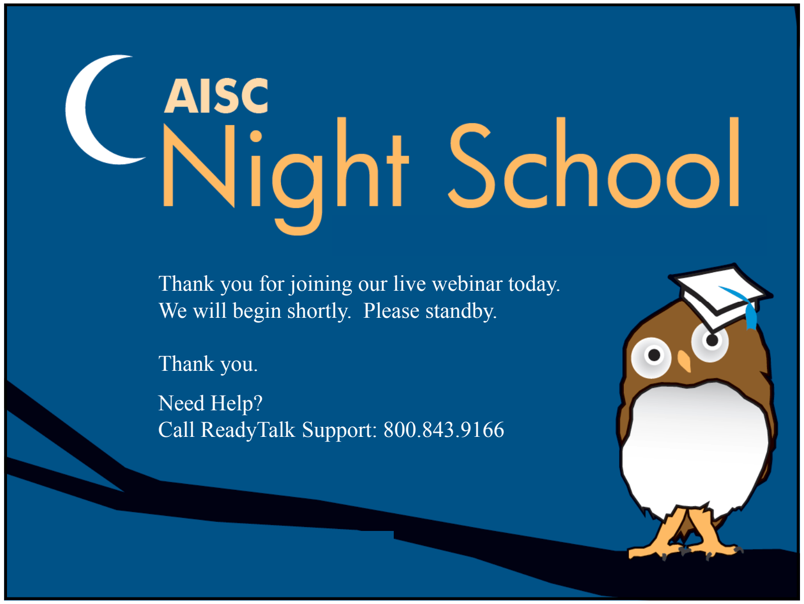

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Course Description

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

structural STEEL

SCHEDULE

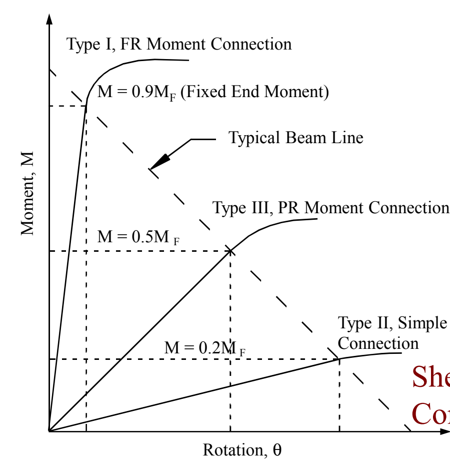
- October 03, 2017 — Fundamental Concepts Part I
- October 10, 2017 — Fundamental Concepts Part II
- **October 17, 2017 — Shear Connections Part I**
- **October 24, 2017 — Shear Connections Part II**
- **November 07, 2017 — Moment Connections Part I**
- **November 14, 2017 — Moment Connections Part II**
- **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



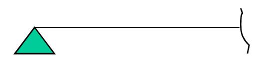
Shear or Framing Connections

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

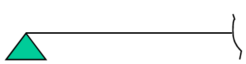
Design Considerations

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



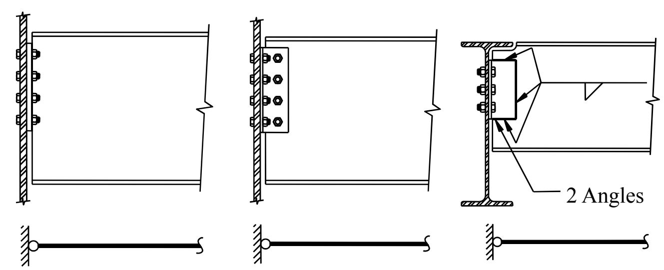
Design Considerations

- 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 ≤ 5/8 in.
  - Wide gage
  - Wide vertical weld spacing
- Stability Consideration
  - Depth of Connection ≥ T/2

    (T is clear distance between fillets – Tabulated in *Manual* Table 1-1)

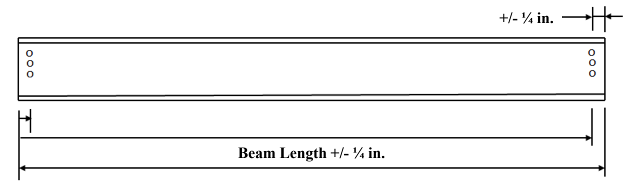
Design Considerations

- Beam Length Tolerance +/- 1/4 in.

  For design:

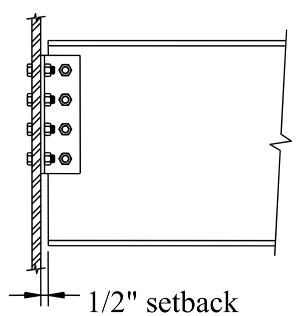
  Setbacks in calculations are usually 1/2 in.

  End edge distances are taken in calcs 1/4 in. less than detailed.



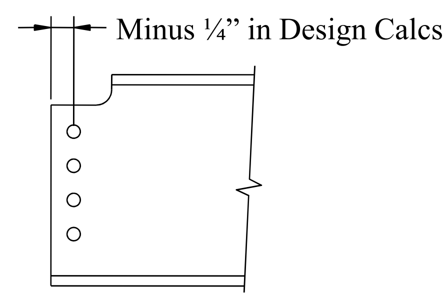
Design Considerations

- Beam Length Tolerance +/- 1/4 in.



Design Considerations

- Beam Length Tolerance +/- 1/4 in.

