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Fundamentals of Connection Design
Session 4: Shear Connections, Part II
November 13, 2019

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Course Description

Shear Connections, Part II
November 13, 2019

This session will cover single-plate shear connection design, including both conventional and extended single-plate connections. The differences between the two will be contrasted in design examples. The design of single angle connections, stiffened and unstiffened seated connections will also be discussed. The presentation of stiffened seated connections will offer a discussion on a simplified approach.

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Learning Objectives

- Describe the advantages and disadvantages of single-angle connections.
- Describe the advantages and disadvantages of conventional single-plate connections.
- Explain the differences between conventional and extended single-plate connection design.
- Compare stiffened and unstiffened seated connections.

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Fundamentals of Connection Design

Session 4: Shear Connections, Part II
November 13, 2019

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

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Schedule

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

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SHEAR CONNECTIONS PART II

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TOPICS

- Single-Angle Connections
- Single-Plate or Shear Tab Connections
- Unstiffened Seated Connections
- Stiffened Seated Connections

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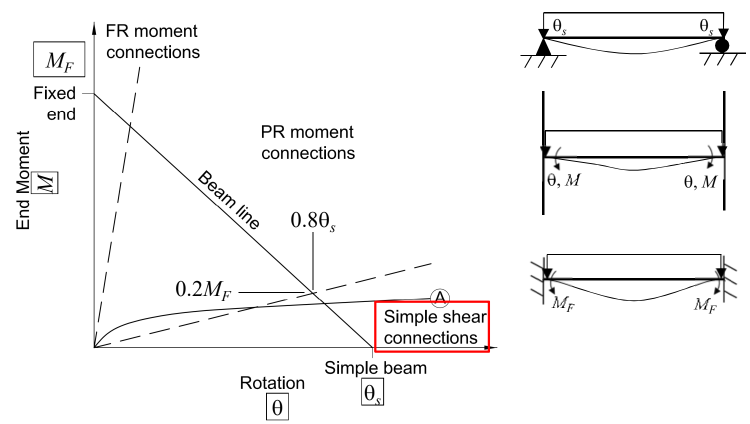
Connection Classification



Manual Figure 10-1

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Single-Angle Connections

Single-Angle Connections

Advantages

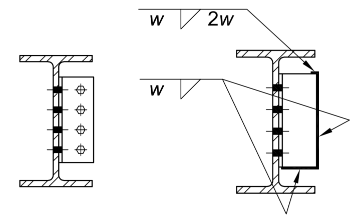
- Eliminates Double Sided Erection Problem
- Fewer Parts

Disadvantages

- Larger Angle Required
- Larger Bolts or Weld
- Cannot Resist Axial Forces

Comment

Not recommended for laterally unbraced beams.



Manual Figure 10-13

Single-Angle Connections

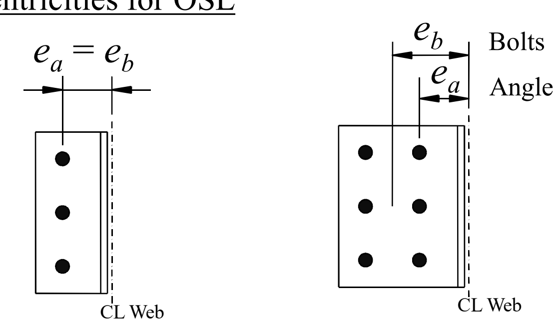
<u>Recommended Minimum Angle Thicknesses</u>

Manual Page 10-116.

Bolt Diameter d (in.)	Minimum Angle Thickness t_{min} (in.)
3/4	3/8
7/8	3/8
1	1/2

Single-Angle Connections: Bolted

<u>Eccentricities for OSL</u>



<u>Single Column of Bolts</u> <u>Double Column of Bolts</u>

Single-Angle Connections: Bolted

Notes

- Beam-side eccentricity: ignored when one column of bolts and eccentricity $\leq$ 3 in.
- Holes
  - Beam side: standard holes or short slots.
  - Support side: standard holes.
- Single angle should be connected to supporting member in the shop.
- Additional Limit States at OSL
  - Angle Flexural Yielding and Rupture
  - Bolt Eccentric Shear and Bearing / Tearout

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Single-Angle Connections: Bolted

Angle Flexural Yielding of OSL

Criterion

$M_u \leq \phi M_n$

LRFD Required Moment

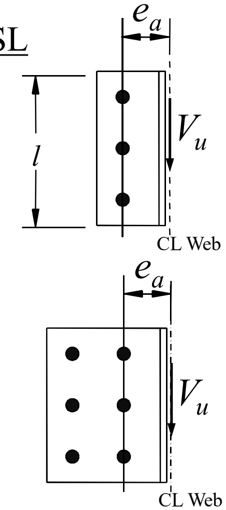
$M_u = V_u e_a$

Flexural Design Strength

$\phi M_n = \phi F_y S_g$

$S_g = t_a l^2/6$

$\phi = 0.9$



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Single-Angle Connections: Bolted

Angle Flexural Rupture of OSL

Criterion

$M_u \leq \phi M_n$

LRFD Required Moment

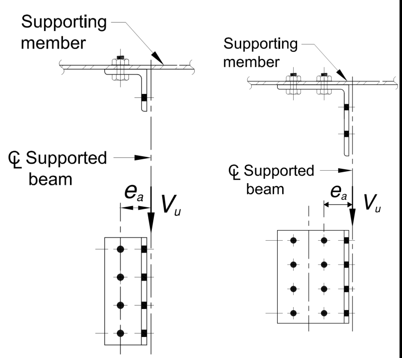
$M_u = V_u e_a$

Flexural Design Strength

$\phi M_n = \phi F_u Z_{net}$

$\phi = 0.75$

Z_{net} from *Manual* Table 15-3



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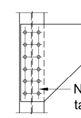
Single-Angle Connections: Bolted

Table 15-3
Net Plastic Section Modulus, Z_{net}, in.^{3}
(Standard Holes)



# Bolts in One Vertical Row, *n*	Bracket Plate Depth, *a*, in.	Nominal Bolt Diameter, *d*, in. 3/4					7/8		
		Bracket Plate Thickness, *t*, in.							
		1/4	3/8	1/2	5/8	3/4	3/8	1/2	5/8
2	6	1.59	2.39	3.19	3.98	4.78	2.25	3.00	3.75
3	9	3.70	5.55	7.40	9.26	11.1	5.25	7.00	8.75
4	12	6.38	9.56	12.8	15.9	19.1	9.00	12.0	15.0
5	15	10.1	15.1	20.2	25.2	30.2	14.3	19.0	23.8
6	18	14.3	21.5	28.7	35.9	43.0	20.3	27.0	33.8
7	21	19.6	29.5	39.3	49.1	58.9	27.8	37.0	46.3
8	24	25.5	38.3	51.0	63.8	76.5	36.0	48.0	60.0
9	27	32.4	48.6	64.8	81.0	97.2	45.8	61.0	76.3
10	30	39.8	59.8	79.7	99.6	120	56.3	75.0	93.8
12	36	57.4	86.1	115	143	172	81.0	108	135
14	42	78.1	117	156	195	234	110	147	184

Manual Table 15-3

s = 3 in.

l_{ev} = 1-1/2 in.

3/4, 7/8, 1 in. Bolts

$d'_h = d_h$ + 1/16 in.

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Single-Angle Connections: Bolted

Eccentric Shear Transfer at OSL

Manual Page 10-118 and Design Example II.A-28A

Instantaneous Center of Rotation Method

$$\phi V_n = C\,(\phi r_n)$$

where

C = eccentrically loaded bolt group coefficient from Table 10-11 or from Table 7-6, 7-7, etc.

