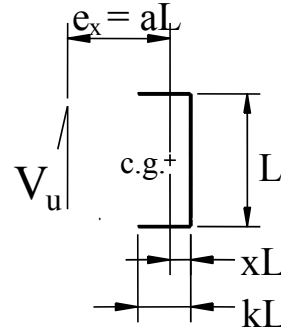
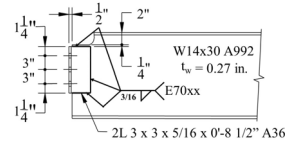
Welded/Bolted Double Angles Example

Determine C from Table 8-8:

2-L3x3x5/16 x 0'-8 1/2"

$L = 8.5$ in.

$k = (3 - 0.5 - 0.25) / 8.5 = 0.26$



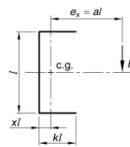
Welded/Bolted Double Angles Example

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
 $R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

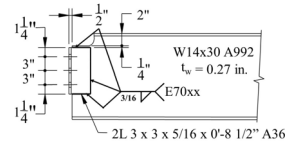
LRFD				ASD			
$C_{min} = \frac{P_u}{\phi C_1 D l}$	$D_{min} = \frac{P_u}{\phi C C_1 l}$	$l_{min} = \frac{P_u}{\phi C C_1 D}$		$C_{min} = \frac{\Omega P_u}{C_1 D l}$	$D_{min} = \frac{\Omega P_u}{C C_1 l}$	$l_{min} = \frac{\Omega P_u}{C C_1 D}$	

where
 P_u = required force, P_u or P_u , 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	0.344	0.417	0.500	0.591	0.686	0.784	0.889	1.00	1.12	1.37	1.66	1.97	2.31	2.68			
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800			



$R_n = CC_1Dl$
 $\phi = 0.75$
 Parameters:
 $C_1 = 1.0$
 $k = 0.26$
 $\Rightarrow x = 0.0465$
 $x \Rightarrow a$
 $x \ \& \ a \Rightarrow C$



Welded/Bolted Double Angles Example

Determine C from Table 8-8:

2-L3x3x5/16 x 0'-8 1/2"

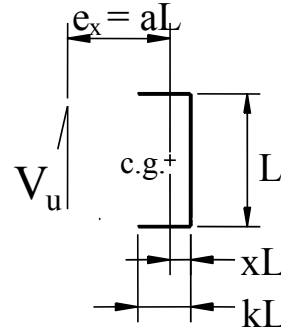
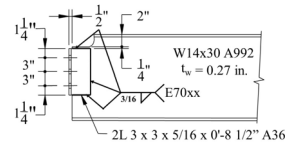
$L = 8.5$ in.

$k = (3 - 0.5 - 0.25) / 8.5 = 0.26$

$\Rightarrow x = 0.0465$

$xL = 0.0465 \times 8.5 = 0.40$ in.

$a = (3.0 - 0.40) / 8.5 = 0.31$



Welded/Bolted Double Angles Example

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

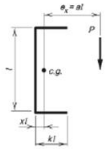
Available strength of a weld group, ϕR_n or R_n/Ω , is determined with
 $R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

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

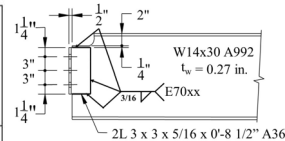
where

- P = required force, P_u or P_u/ϕ , 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).



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
0.30	1.55	1.95	2.36	2.79	3.23	3.68	4.14	4.60	5.08	5.55	6.03	7.01	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	6.30	7.24	8.19	9.16	10.1



$$R_n = CC_1Dl$$

$$\phi = 0.75$$

Parameters:

$$C_1 = 1.0$$

$$k = 0.26$$

$$\Rightarrow x = 0.0465$$

$$\Rightarrow a = 0.31$$

$$x \text{ \& \ } a \Rightarrow C = 2.62$$

Welded/Bolted Double Angles Example

Using *Manual* Table 8-8

$$\Rightarrow C = 2.62$$

With $C_1 = 1.0$

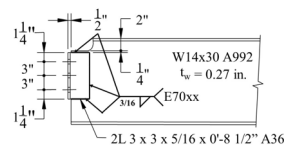
$$D = 2 \text{ (Two Ls)} \times 3$$

Weld Rupture Strength

$$\phi V_n = \phi C C_1 D L$$

$$= (0.75)(2.62) (1.0) (2 \times 3) (8.5)$$

$$= \underline{100 \text{ k}}$$



$$\phi V_n = \underline{100 \text{ k}}$$

Welded/Bolted Double Angles Example

3. Beam Web Strength at Weld

$$\phi V_n = \frac{\text{Design Rupture Strength of Plate/in.}}{\text{Design Rupture Strength of Weld/in.}} \times$$