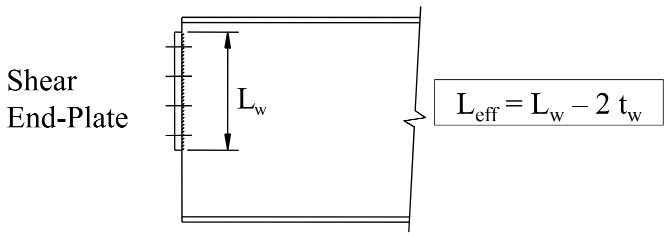


Lecturer: Tom Murray, P.E., PhD
Emeritus Professor, Virginia Tech

Design Considerations

- Effective Weld Length

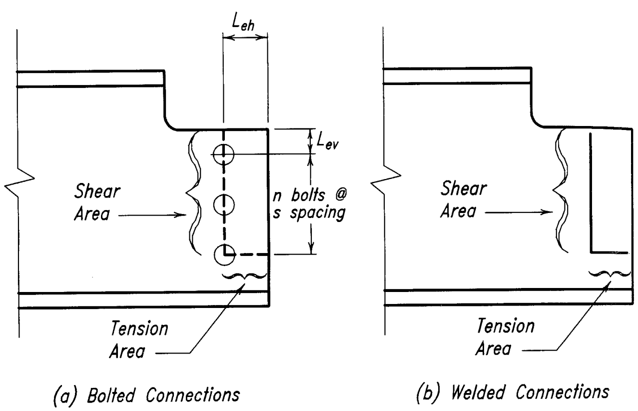
  When a weld terminates in the "air", the dimensioned weld length is reduced by the weld size for calculations <u>except</u> for angles welded to a beam web.



New Limit States

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

Block Shear in Coped Beams



Block Shear in Coped Beams

<u>Block Shear Strength</u>

Specification Section J4.3

$\phi = 0.75$

$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$

$\leq 0.6F_yA_{gv} + U_{bs}\ F_uA_{nt}$

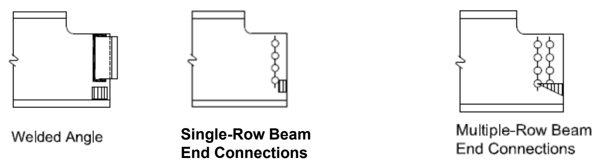
U_{bs} = 1.0 when tension stress is uniform

= 0.5 otherwise

Equivalent to:

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

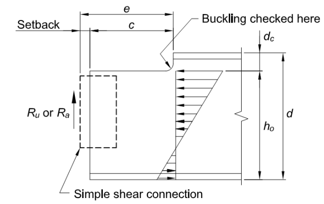
Block Shear in Coped Beams



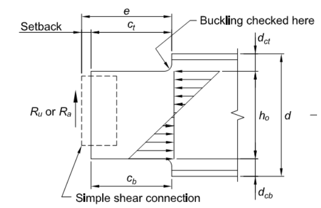
$U_{bs} = 1.0$ $U_{bs} = 0.5$

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

Coped Beam Flexural Strength



Single Cope



Double Cope

Design Strength

$\phi_b = 0.9$

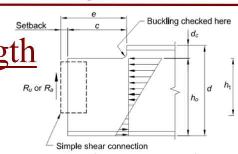
$M_u = R_u e \le \phi_b M_n$

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



$\lambda \le \lambda_p$

$M_n = M_p = F_y Z_{net}$ (Eqn.9-6)

$\lambda_p < \lambda \le 2\lambda_p$

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

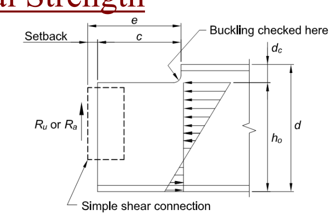
Z_{net} = net plastic section modulus

Single Coped Beam Flexural Strength

Single Coped Beam Flexural Strength

λ = web slenderness $= h_o/t_w$

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



Single Cope

$$k_1 = \max\begin{vmatrix} fk \\ 1.61 \end{vmatrix}$$

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

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

$$f = \begin{vmatrix} 2\dfrac{c}{d} & \text{if } \dfrac{c}{d} \le 1.0 \\ \min\begin{vmatrix} 1 + c/d \\ 3.0 \end{vmatrix} & \text{if } \dfrac{c}{d} > 1.0 \end{vmatrix}$$

Double Coped Beam Flexural Strength

Double Coped Beam Flexural Strength
Specification Section F11(Modified Equations)

$\lambda \le \lambda_p$

$M_n = M_p = F_y Z \le 1.6 M_y$ (*Spec.* Eqn. F11-1)

$\lambda_p < \lambda \le \lambda_r$

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

$\lambda > \lambda_r$

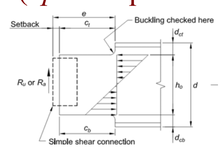
$M_n = (1.9EC_b/\lambda)S_x \le M_p$ (*Spec.* Eqn. F11-4)

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

$\lambda_p = 0.08E/F_y$

$\lambda_r = 1.9E/F_y$

$Z = t_w h_o^2/4$



Double Coped Beam Flexural Strength

Local Web Buckling
Manual Page 9-9

If $c_b \ge c_t$

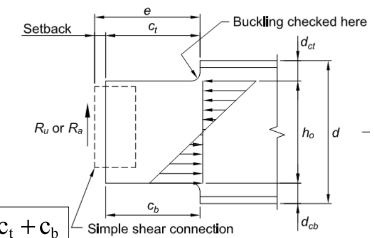
$L_b = c_t$

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

Otherwise

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

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

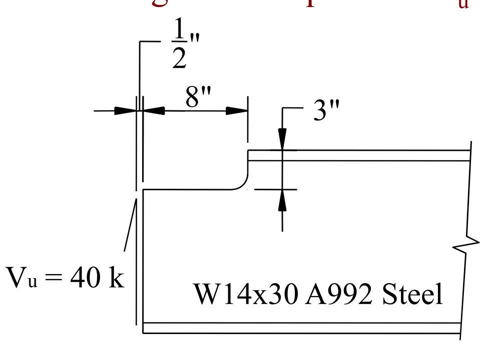


Note: Manual Page 9-9 shows $\le$. Should be $\ge$ as shown. Errata forthcoming.