ϕr_n = effective fastener strength of the outermost (worst case) bolt.

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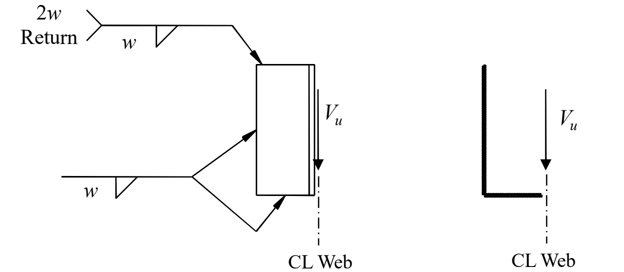
Single-Angle Connections: Welded

Eccentric Shear Strength of Weld Group



Manual Table 10-12
or Table 8-10

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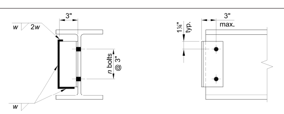
Single-Angle Connections: Welded

Table 10-12 (continued)
Bolted/Welded
Single-Angle Connections



Number of Bolts in One Vertical Row	Bolt and Angle Strength, kips, Group A Bolts, 3/4 in., ASD	3/4 in., LRFD	7/8 in., ASD	7/8 in., LRFD	Angle Size (F_y = 36 ksi)	Angle Length, in.	Weld (70 ksi) Size, w, in.	Weld Available Strength, kips, ASD	Weld Available Strength, kips, LRFD	Minimum t_w of Supporting Member with Angles Both Sides of Web, in.
6	71.6	107	71.3	107	L4×3×3/8	17 1/2	5/16	94.3	141	0.475
							1/4	75.5	113	0.380
							3/16	56.6	84.9	0.285
5	59.7	89.5	59.1	88.7		14 1/2	5/16	79.1	119	0.475
							1/4	63.3	94.9	0.380
							3/16	47.4	71.2	0.285
4	47.6	71.4	47.0	70.4		11 1/2	5/16	62.9	94.4	0.475
							1/4	50.3	75.5	0.380
							3/16	37.8	56.6	0.285
3	35.5	53.2	34.8	52.2		8 1/2	5/16	45.7	68.5	0.475
							1/4	36.6	54.8	0.380
							3/16	27.4	41.1	0.285
2	23.3	35.0	22.7	34.0		5 1/2	5/16	28.2	42.2	0.475
							1/4	22.5	33.8	0.380
							3/16	16.9	25.3	0.285

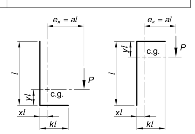
Table 8-10
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

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

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

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	2.04	2.23	2.41	2.69	2.97	3.25	3.53	3.80	4.08	4.36	4.92	5.47	6.03	6.59	7.15
0.10	1.86	2.04	2.28	2.53	2.78	3.04	3.31	3.57	3.84	4.11	4.38	4.93	5.48	6.00	6.55	7.10
0.15	1.83	2.03	2.25	2.49	2.74	2.99	3.24	3.50	3.75	4.01	4.28	4.81	5.34	5.89	6.44	7.00
0.20	1.76	1.97	2.18	2.40	2.64	2.87	3.11	3.36	3.60	3.85	4.11	4.62	5.14	5.66	6.20	6.73
0.25	1.66	1.86	2.07	2.29	2.50	2.73	2.95	3.19	3.42	3.66	3.90	4.40	4.90	5.42	5.94	6.47
0.30	1.55	1.74	1.94	2.15	2.36	2.57	2.78	3.00	3.22	3.45	3.69	4.17	4.66	5.17	5.68	6.20
0.40	1.33	1.49	1.67	1.85	2.05	2.24	2.44	2.63	2.84	3.05	3.27	3.73	4.20	4.69	5.19	5.70
3.0	0.219	0.246	0.273	0.304	0.339	0.379	0.425	0.475	0.531	0.587	0.645	0.776	0.924	1.09	1.27	1.46
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

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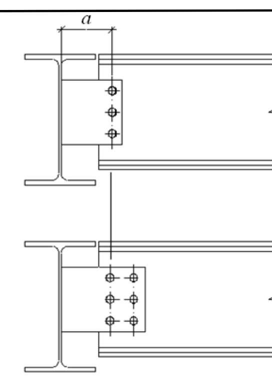
Single-Plate Shear Connections



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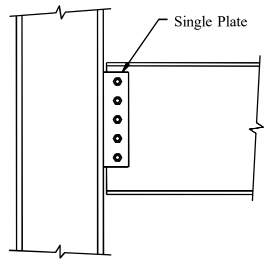
Single-Plate Connections

Advantages

- Simple – Few Parts
- No Welding on Beam
- Can be Designed to Resist Axial Force

Disadvantages

- Stiffer than Other Types
- Requires Careful Design
- Low to Moderate Strength



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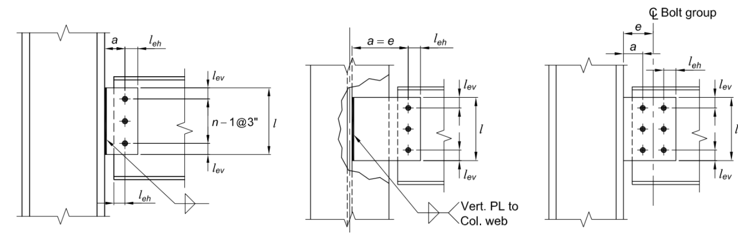
Single-Plate Connections



(Also used to eliminate beam copes)

<u>Conventional Single-Plates</u>

- Various geometric limitations.
- Simplified Design → avoids checking some limit states.

<u>Extended Single-Plates</u>

- Very few limitations.
- General Design → consider all limit states.

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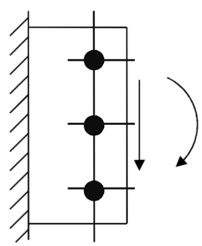
Single-Plate Connections

<u>Limit States</u>

- Beam Side Limit States
- Eccentric Bolt Shear
- Shear Transfer
- Plate
  - Shear Yielding
  - Shear Rupture
  - Block Shear
  - Flexural Rupture
  - Plate Buckling
  - Combined Shear and Flexure



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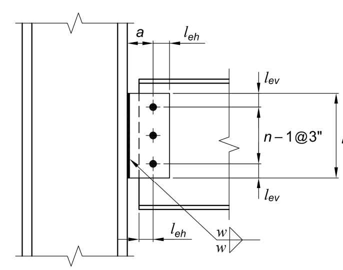
Conventional Single-Plate Connections

<u>Limitations</u>

- One column of bolts.
- $2 \le n \le 12$
- $a \le$ 3-1/2 in.
- $l_{ev} \ge$ limits in *Specification* Table J3.4
- $w \ge 5/8\, t_p$ on both sides of plate. Develops the plate if $F_y \le 50$ ksi.
- Max. t_p **<u>or</u>** t_w from Table 10-9
- $l_{eh} \ge 2d$ for the plate **<u>or</u>** beam web.



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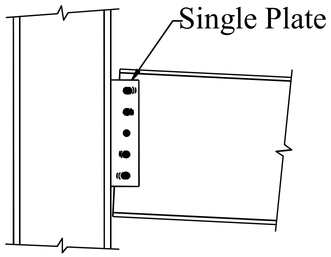
Conventional Single-Plate Connections

Ductility – Maximum Thickness Requirement

Bolts must "plow" in plate or web. Thus, plate or web thickness is limited.



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Conventional Single-Plate Connections

Bolts and Plate Checked for Eccentric Shear

$$M_u = V_u e$$

where e is from *Manual* Table 10-9.