Design Connection Weld Rupture Strength, k

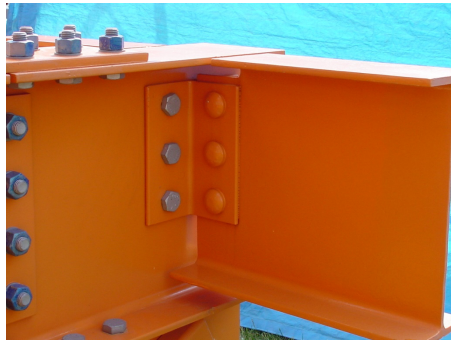
$$= \frac{\phi 0.6 F_u t_w (1.0)}{(1.392) (2 \times 3) (1.0)} \times 100$$

$$= \frac{0.75 (0.6 \times 65) (0.27) (1.0)}{(1.392) (2 \times 3) (1.0)} \times 100$$

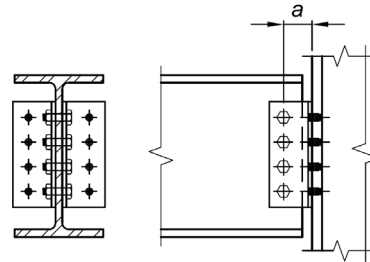
$$= \underline{94.8 \text{ k}}$$

$$\phi V_n = \underline{94.8 \text{ k}}$$

Bolted/Bolted Double Angle Connections



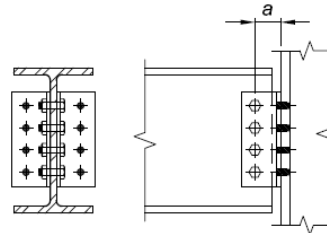
Beam to Girder



Beam to Column Flange

Bolted/Bolted Double Angle Connections

Bolt eccentricity ignored
in bolted/bolted double
angle connections.

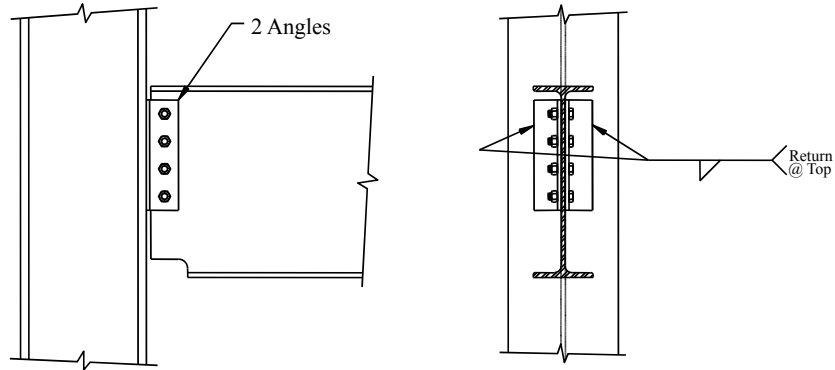


Shear transfer between angles and beam web and
angles and supporting element as previous:

Min (Bearing/Tear Out & Bolt Shear Rupture)
at each hole/bolt.

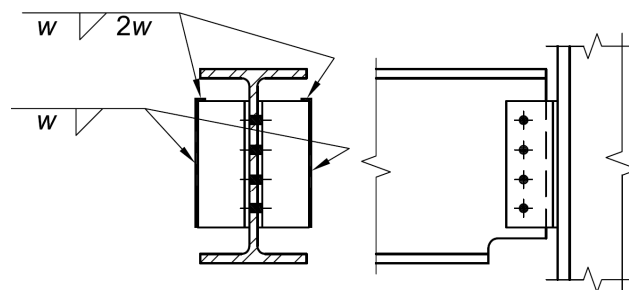
No New Limit States

Double Angle Knife Connections



Bolted/Welded Double Angle Knife Connection

Double Angle Knife Connections



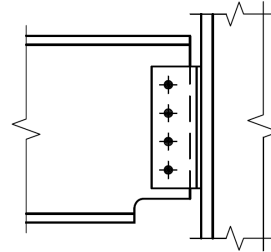
Note: Weld returns on top of angles
per *Specification* Section J2.2b

Bolted/Welded to Column Flange

Bolted/Welded Double Angles

Bolted to Beam / Welded to Column Flange

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



New Limit States:

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

Bolted/Welded Double Angles

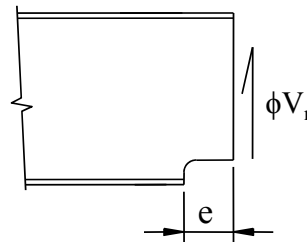
Coped Beam Web Strength at Tension Flange