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

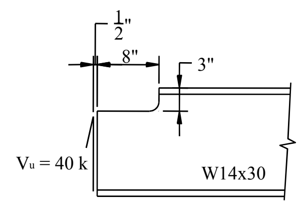
$d = 13.8$ in. $t_w = 0.270$ in.

$b_f = 6.73$ in. $t_f = 0.385$ in.

$h_o = 13.8 - 3.0 = 10.8$ in.

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

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

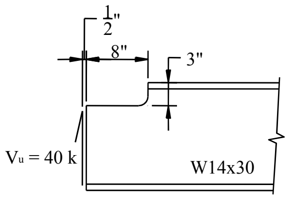


Note: The distance h_o above is not the same as h_o that is tabulated in *Manual* Table 1-1 *W-Shapes Dimensions*.

Coped Beam Flexural Strength Example

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

Shape	d, 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
W14×132	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	24.3	18.0	15.2	12.6	10.2	8.07	6.18			
W14×82	14.3	0.855	123	28.0	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				
W14×53	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				
W14×38	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	10.8	9.14	7.62	6.22	4.95				
×30	13.8	0.385	42.0	13.2	9.88	8.37	6.96	5.68	4.51				
W14×26	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				
W12×336	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			



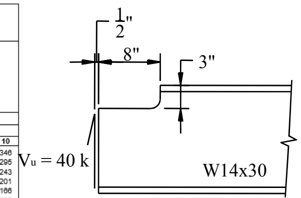
$S_{net} = 8.37\ in^3$

Coped Beam Flexural Strength Example

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

Shape	d, 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
W14×873	23.6	5.51	2030	916	–	–	–	–	532	480	432	387	346
×808	22.8	5.12	1830	817	–	–	–	–	463	416	372	332	295
×730	22.4	4.91	1660	669	–	–	–	421	380	341	306	273	243
×665	21.6	4.52	1480	579	–	–	–	358	321	287	256	227	201
×605	20.9	4.16	1320	503	–	–	–	305	272	242	214	189	166
×550	20.2	3.82	1180	434	–	–	289	258	229	203	179	157	137
×500	19.6	3.50	1050	379	–	–	248	221	195	172	150	131	
×455	19.0	3.21	936	331	–	–	213	189	166	145	126	109	
×426	18.7	3.04	869	301	–	–	193	170	149	130	113	97.0	
×398	18.3	2.85	801	273	–	195	172	152	132	115	99.0	84.8	
×370	17.9	2.66	736	246	–	174	154	134	117	101	86.5		
×342	17.5	2.47	672	219	–	154	135	118	102	87.6	74.7		
×311	17.1	2.26	603	193	–	134	117	102	87.5	74.6	63.1		
×283	16.7	2.07	542	169	–	117	101	87.5	74.9	63.5	53.3		
×257	16.4	1.89	487	150	117	102	88.8	76.3	64.9	54.7	45.6		
×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	66.2	56.3	47.3	39.3			
×193	15.5	1.44	355	103	78.8	68.1	58.3	49.4	41.4	34.2			
×176	15.2	1.31	320	92.2	70.3	60.6	51.6	43.5	36.2	29.7			
×159	15.0	1.19	287	81.0	61.5	52.8	44.9	37.6	31.2	25.4			
×145	14.8	1.09	260	72.2	54.5	46.7	39.6	33.1	27.2	22.1			
W14×53	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				
W14×38	14.1	0.515	61.5	29.1	21.7	18.3	15.1	12.3	9.77	7.54			
×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	17.9	15.1	12.5	10.1	7.91				

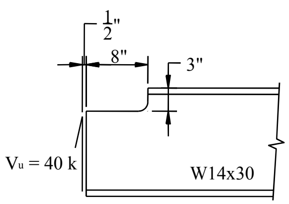
– Indicates that cope depth is less than flange thickness
Note: Values are omitted when cope depth exceeds d/2.



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

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:

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

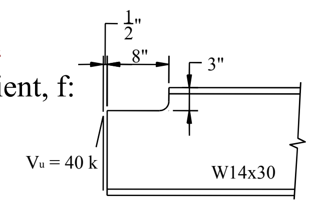
Plate Buckling Coefficient, k:

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

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

Coped Beam Flexural Strength Example

Local Web Flexural Strength



Modified Plate Buckling Coefficient, f:

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

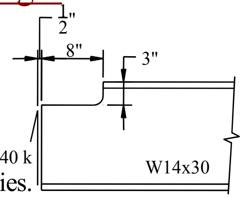
$$f = 2\frac{c}{d} = 2(0.580) = 1.16$$

Modified Plate Bending Coefficient, k_1:

$$k_1 = \max\left|\begin{array}{l} fk = (1.16)(3.61) = 4.19 \\ 1.61 \end{array}\right. = 4.19$$

Coped Beam Flexural Strength Example

Local Web Nominal Flexural Strength



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

$= (0.475)\sqrt{(4.19)(29{,}000/50)}$

$= 23.4$

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

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

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

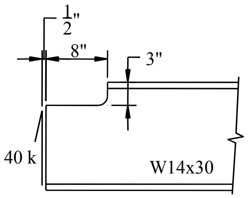
$M_n = M_p - (M_p - M_y)(\lambda/\lambda_p - 1)$ ~~$\le 1.6M_y$~~ (*Manual* Eqn. 9-7)

$= 755 - (755-419)(40/24.8 - 1)$

$= 549$ kip-in.