Table 10-9
Design Values for Conventional Single-Plate Shear Connections

n	Hole Type	e, in.	Maximum t_p or t_w, in.
2 to 5	SSLT	$a/2$	None
	STD	$a/2$	$d/2 + 1/16$
6 to 12	SSLT	$a/2$	$d/2 + 1/16$
	STD	a	$d/2 - 1/16$

Note: Plate buckling will not control plate design.

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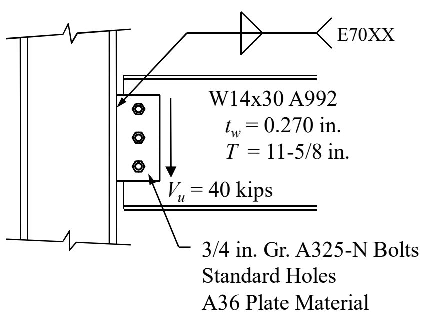
Conventional Single-Plate Connection Ex.

Example: Determine required number of bolts and plate and weld sizes.



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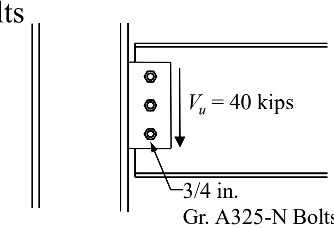
Conventional Single-Plate Connection Ex.

Bolt Shear Rupture

Select trial number of bolts based on direct shear.

Try 3 – 3/4 in. Gr. A325-N Bolts

$$\phi V_n = (\phi F_{nv} A_b)n$$
$$= (17.9 \text{ kips})(3 \text{ bolts})$$
$$= 53.7 \text{ kips} > V_u = 40 \text{ kips}$$



Still need to consider eccentric shear.

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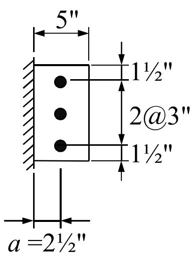
Conventional Single-Plate Connection Ex.

Trial Plate Geometry



Conventional Single-Plate Limitations

$2 \le n \le 12$

$a = 2\frac{1}{2}$ in. < $3\frac{1}{2}$ in.

$l_{eh} = 2.5$ in. $\ge 2d = (2)(3/4$ in.$) = 1.5$ in.

$l_{ev} = 1.5$ in. > 1.0 in. (*Spec.* Table J3.4)

All OK

Also: $l = 9$ in. > $T / 2 = 11\text{-}5/8 / 2 = 5.81$ in. → OK

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Conventional Single-Plate Connection Ex.

Rotational Ductility

2 to 5 Bolts

$t_{max} = d / 2 + 1/16 = (3/4)/2 + 1/16 = 0.438$ in.

Try 1/4 in. plate < 0.438 in. (Note: $t_w = 0.270$ in.)

Eccentricity

$e = a / 2 = 2.5 / 2 = 1.25$ in.

Table 10-9
Design Values for Conventional Single-Plate Shear Connections

n	Hole Type	e, in.	Maximum t_p or t_w, in.
2 to 5	SSLT	$a/2$	None
	STD	$a/2$	$d/2 + 1/16$
6 to 12	SSLT	$a/2$	$d/2 + 1/16$
	STD	a	$d/2 - 1/16$

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Conventional Single-Plate Connection Ex.

Check Eccentric Bolt Shear Strength

$\phi r_n = 17.9$ kips

$n = 3 \quad e_x = 1.25$ in.

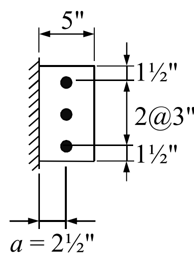
From *Manual* Table 7-6, $C = 2.59$

$\phi V_n = C\,\phi r_n$

$= (2.59)(17.9)$

$= 46.4$ kips > 40 kips

OK



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Conventional Single-Plate Connection Ex.

PLATE LIMIT STATES

$t = 1/4$ in. $F_y = 36$ ksi $F_u = 58$ ksi

Shear Yielding

$\phi V_n = \phi 0.6\, F_y A_{gv}$

$= (1.0)(0.6)(36)(0.25)(9)$

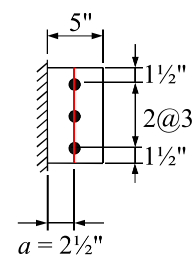
$= 48.6$ kips > 40 kips OK

Shear Rupture

$\phi V_n = \phi 0.6 F_u A_{nv}$

$= 0.75(0.6)(58)(1/4)[9 - (3)(7/8)]$

$= 41.6$ kips > 40 kips OK



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Conventional Single-Plate Connection Ex.

Block Shear

$\phi V_n = 52.8$ kips > 40 kips OK

Shear Transfer

Bolt Shear Rupture = 23.9 kips

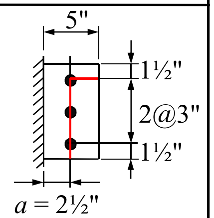
Beam web will not control

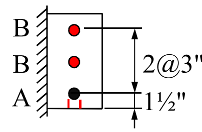
Plate Bearing: $r_n = 26.1$ kips

Plate Tearout at A: $r_n = 19.0$ kips

Plate Tearout at B: $r_n = 38.1$ kips

$$\phi V_n = 0.75(19.0 + 23.9 + 23.9)$$
$$= 50.1 \text{ kips} > 40 \text{ kips OK}$$





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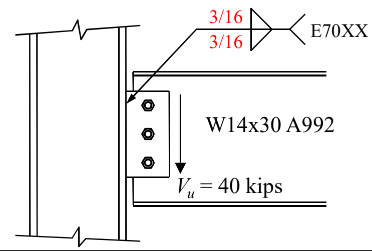
Conventional Single-Plate Connection Ex.

Required Fillet Weld Size

$w = 5/8t_p = 5/8(¼) = 5/32$ in. → 3/16 in. both sides

No other calculations needed.



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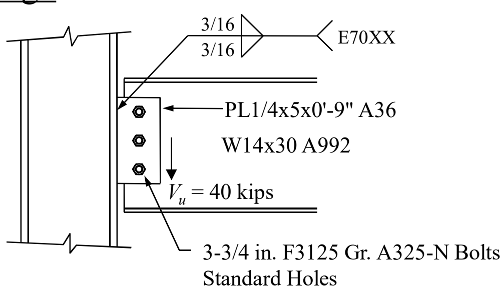
Conventional Single-Plate Connection Ex.

Final Design



$$V_u = 40 \text{ kips} \le \phi V_n = 41.6 \text{ kips}$$
(Plate Shear Rupture Controls)

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Extended Single-Plate Connections

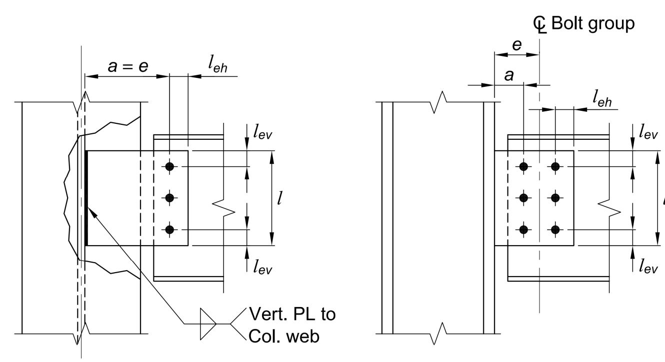

Fig. 10-12. Single-plate connection—Extended Configuration.