$$\phi_b = 0.9$$

$$V_n = F_y S_{net} - e M_n / e$$

where

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

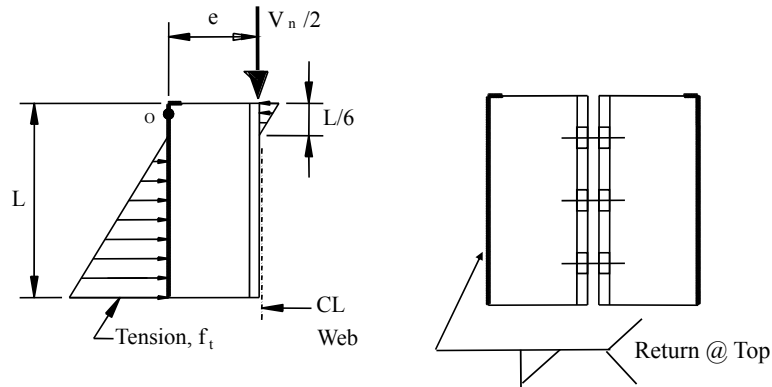


S_{net} = elastic section modulus from *Manual* Table 9-2

Z_{net} = plastic section modulus from AISC Design Ex.

Bolted/Welded Double Angles

Weld Strength on OSLs



Bolted/Welded Double Angles

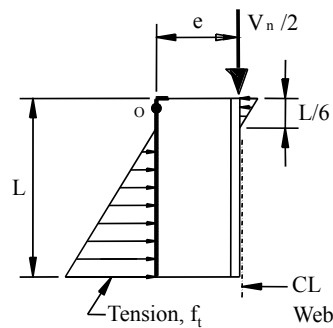
Weld Strength on OSLs

$$f_v = \frac{(V_n/2)}{L}$$

$$\sum M_o = 0$$

$$\frac{1}{2} f_t \left(\frac{5}{6} L \right) \left(\frac{2}{3} L \right) = \frac{V_n}{2} e$$

$$f_t = 1.8 \frac{V_n e}{L^2}$$



Bolted/Welded Double Angles

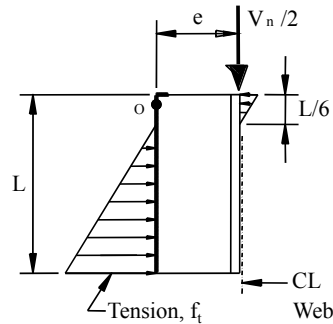
Weld Strength on OSLs

$$f_w = \sqrt{f_t^2 + f_v^2}$$

$$= \frac{V_n}{2L^2} \sqrt{L^2 + 12.96 e^2}$$

with $\phi f_w = 1.392D$

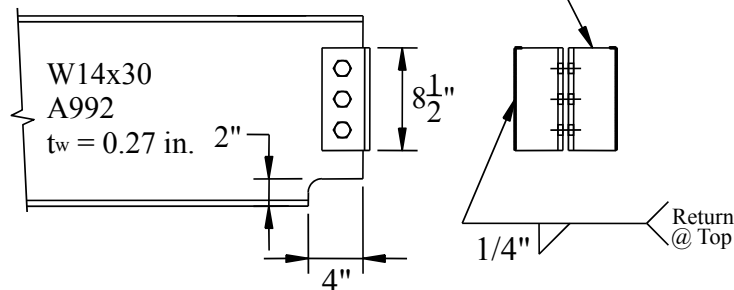
$$\phi R_n = 2 \left(\frac{1.392 DL}{\sqrt{1 + 12.96 e^2 / L^2}} \right) \quad (\text{Manual p.10-11})$$



Bolted/Welded Double Angles Example

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

2L 3 x 3 x 5/16 x 0'-8 1/2" A36



$$e = 3 + 0.27/2$$

$$= 3.14 \text{ in.}$$

L = 8.5 in.

3/4" A325-N Bolts
 E70XX

Bolted/Welded Double Angles Example

Weld design rupture strength at OSLs:

$$\begin{aligned}\phi V_n &= \frac{2 L^2 (1.392 D)}{\sqrt{L^2 + 12.96 e^2}} \\ &= \frac{2 (8.5)^2 (1.392 \times 4)}{\sqrt{8.5^2 + 12.96 \times (3.14)^2}} \\ &= \underline{56.9 \text{ k}}\end{aligned}$$

Note: Weld returns ($2t_w$) at top of angles have been neglected.

End of Session 3

Thank You for
Attending

Next Up

Next Session

- October 24, 2017 Shear Connections Part II

TOPICS

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

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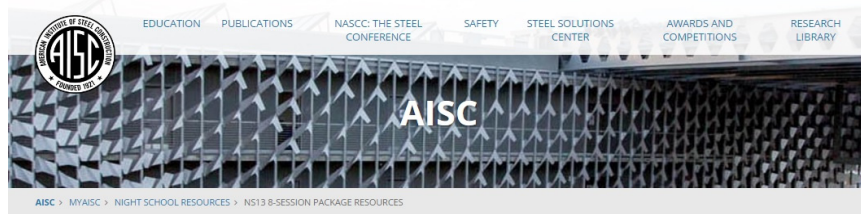
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Course Resources

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

Night School Resources for 8-session package Registrants



Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	View Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Design	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	

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



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