Coped Beam Flexural Strength Example

Single Coped Beam Flexual Strength



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

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

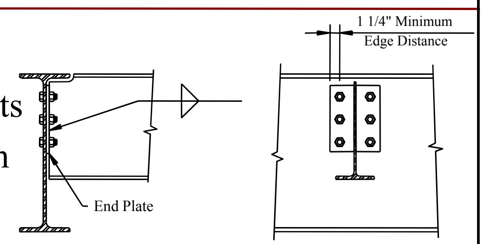
$= 340 \text{ kip-in.} \le \phi M_n = 494$ kip-in Adequate

SHEAR END-PLATE CONNECTIONS

Shear End-Plate Connections

Advantages:

- Simple – Few Parts
- No Holes in Beam



Disadvantages:

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

- Requires Beam to be Cut to Exact Length

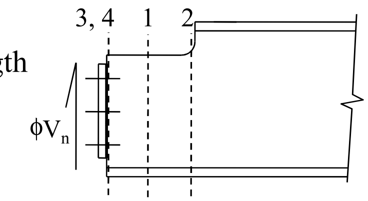
Shear End-Plate Connection Limit States

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:

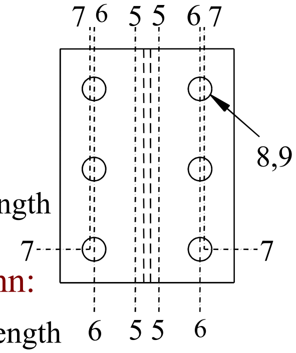
5. Gross Shear Yielding
6. Net Shear Rupture
7. Block Shear Strength

Shear Transfer at the Elements

8. Bearing and Tear-Out Strength
   Bolt Shear Rupture

Supporting Girder or Column:

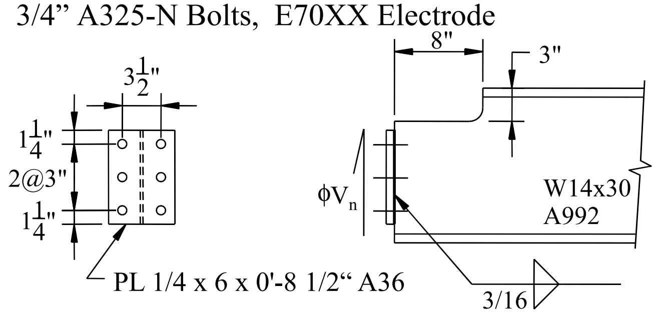
9. Bearing and Tear-Out Strength



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.

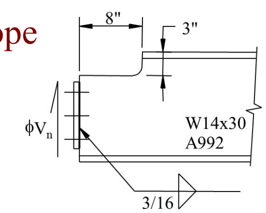
W14x30 F_y = 50 ksi F_u = 65 ksi
d = 13.8 in. t_w = 0.27 in.

Shear End-Plate Connection Example

1. Gross Shear Yielding at Cope

d = 13.8 in.

d_{ct} = 3.0 in.



$\phi V_n = 1.0\ (0.6\ F_y)\ (d - d_{ct})\ t_w$

$= 1.0\ (0.6 \times 50)\ (13.8 - 3.0)\ (0.27)$

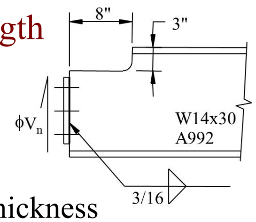
= 87.5 k

Shear End-Plate Connection Example

2. Coped Beam Flexural Strength

From previous example

$\phi M_n = 494$ in.-kips



With e = cope length + plate thickness
= 8.0 + 0.25 = 8.25 in.

$\phi V_n = 494 / 8.25$
= 59.9 k

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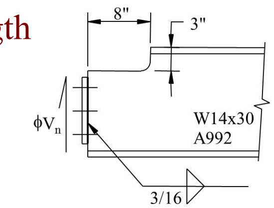
Shear End-Plate Connection Example

3. Web Shear Rupture Strength at Weld

Plate L = 8.5 in.

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

Beam Web $t_w = 0.27$ in.



$\phi V_n = 0.75\ (0.6\ F_u)\ (L - 2\ t_{weld})\ t_w$
= 0.75 (0.6 x 65) [8.5 – (2 x 3/16)] (0.27)
= 64.2 k

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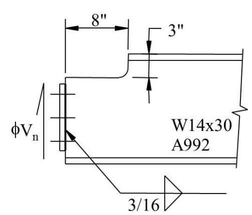
Shear End-Plate Connection Example

4. Weld Rupture Strength

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



$\phi V_n = (D \times 1.392)\ (L - 2\ t_{weld})$
= (2 x 3 x 1.392) [8.5 – (2 x 3/16)]
= 67.9 k

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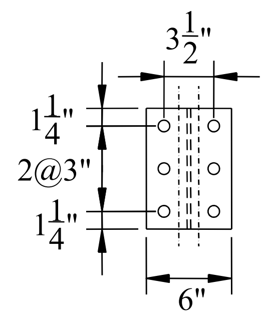
Shear End-Plate Connection Example

Plate Limit States:

$t_p = 1/4$ in.
A36 Steel:
$F_y = 36$ ksi $F_u = 58$ ksi



5. Gross Shear Yielding

$\phi V_n = 1.0\ (0.6\ F_y)\ (2\ L\ t_p)$
= 1.0 (0.6 x 36) (2 x 8.5 x 1/4)
= 91.8 k

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Lecturer: Tom Murray, P.E., PhD
Emeritus Professor, Virginia Tech

Shear End-Plate Connection Example

6. Net Shear Rupture

$d_h' = 3/4 + 1/16 + 1/16 = 7/8$ in.

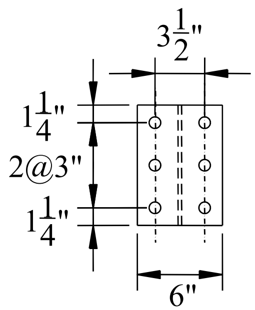
$A_n = (8.5 - 3 \times 7/8)(1/4)(2)$

$= 2.94$ in.^{2}

$\phi V_n = 0.75\,(0.6\,F_u)\,(A_n)$

$= 0.75\,(0.6 \times 58)\,(2.94)$

$=$ 76.7 k



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Shear End-Plate Connection Example

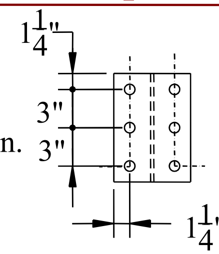
7. Block Shear Strength

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

d = 3/4 in. d_h = 13/16 in. d'_h = 7/8 in.