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Extended Single-Plate Connections

Limitations

- No limit on *a*
- No limit on number of bolts in a column
- No limit on number of columns of bolts
- l_{ev} and l_{eh} per *Specification* Table J3.4
- Maximum plate thickness such that:

Plate Flexural Strength ≤ Bolt Group Flexural Strength
or
Satisfy conventional plate max. thickness requirements.

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Extended Single-Plate Connections

Rotational Ductility

Plate Flexural Strength ≤ Bolt Group Flexural Strength

$$F_y S_g \leq M_{max} = (F_{nv} / 0.9) A_b C'$$

F_y = Plate yield stress
S_g = Plate elastic section modulus = $t_p l^2/6$
t_p = Plate thickness
l = Plate depth
F_{nv} = Bolt shear rupture strength (Table J3.2)
A_b = Bolt area
C' = equivalent eccentricity for pure moment (*Manual* Tables 7-6 through 7-13)

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Extended Single-Plate Connections

Rotational Ductility

Thus, the maximum single-plate thickness is:

$$t_{max} = \frac{6M_{max}}{F_y l^2} = \frac{6(F_{nv} / 0.9) A_b C'}{F_y l^2} \quad (\textit{Manual}\text{ Eq. 10-3})$$

Or the "plowing rules" for conventional connections can be used. See "Exceptions" on Page 10-90.

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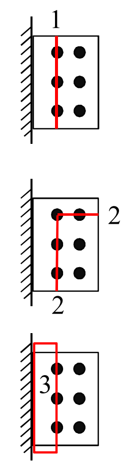
Extended Single-Plate Connections

Plate Limit States

- Shear Yielding (Just left of Section 1)
- Shear Rupture (Section 1)
- Flexural Rupture (Section 1)
- Shear Yielding + Flexural Yielding Interaction (Just left of Section 1)
- Block Shear (Section 2)
- Plate Buckling (Region 3)



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Extended Single-Plate Connections

Plate Shear Yielding, Shear Rupture, and Block Shear

Same as previous.

Plate Flexural Rupture

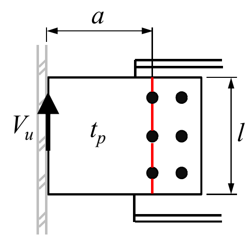
Same as for single angles.

$M_u \le \phi M_n \qquad \phi = 0.75$

$M_u = V_u a$

$M_n = F_u Z_{net}$

Z_{net} from Table 15-3



Extended Single-Plate Connections

Shear and Flexure Interaction

$$\left(\frac{V_u}{\phi_v V_n}\right)^2 + \left(\frac{M_u}{\phi_b M_n}\right)^2 \le 1.0 \qquad (\textit{Manual}\text{ Eq. 10-5})$$

$\phi_v = 1.00 \quad \phi_b = 0.90$

V_u = Required shear strength

$V_n = 0.6F_y A_g$

M_u = Required flexural strength = $V_u a$

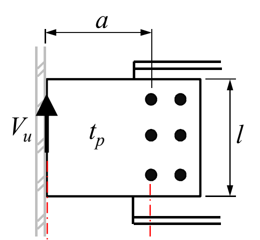
$M_n = F_y Z_{pl} = F_y(t_p l^2/4)$

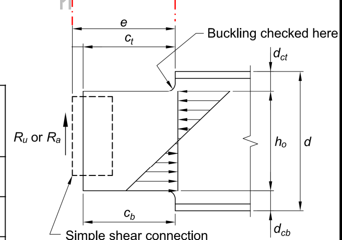
Extended Single-Plate Connections

Plate Buckling

- Treat as the web of a double-coped beam.
- $M_u \le \phi M_n \qquad \phi = 0.90$
- $M_u = V_u a$
- M_n: *Spec.* F11 with $C_b \ge 1.84$ from *Manual* Eq. 9-15.

Spec. F11 Variable	Single-Plate Shear Connection Variable
L_b	a
d	l
t	t_p





Extended Single-Plate Connections

Plate Buckling (Modified *Spec.* F11 Equations)

$\lambda \le \lambda_p$

$M_n = M_p = F_y Z_{pl}$ (*Spec.* Eq. 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.* Eq. F11-2)

$\lambda > \lambda_r$

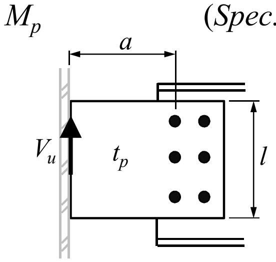
$M_n = C_b(1.9E/\lambda)S_x \le M_p$ (*Spec.* Eq. F11-3)

where

$\lambda = al/t_p^2$

$\lambda_p = 0.08E/F_y$

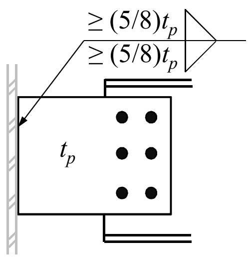
$\lambda_r = 1.90E/F_y$



Extended Single-Plate Connections

Other Limit States

- Eccentrically loaded bolt group shear strength.
- Shear Transfer
- Fillet Weld Strength
  - Develop the plate → no strength calculation needed.



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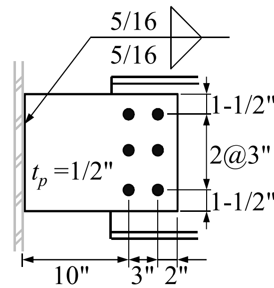
Extended Single-Plate Connection Example

Example: evaluate the extended single-plate connection for V_u = 30 kips.



Beam: W14x43 A992
t_w = 0.305 in.

Column: W14x90 A992
t_w = 0.440 in.

PL1/2x9x1'-3" A36

7/8 in. Gr. A325-N Bolts

E70xx Welds

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Extended Single-Plate Connection Example

Rotational Ductility

$$t_{max} = \frac{6M_{max}}{F_y l^2} = \frac{6(F_{nv}/0.9)A_b C'}{F_y l^2}$$

7/8 in. Gr. A325-N Bolt:

F_{nv} = 54 ksi from Table J3.2

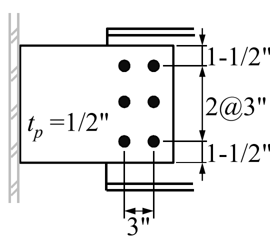
A_b = 0.601 in.^{2}

Plate: F_y = 36 ksi l = 9 in.

C' from *Table* 7-7 (3 in. horizontal spacing) with

s = 3 in. (vertical spacing)

n = 3 in. (rows of bolts)



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Extended Single-Plate Connection Example

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

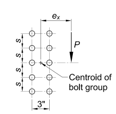
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

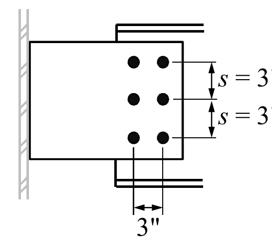
or

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

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8
	6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8
	7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8
	8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8
	9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8
	10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9
	12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2
	14	0.19	0.57	1.08	1.78	2.62	3.66	4.82	6.15	7.61	9.19	10.9	12.7
	16	0.17	0.51	0.95	1.57	2.32	3.24	4.27	5.47	6.79	8.23	9.78	11.4
	18	0.15	0.45	0.85	1.41	2.07	2.90	3.83	4.92	6.11	7.43	8.85	10.4
	20	0.14	0.41	0.77	1.27	1.88	2.63	3.48	4.47	5.55	6.76	8.07	9.48
	24	0.12	0.34	0.65	1.07	1.58	2.21	2.93	3.77	4.69	5.72	6.85	8.06
	28	0.10	0.29	0.56	0.92	1.36	1.90	2.53	3.25	4.05	4.95	5.93	7.00
	32	0.09	0.26	0.49	0.80	1.19	1.67	2.22	2.86	3.57	4.36	5.23	6.18
	36	0.08	0.23	0.43	0.72	1.06	1.49	1.98	2.55	3.18	3.90	4.67	5.52
	C', in.	2.94	8.33	15.8	26.0	38.7	54.2	72.2	93.1	117	143	172	204



s = 3 in. & n = 3
⇒ C' = 15.8 in.