$\phi = 0.75$

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



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Shear End-Plate Connection Example

7. Plate Block Shear

- Shear Yielding

$0.6F_yA_{gv} = (0.6 \times 36)(2 \times 0.25 \times 7.25)$

$= 78.3$ k

- Shear Rupture

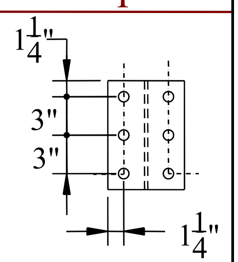
$0.6F_uA_{nv} = (0.6 \times 58)\,(7.25 - 2.5 \times 7/8)\,(2 \times 1/4)$

$= 88.1$ k Shear Yielding Controls

- Tension Rupture

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

$= 23.6$ k



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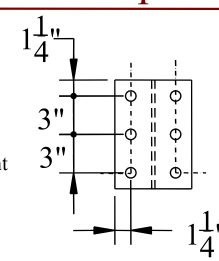
Shear End-Plate Connection Example

7. Plate Block Shear

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

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

$\phi V_n = 0.75 \times 101.9 =$ 76.4 k



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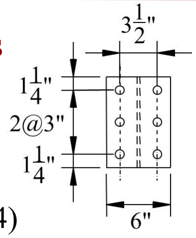
Shear End-Plate Connection Example

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.

Brg: $2.4\ F_u\ d_b\ t$ = (2.4 x 58) (3/4 x 1/4)
= 26.1 k

Edge: $1.2\ F_u\ L_c\ t$ = (1.2 x 58) (1.25 – 13/32) (1/4)
= <u>14.7 k</u> < 26.1 k

Other: $1.2\ F_u\ L_c\ t$ = (1.2 x 58) (3 – 13/16) (1/4)
= 38.1 k > <u>26.1 k</u>

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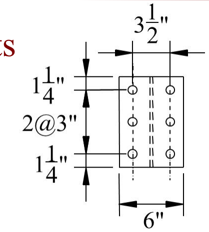
Shear End-Plate Connection Example

8. Shear Transfer Between Elements



- Bolt Shear Rupture

3/4" A325-N F_{nv} = 54 ksi

r_n = (54 ksi)(0.4418 in^2) = 23.8 k

- Design Shear Transfer Strength

V_n = 2[min. (Edge B/T.O., Bolt Shear Rupture)]
4[min. (Other B/T.O., Bolt Shear Rupture)]
= 2[min. (<u>14.7</u>, 23.8)] + 4[min. (26.1, <u>23.8</u>)]
= 125 k

ϕV_n = 0.75 x 125 = <u>93.4 k</u>

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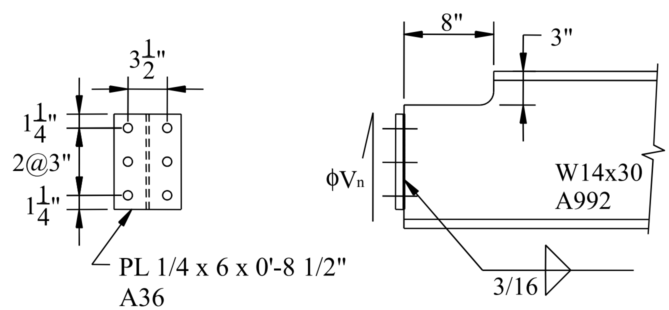
Shear End-Plate Connection Example

<u>CONNECTION DESIGN STRENGTH</u>



Coped Beam Flexural Strength Controls

<u>ϕV_n = 59.9 k</u>

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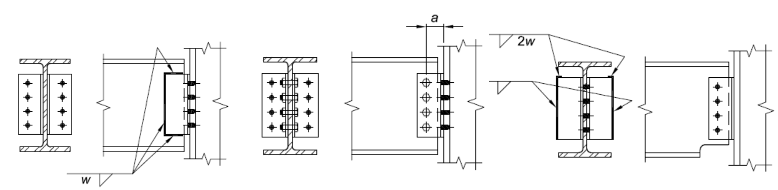
DOUBLE ANGLE CONNECTIONS



<u>Welded/Bolted</u> <u>Bolted/Bolted</u> <u>Bolted/Welded</u>

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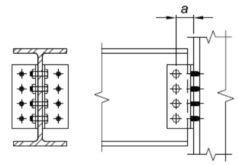
Double Angle Connections

Advantages:

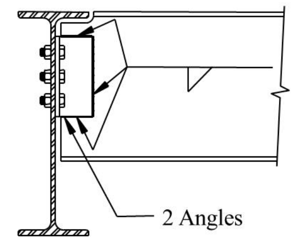
- Beam Length can Vary
- Weld or Bolt to Beam



Disadvantages:

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

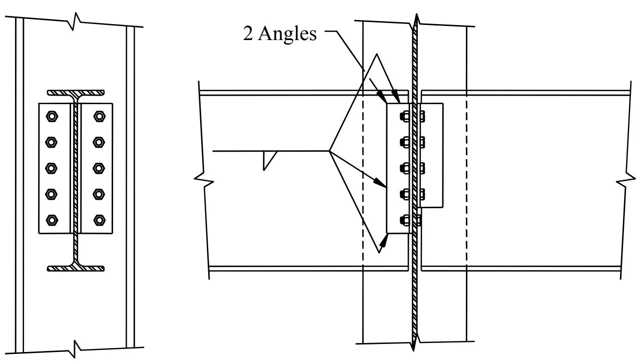
Welded/Bolted Double Angle Connections



Beam Dropped for Joist Seat

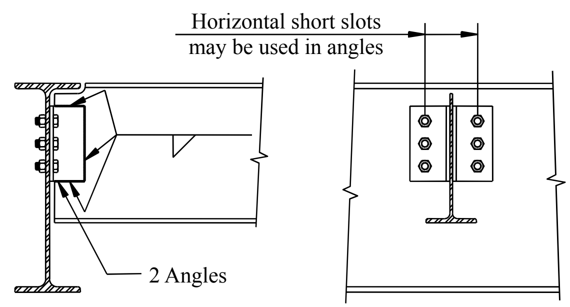
Coped Beam

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

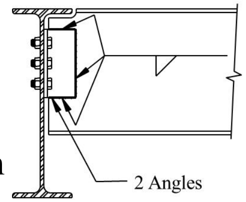
Limit States

Beam:

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

Weld:

- Weld Rupture Due to Shear Plus Torsion



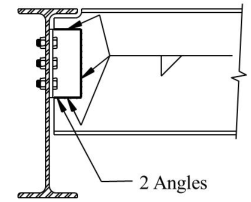
Welded/Bolted Double Angle Connections

Angles:

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

Shear Transfer Between Elements:

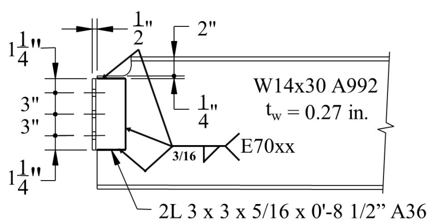
- Angle Bearing/Tear Out
- Bolt Shear Rupture
- Supporting Element Bearing/Tear Out



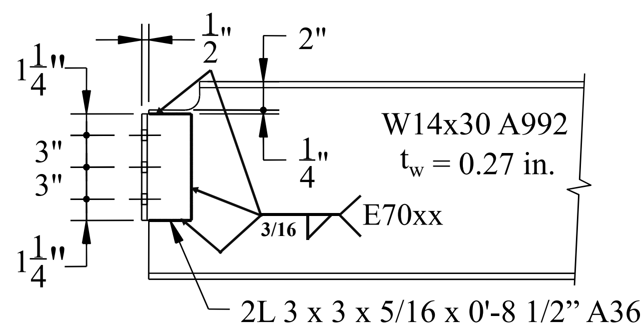
Welded/Bolted Double Angles Example

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

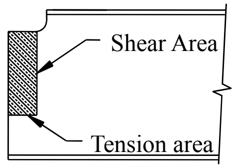
  $0.6\ F_y\ A_{gv} = 0.6\ (50)\ (8.5 + 0.25)(0.27)$

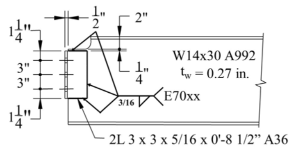
  $= \underline{70.9\ k}$

- Tension Rupture

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

  $= \underline{39.5\ k}$

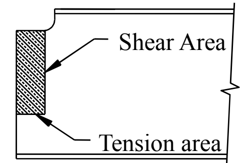




Welded/Bolted Double Angles Example

1. Beam Web Block Shear

- Beam Web Block Shear Strength



$\phi V_n = 0.75$ (min shear + U_{bs} tension rupture)

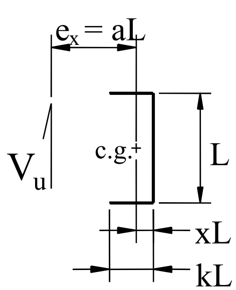
$= 0.75\ (70.9 + 1.0 \times 39.5)$

$= \underline{82.8\ k}$

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

Welded/Bolted Double Angles Example

2. Weld Rupture Due to Eccentric Shear



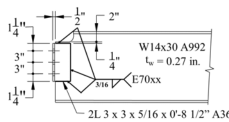
$\phi = 0.75$

$V_n = C\ C_1\ D\ l$

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

$C_1 = F_u$ of weld metal / 70

D = number of 1/16 ths



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

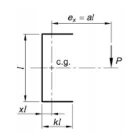
Welded/Bolted Double Angles Example

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

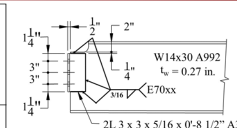
LRFD			ASD		
$C_{min} = \frac{P_u}{\phi C_1 D l}$	$D_{min} = \frac{P_u}{\phi C C_1 l}$	$l_{min} = \frac{P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 D l}$	$D_{min} = \frac{\Omega P_a}{C C_1 l}$	$l_{min} = \frac{\Omega P_a}{C C_1 D}$

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



Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	8.26	9.37	10.5	11.6	12.7
0.10	1.86	2.28	2.78	3.30	3.83	4.37	4.92	5.46	6.01	6.56	7.11	8.22	9.32	10.4	11.5	12.7
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.41	6.94	8.02	9.11	10.2	11.3	12.4
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



$\phi = 0.75$

$R_n = CC_1Dl$

Parameters:

$C_1 = Exx/70$

$k \Rightarrow x$

$x \Rightarrow a$

$x\ \&\ a \Rightarrow C$

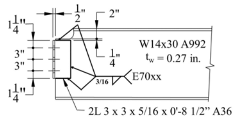
Welded/Bolted Double Angles Example

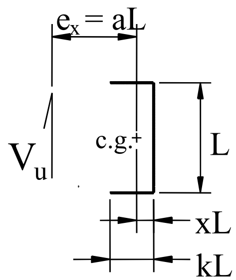
Determine C from Table 8-8:

2-L3x3x5/16 x 0’-8 ½”

L = 8.5 in.

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





Welded/Bolted Double Angles Example

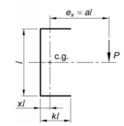
Table 8-8
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

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

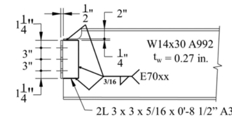
LRFD			ASD		
$C_{min} = \frac{P_u}{\phi C_1 D l}$	$D_{min} = \frac{P_u}{\phi C C_1 l}$	$l_{min} = \frac{P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 D l}$	$D_{min} = \frac{\Omega P_a}{C C_1 l}$	$l_{min} = \frac{\Omega P_a}{C C_1 D}$

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

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



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



$R_n = CC_1Dl$
$\phi = 0.75$
Parameters:
$C_1 = 1.0$
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 ½”

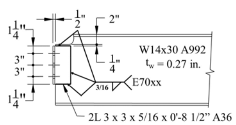
L = 8.5 in.