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Extended Single-Plate Connection Example

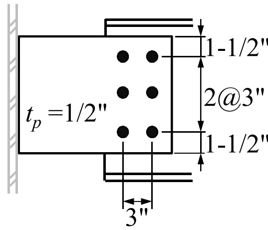
Rotational Ductility

$$t_{max} = \frac{6(F_{nv}/0.9)A_bC'}{F_y l^2}$$

$$= \frac{(6)(54/0.9)(0.601)(15.8)}{(36)(9^2)}$$

$$= 1.18 \text{ in.}$$

$t_p = 1/2 \text{ in.} < 1.18 \text{ in., OK}$



Note: also passes – barely – using the "plowing" max thickness from Table 10-9.

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Extended Single-Plate Connection Example

<u>Limit State Checks</u>

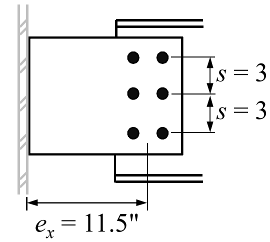
Eccentric Bolt Shear Rupture

7/8 in. Gr. A325-N bolts $\phi r_n = 24.3$ kips/bolt

From *Manual* Table 7-7 with $s = 3$ in., $n = 3$, and $e_x = 11.5$ in.:

$C = 1.30$

$\phi R_n = (1.30)(24.3)$
$= 31.6 \text{ kips} > 30 \text{ kips, OK}$



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Extended Single-Plate Connection Example

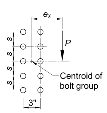
Table 7-7
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

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

$R_n = Cr_n$
or

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

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8
	6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8
	7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8
	8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8
	9	0.28	0.86	1.60	2.65	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	12.7	14.7	16.8
	10	0.26	0.78	1.46	2.42	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	11.8	13.8	15.9
	12	0.22	0.66	1.24	2.06	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	10.4	12.2	14.2
	14	0.19	0.57	1.08	1.78	2.62	[illegible]	[illegible]	[illegible]	7.61	9.19	10.9	12.7
	16	0.17	0.51	0.95	1.57	2.32	3.24	4.27	5.47	6.79	8.23	9.78	11.4
	18	0.15	0.45	0.85	1.41	2.07	2.90	3.83	4.92	6.11	7.43	8.85	10.4
	20	0.14	0.41	0.77	1.27	1.88	2.63	3.48	4.47	5.55	6.76	8.07	9.48
	24	0.12	0.34	0.65	1.07	1.58	2.21	2.93	3.77	4.69	5.72	6.85	8.06
	28	0.10	0.29	0.56	0.92	1.36	1.90	2.53	3.25	4.05	4.95	5.93	7.00
	32	0.09	0.26	0.49	0.80	1.19	1.67	2.22	2.86	3.57	4.36	5.23	6.18
	36	0.08	0.23	0.43	0.72	1.06	1.49	1.98	2.55	3.18	3.90	4.67	5.52
	C', in.	2.94	8.33	15.8	26.0	38.7	54.2	72.2	93.1	117	143	172	204

$C = 1.30$

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Extended Single-Plate Connection Example

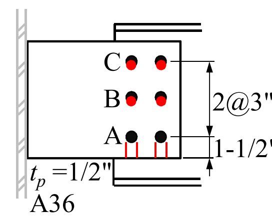
Shear Transfer

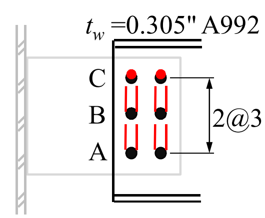
Bolt Shear Rupture
$r_n = 32.4$ kips

Plate
Bearing: $r_n = 60.9$ kips
Bolt A tearout: $r_n = 35.9$ kips
Bolts B, C tearout: $r_n = 71.8$ kips

Web
Bearing: $r_n = 41.6$ kips
Bolts A, B tearout = 49.1 kips

Bolt Shear Rupture Controls
$\phi R_n = 31.6 \text{ kips} > 30 \text{ kips, OK}$





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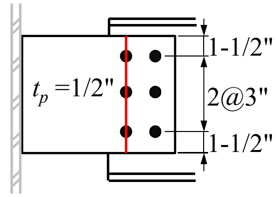
Extended Single-Plate Connection Example

Shear Yielding

$$\phi R_n = \phi 0.6 F_y A_{gv}$$
$$= (1.00)(0.6)(36)(1/2)(9)$$
$$= 97.2 \text{ kips} > 30 \text{ kips, OK}$$

Shear Rupture

$$\phi R_n = \phi 0.6 F_u A_{nv}$$
$$= (0.75)(0.6)(58)(1/2)\left[9-(3)(1 \text{ in.})\right]$$
$$= 78.3 \text{ kips} > 30 \text{ kips, OK}$$



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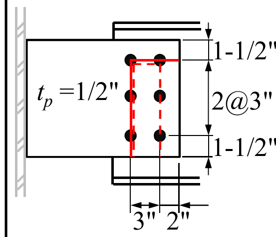
Extended Single-Plate Connection Example

Block Shear

$$R_n = \min \begin{vmatrix} 0.6 F_u A_{nv} \\ 0.6 F_y A_{gv} \end{vmatrix} + U_{bs} F_u A_{nt}$$
$$A_{gv} = (1/2)(7.5) = 3.75 \text{ in.}^2$$
$$A_{nv} = (1/2)\left[7.5 - 2.5(1)\right] = 2.50 \text{ in.}^2$$
$$A_{nt} = (1/2)\left[5 - 1.5(1)\right] = 1.75 \text{ in.}^2$$
$$\boxed{U_{bs} = 0.5 \text{ (\textit{Spec.} Fig. C-J4.2)}}$$
$$R_n = \min \begin{vmatrix} 87.0 \text{ kips} \\ 81.0 \text{ kips} \end{vmatrix} + (0.5)(58)(1.75)$$
$$= 132 \text{ kips}$$
$$\phi R_n = 98.8 \text{ kips} > 30 \text{ kips, OK}$$



Controls by inspection

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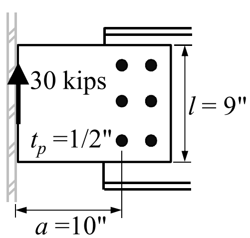
Extended Single-Plate Connection Example

Plate Buckling



$$M_u = V_u a = (30)(10) = 300 \text{ kip-in.}$$
$$\lambda = \frac{al}{t_p^2} = \frac{(10)(9)}{0.5^2} = 360$$
$$\lambda_p = 0.08E / F_y$$
$$= (0.08)(29000/36) = 64.4$$
$$\lambda_r = 1.9E / F_y = 1530$$
$$\lambda_p < \lambda < \lambda_r \Rightarrow \text{ Inelastic Buckling}$$
$$M_y = F_y S_x = F_y (t_p l^2 / 6) = (36)(1/2)(9^2)/6 = 243 \text{ kip-in.}$$
$$M_p = 1.5 M_y = 365 \text{ kip-in. (rectangle)}$$