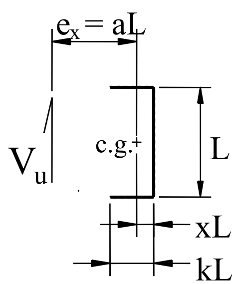
k = (3 - 0.5 - 0.25)/ 8.5 = 0.26

$\Rightarrow$ x = 0.0465

xL = 0.0465 x 8.5 = 0.40 in.

a = (3.0 – 0.40) / 8.5 = 0.31





Welded/Bolted Double Angles Example

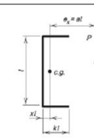
Table 8-8
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

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

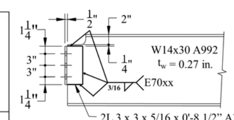
LRFD			ASD		
$C_{min} = \frac{P_u}{\phi C_1 D l}$	$D_{min} = \frac{P_u}{\phi C C_1 l}$	$l_{min} = \frac{P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 D l}$	$D_{min} = \frac{\Omega P_a}{C C_1 l}$	$l_{min} = \frac{\Omega P_a}{C C_1 D}$

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

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* 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 & a $\Rightarrow$ C = 2.62

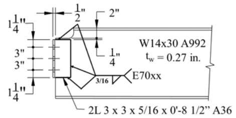
Welded/Bolted Double Angles Example

Using *Manual* Table 8-8

$\Rightarrow C = 2.62$

With $C_1 = 1.0$

$D = 2$ (Two Ls) x 3



Weld Rupture Strength

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

$= (0.75)(2.62)\,(1.0)\,(2 \text{ x } 3)\,(8.5)$

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

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

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Welded/Bolted Double Angles Example

3. Beam Web Strength at Weld

$$\phi V_n = \frac{\text{Design Rupture Strength of Plate/in.}}{\text{Design Rupture Strength of Weld/in.}} \text{ x Design Connection Weld Rupture Strength, k}$$

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

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

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

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

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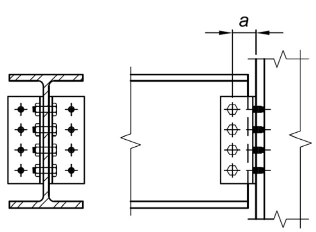
Bolted/Bolted Double Angle Connections



Beam to Girder

Beam to Column Flange

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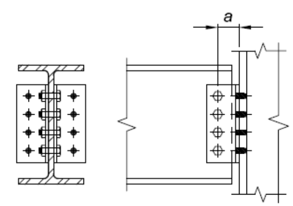
Bolted/Bolted Double Angle Connections

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

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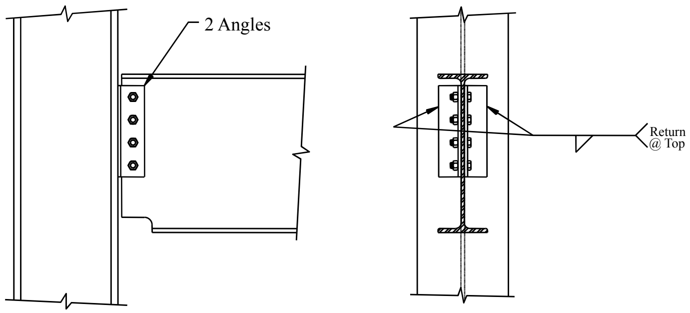
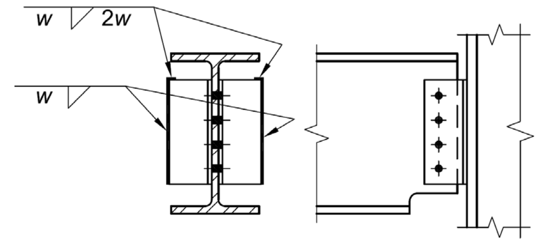
Double Angle Knife Connections



Bolted/Welded Double Angle Knife Connection

Double Angle Knife Connections



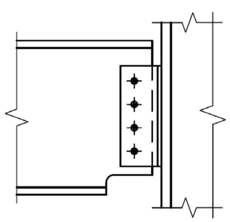
Note: Weld returns on top of angles per *Specification* Section J2.2b

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

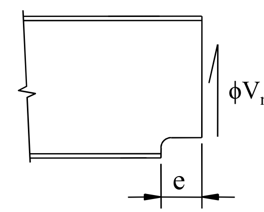
Coped Beam Web Strength at Tension Flange

$\phi_b = 0.9$

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

where

$$M_n = \min \left| \begin{array}{l} M_p = F_y Z_{net} \\ 1.6 M_y = 1.6 F_y S_{net} \end{array} \right.$$

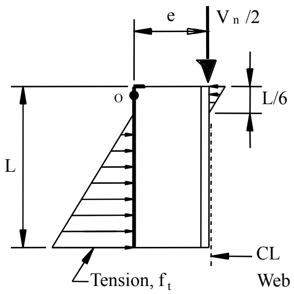


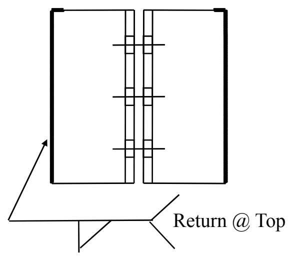
S_{net} = elastic section modulus from *Manual* Table 9-2