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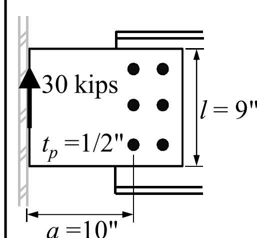
Extended Single-Plate Connection Example

Plate Buckling



Conservatively: $C_b = 1.84$

$$M_n = C_b \left(1.52 - 0.274\lambda \frac{F_y}{E}\right) M_y \le M_p$$
$$= 1.84\left(1.52 - 0.274(360)\frac{36}{29000}\right)243$$
$$= 625 \text{ kip-in.} > M_p = 365 \text{ kip-in.}$$
$$\phi M_n = \phi M_p = (0.9)(365 \text{ kip-in.})$$
$$= 329 \text{ kip-in.} > M_u = 300 \text{ kip-in., OK}$$

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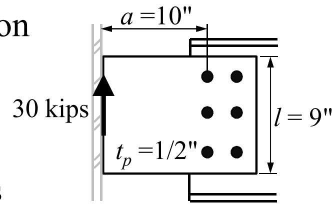
Extended Single-Plate Connection Example

Shear and Flexure Interaction

$V_u = 30$ kips

$M_u = V_u a = 300$ kip-in.



$$\phi_v V_n = \phi_v 0.6 F_y A_{gv} = 97.2 \text{ kips}$$

$$\phi_b M_n = \phi_b F_y Z_{pl} = (0.9)(36)\frac{(1/2)(9^2)}{4} = 328 \text{ kip-in.}$$

$$\left(\frac{V_u}{\phi_v V_n}\right)^2 + \left(\frac{M_u}{\phi_b M_n}\right)^2 \le 1.0$$

$$\left(\frac{30}{97.2}\right)^2 + \left(\frac{300}{328}\right)^2 = 0.932 < 1.0, \text{ OK}$$

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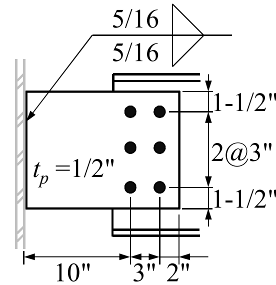
Extended Single-Plate Connection Example

Weld Strength



$$w \ge \frac{5}{8} t_p = \frac{5}{8}(1/2) = 5/16 \text{ in.}$$

Table J2.4: min is 3/16 in.

5/16 in. welds are adequate

Connection is adequate for $V_u = 30$ kips

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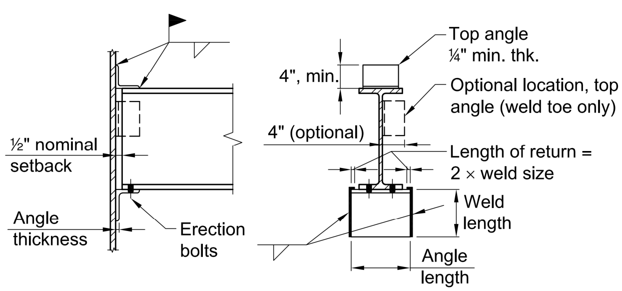
WELDED UNSTIFFENED SEATED CONNECTIONS



(b) All-welded

Fig. 10-7. Unstiffened seated connections.

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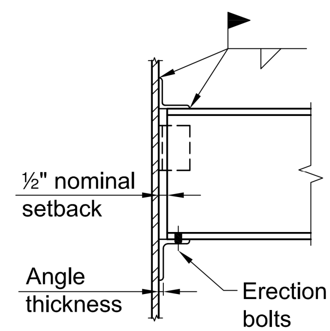
Welded Unstiffened Seated Connections

Advantages
- Few Parts
- Few Bolts
- Convenient at the web of a column

Disadvantages
- Requires Top Angle
- Limited strength
- Cannot Resist Axial Force



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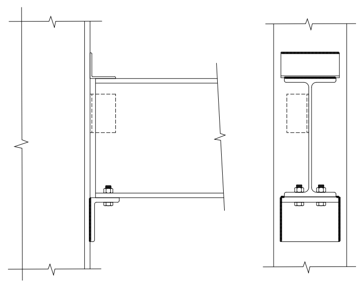
Welded Unstiffened Seated Connections

Limit States

- Beam Web Local Yielding – *Spec.* J10.2
- Beam Web Local Crippling – *Spec.* J10.3
- Seat Angle Bending
- Seat Angle Shear Yielding
- Weld Eccentric Shear Rupture



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Welded Unstiffened Seated Connections

Design Model for Angle Flexure

l_b is the max required for

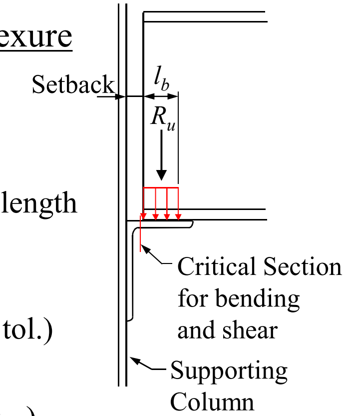
Web local yielding, $l_{b,WLY}$

Web local crippling, $l_{b,WLC}$

If l_b + Setback > horizontal leg length then → NG

Setback = 1/2 in. (nominal) + 1/4 in. (beam length tol.) = 3/4 in.

Section J10.2: $l_b \geq k$ (taken as k_{des})



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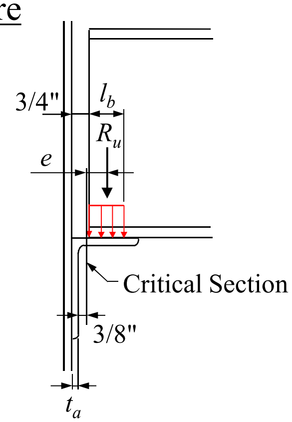
Welded Unstiffened Seated Connections

Design Model for Angle Flexure

$$M_u = R_u e$$

$$l_b = \max(l_{bWLY}, l_{bWLC})$$
$$\text{but} \geq k_{des}$$

$$e = 3/4 \text{ in.} + l_b/2 - (t_a + 3/8 \text{ in.})$$
$$= l_b/2 + 3/8 \text{ in.} - t_a$$



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Welded Unstiffened Seated Connections

Design Model for Angle Flexure

Required Bearing Length for Web Local Yielding

$$\phi R_n = \phi F_{yw} t_w (2.5k + l_b) \qquad \phi = 1.00$$

$$R_u = \phi F_{yw} t_w (2.5k + l_{bWLY})$$

$$l_{bWLY} = \frac{R_u}{F_{yw} t_w} - 2.5k$$

Use k_{des}.

Design Aid: *Manual* Table 9-4

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Welded Unstiffened Seated Connections

Design Model for Angle Flexure

Required Bearing Length for Web Local Crippling

when $l_b/d \le 0.2$

$$l_{bWLC} = \frac{d}{3}\left[\frac{R_u}{\phi 0.40 t_w^2 Q_f}\sqrt{\frac{t_w}{EF_{yw}t_f}} - 1\right]\left(\frac{t_f}{t_w}\right)^{1.5}$$

when $l_b/d > 0.2$

$$l_{bWLC} = \frac{d}{4}\left\{\left[\frac{R_u}{\phi 0.40 t_w^2 Q_f}\sqrt{\frac{t_w}{EF_{yw}t_f}} - 1\right]\left(\frac{t_f}{t_w}\right)^{1.5} + 0.2\right\}$$

$\phi = 0.75$

Design Aid: *Manual* Table 9-4

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Welded Unstiffened Seated Connections

Design Model for Angle Flexure (*Spec.* J4.5)

Required angle thickness from horizontal leg bending:

$$t_{req} = \sqrt{\frac{4R_u e}{\phi F_y L_a}}$$

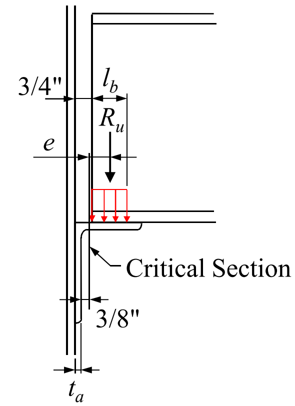
where

$e = l_b/2 + 3/8 \text{ in.} - t_a$

$\phi = 0.90$

F_y = angle yield stress

L_a = angle length



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Welded Unstiffened Seated Connections

Angle Shear Yielding (*Spec.* Eq. J4-3)

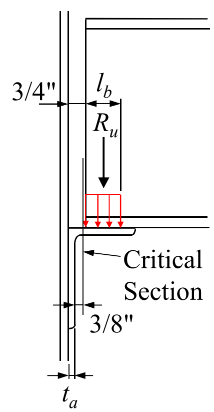
$$R_u \le \phi R_n$$

$$\phi R_n = \phi 0.6 F_y A_{gv}$$

where

$\phi = 1.00$

$A_{gv} = L_a t_a$



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Welded Unstiffened Seated Connections

Weld Rupture Strength

Eccentric Weld Rupture Strength:

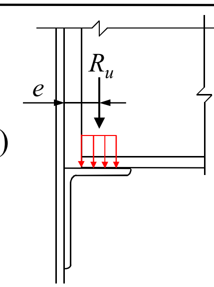
Elastic Method (*Manual* page 10-72)

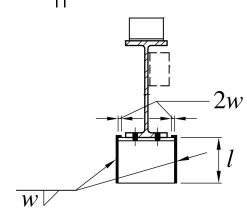
$$\phi R_n = 2\left(\frac{1.392Dl}{\sqrt{1 + 20.25e^2 / l^2}}\right)$$

D = number of 1/16 in. in w

Derivation is similar to that for knife connections except returns are considered.

Can also use ICoR Method in *Manual* Table 8-4.





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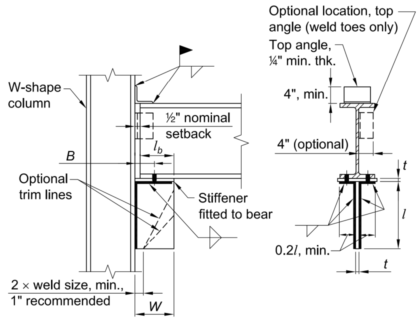
WELDED STIFFENED SEATED CONNECTIONS



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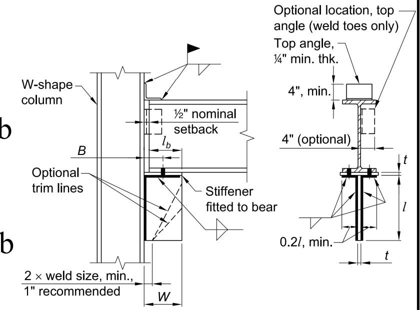
Welded Stiffened Seated Connections

Advantages
- Erection Safety
- Few Parts
- Few Bolts
- Convenient column web connection

Disadvantages
- Introduces Column Web Limit States



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Welded Stiffened Seated Connections

Limit States
- Beam Web Local Yielding
- Beam Web Local Crippling
- Strength of Stiffener Plate
- Eccentric Shear of Welds
- Column Base Metal
- Column Web Punching Shear

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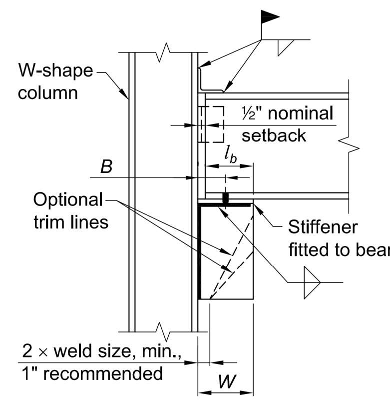
Welded Stiffened Seated Connections

- Simplified approach for bolted / welded covered here. *Manual* Pages 10-78 through 10-81. Table 10-8.
- Other Cases → *Steel Structures* by Salmon, Johnson, and Malhas (2009).



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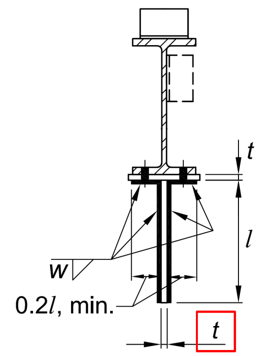
Welded Stiffened Seated Connections

Seat Stiffener Thickness

Manual Page 10-80

$$t \geq t_w \frac{F_{y,beam}}{F_{y,stiffener}}$$

$$t \geq \begin{cases} 2w \text{ if } F_{y,stiffener} = 36 \text{ ksi} \\ 1.5w \text{ if } F_{y,stiffener} = 50 \text{ ksi} \end{cases}$$



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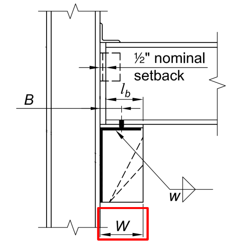
Welded Stiffened Seated Connections

Seat Width, *W*

- Set based on:
  - Setback
    - 1/2 in. nominal plus
    - 1/4 in. beam length tolerance.
  - Required l_b
    - Web Local Yielding
    - Web Local Crippling
    - Design Aid: *Manual* Table 9-4



F_y = 50 ksi

Table 9-4 (continued)
Beam Bearing Constants

$\phi R_n = \phi R_1 + l_b \phi R_2$ (WLY)

Shape	R_1/Ω kips ASD	ϕR_1 kips LRFD	R_2/Ω kip/in. ASD	ϕR_2 kip/in. LRFD	R_3/Ω kips ASD
W21×93	69.1	104	19.3	29.0	103
×83	57.5	86.3	17.2	25.8	81.3
×73	47.0	70.5	15.2	22.8	63.6

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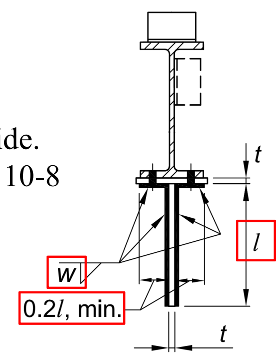
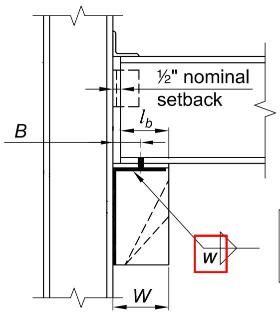
Welded Stiffened Seated Connections

Stiffener Depth and Weld

- T-Shaped Weld
  - Weld length, *l*
  - Horizontal length, 0.2*l* each side.
- Elastic Method → *Manual* Table 10-8





Stiffener-to-seat weld same size as stiffener-to-support weld.