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

Bolted/Welded Double Angles

Weld Strength on OSLs





Bolted/Welded Double Angles

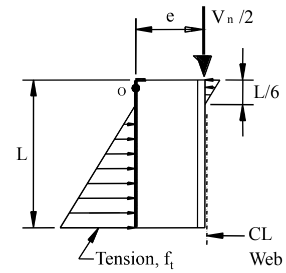
Weld Strength on OSLs

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

$$\sum M_o = 0$$

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

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



Bolted/Welded Double Angles

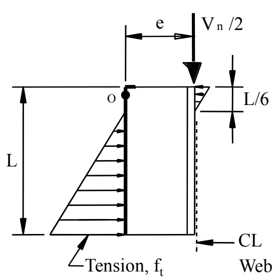
Weld Strength on OSLs

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

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

with $\phi f_w = 1.392D$

$$\phi R_n = 2\left(\frac{1.392 DL}{\sqrt{1 + 12.96 e^2 / L^2}}\right)$$

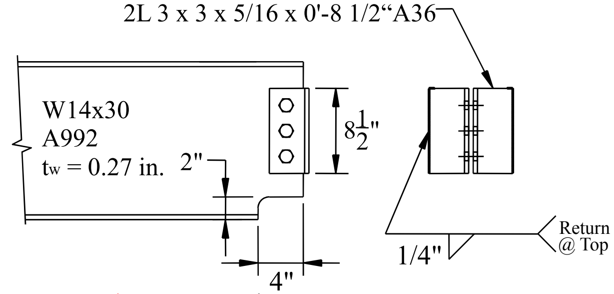


(*Manual* p.10-11)

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 in.

L= 8.5 in.

3/4" A325-N Bolts
E70XX

Bolted/Welded Double Angles Example

Weld design rupture strength at OSLs:

$$\phi V_n = \frac{2 L^2 (1.392 D)}{\sqrt{L^2 + 12.96 e^2}}$$

$$= \frac{2 (8.5)^2 (1.392 \times 4)}{\sqrt{8.5^2 + 12.96 \times (3.14)^2}}$$

$$= 56.9 \text{ k}$$

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

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End of Session 3

Thank You for Attending

Next Up

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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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Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

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

Lecturer: Tom Murray, P.E., PhD
Emeritus Professor, Virginia Tech

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

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

8-Session Registrants

CEU/PDH Certificates

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

8-Session Registrants

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

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

Reasons for quiz:

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

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

8-Session Registrants

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

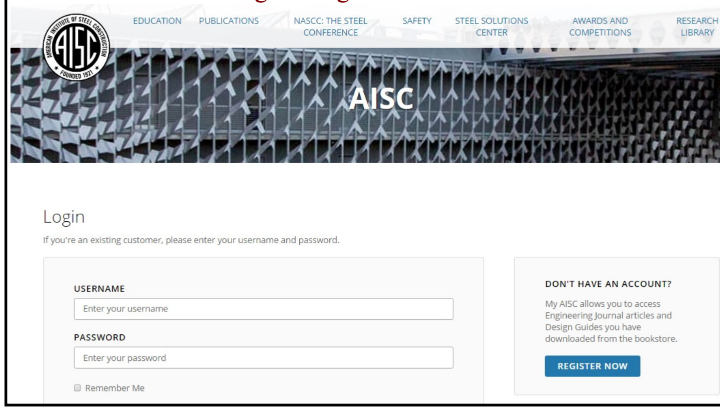
CEUs/PDHS – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHs.

Night School Resources for 8-session package Registrants

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

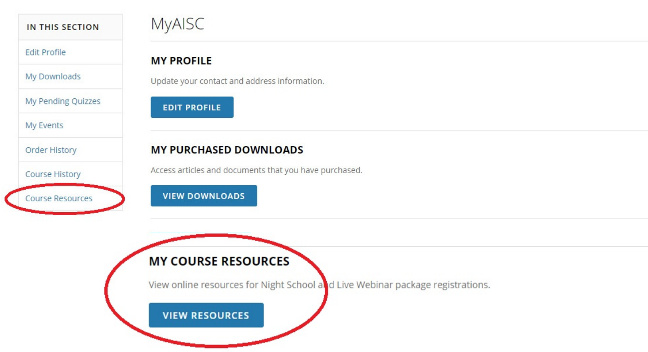
Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.

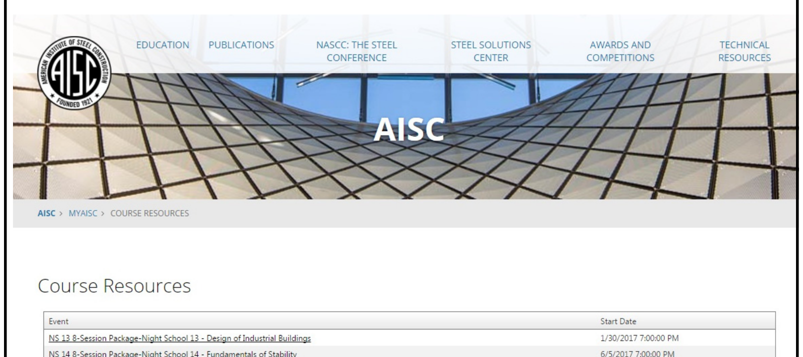


Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.

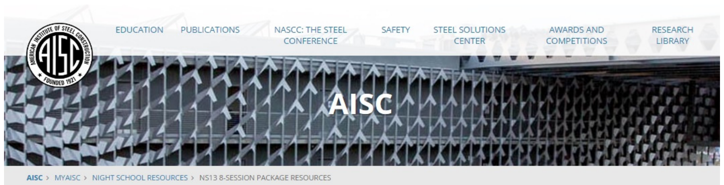


Night School Resources for 8-session package Registrants



Lecturer: Tom Murray, P.E., PhD
Emeritus Professor, Virginia Tech

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 Dsn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	

Night School Resources for 8-session package Registrants

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

Night School Resources for 8-session package Registrants

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

There's always a solution in steel.

Thank You

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

structural STEEL