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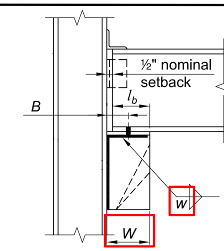
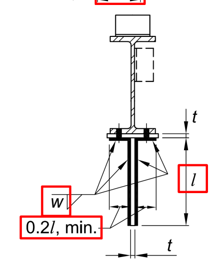
Welded Stiffened Seated Connections

Table 10-8
Bolted/Welded Stiffened Seated Connections
Weld Available Strength, kips

l, in.	Width of Seat, *W*, in.: 4 — w = 70-ksi Weld Size, in. — 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	5 — 70-ksi Weld Size, in. — 5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
6	22.7	34.0	28.4	42.5	34.0	51.1	39.7	59.6	23.5	35.2	28.2	42.2
7	29.9	44.9	37.4	56.1	44.9	67.3	52.4	78.6	31.2	46.9	37.5	56.2
8	37.8	56.7	47.2	70.8	56.7	85.0	66.1	99.2	39.8	59.8	47.8	71.7
9	46.1	69.2	57.7	86.5	69.2	104	80.7	121	49.1	73.7	59.0	88.5
10	54.9	82.3	68.6	103	82.3	123	96.0	144	59.0	88.5	70.8	106
11	63.9	95.8	79.8	120	95.8	144	112	168	69.4	104	83.3	125
12	73.1	110	91.4	137	110	165	128	192	80.2	120	96.2	144
13	82.5	124	103	155	124	186	144	217	91.3	137	110	164
14	92.1	138	115	173	138	207	161	242	103	154	123	185
15	102	152	127	191	152	229	178	267	114	171	137	206
16	111	167	139	209	167	250	195	292	126	189	151	227





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Welded Stiffened Seated Connections

Column Web Base Metal at Welds

t_{min} approach from *Manual* Eq. 9-2.

If stiffener only on one side,

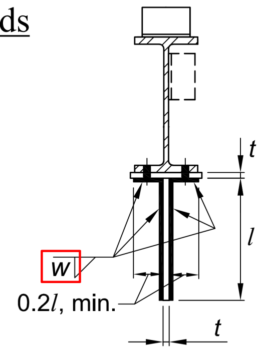
$$t_{min} = \frac{3.09D}{F_u}$$

If stiffener on both sides,

$$t_{min} = (t_{min})_{Left} + (t_{min})_{Right}$$

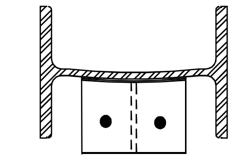
If column $t_w < t_{min}$,

$$\phi R_n = \left(\phi R_{n,Weld}\right)\frac{t_w}{t_{min}}$$

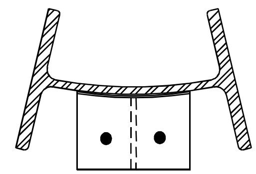


Welded Stiffened Seated Connections

Column Web Punching Shear



Stress Concentration
Weld Fracture



Flange Rotation
Decreased Column Strength

Solutions
- Stiff column web
- Web stiffener on opposite of web.

Welded Stiffened Seated Connections

Column Web Punching Shear

Manual Page 10-79 Simplified Approach for:

- W8x24 and heavier
- W10x33 and heavier
- W12x40 and heavier
- W14x43 and heavier

Welded Stiffened Seated Connections

Column Web Punching Shear

Simplified Approach

Top Angle $t \geq 1/4$ in.

Beam not welded to the seat plate.

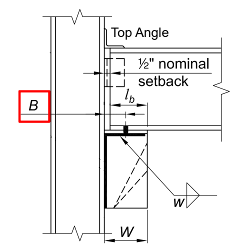
High-strength bolts with:

$$B \leq \max \begin{vmatrix} W/2 \\ \text{2-5/8 in.} \end{vmatrix}$$

When

W = 8 in. or 9 in.
3-1/2 in. < $B \leq W/2$
(Slightly different for W14x43)

See Sputo and Ellifritt (1991)



THE END
Thank You for Attending

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- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

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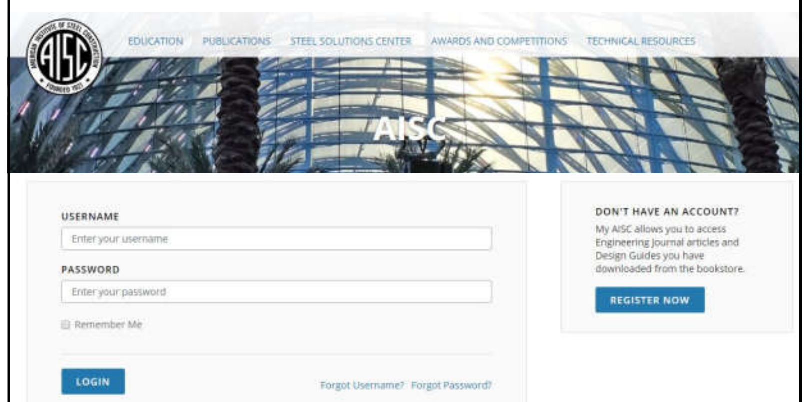

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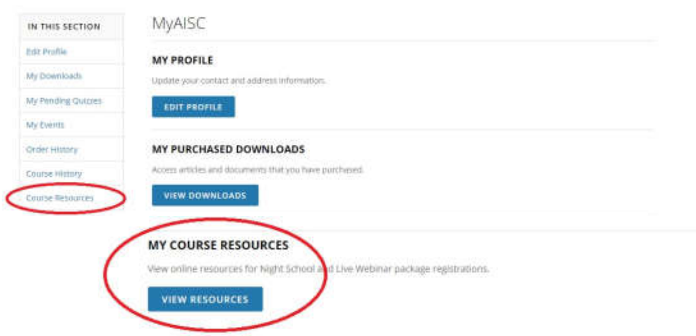

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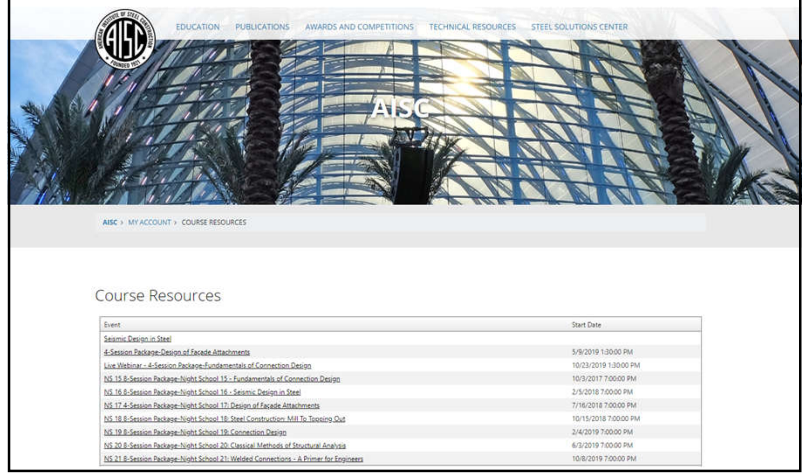

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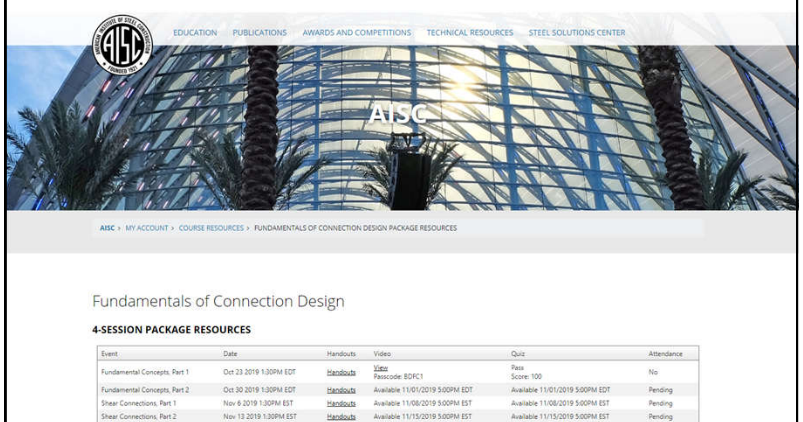

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