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Course Description

Session 4: October 24, 2017- Shear Connections, Part II

This live webinar will cover the design of welded and bolted single angle connections. Single plate connection design, including both conventional and extended single plate connections will be discussed. The differences between the two will be contrasted in design examples. The design of stiffened and unstiffened seated connections will also discussed. The presentation of stiffened seated connections will include a discussion on a simplified approach.

Learning Objectives

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

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

There's always a solution in steel.

Fundamentals of Connection Design

Session 4: Shear Connections, Part II

October 24,2017

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**

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SHEAR (FRAMING) CONNECTIONS PART II

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TOPICS

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

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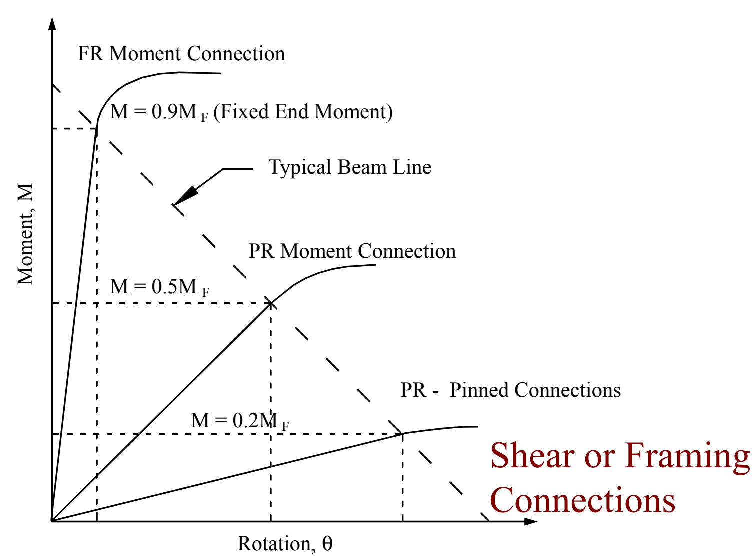
Shear (Framing) Connections



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SINGLE-ANGLE CONNECTIONS

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Single-Angle Connections

Advantages:

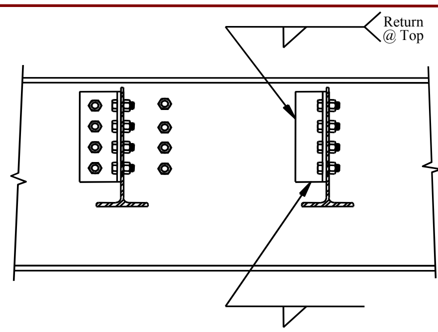
- Eliminates Double Sided Erection Problem
- Fewer Parts

Disadvantages:

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

Comment:

Not recommended for laterally unbraced beams.



Bolted and Welded Alternatives

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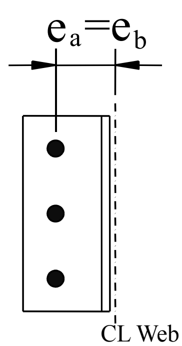
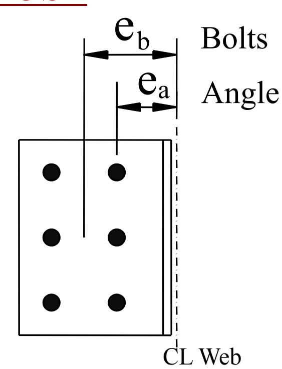
Single-Angle Connections: Bolted

Eccentricity Assumptions for OSL



Bolted Single Column



Bolted Double Column

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Single-Angle Connections: Bolted

Notes:

- Eccentricity is ignored on the beam side when the connection is a single column.
- Standard holes or short slots can be used on the beam side.
- Only standard holes should be used on the supporting member side.
- Single angle should be connected to supporting member in the shop.
- New Limit States: Angle Flexural Yielding and Rupture; Bolt Eccentric Shear and Bearing

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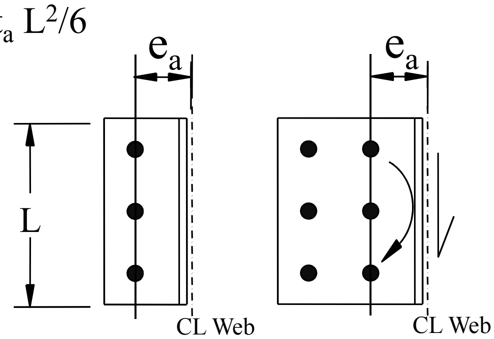
Single-Angle Connections: Bolted

Angle Flexural Yielding of OSL

$\phi V_n = 0.9\ F_y\ S_g\ /\ e_a$

$S_g = t_a\ L^2/6$



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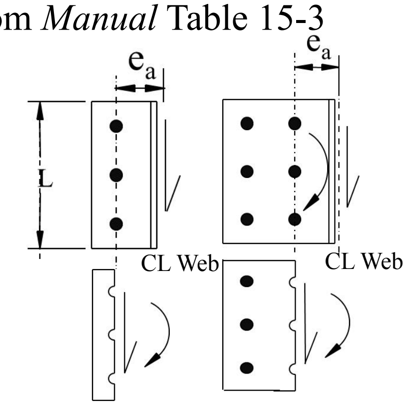
Single-Angle Connections: Bolted

Angle Flexural Rupture of OSL

$\phi V_n = 0.75\ F_u\ Z_{net}\ /\ e_a$

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



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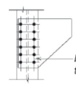
Single-Angle Connections: Bolted

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



# Bolts in One Vertical Row, *n*	Bracket Plate Depth, *d*, in.	Nominal Bolt Diameter, *d*, in. 3/4 — Bracket Plate Thickness, *t*, in. 1/4	3/4 — 3/8	3/4 — 1/2	3/4 — 5/8	3/4 — 3/4	7/8 — 3/8	7/8 — 1/2	7/8 — 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
16	48	102	153	204	255	306	144	192	240
18	54	129	194	258	323	387	182	243	304
20	60	159	239	319	398	478	225	300	375
22	66	193	289	386	482	579	272	363	454
24	72	230	344	459	574	689	324	432	540
26	78	269	404	539	673	808	380	507	634
28	84	312	469	625	781	937	441	588	735
30	90	359	538	717	896	1080	506	675	844
32	96	408	612	816	1020	1220	576	768	960
34	102	461	691	921	1150	1380	650	867	1080
36	108	516	775	1030	1290	1550	729	972	1220

Manual Table 15-3

$d'_h = d_h + 1/16$

$S = 3$ in.

$L_{ev} = 1½$ in.

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

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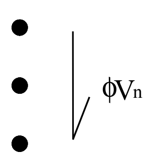
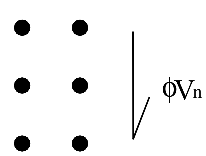
Single-Angle Connections: Bolted

Eccentric Shear of Bolts

Manual Ultimate Strength Method

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





C: Table 7-6   Tables 7-7 thru 7-13

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Single-Angle Connections: Bolted

Bearing and Tear Out

$\phi V_n = C\ (\phi r_{vb})$

where ϕr_{vb} is the bearing/tear out strength at the outermost bolt.

Note: Shear transfer strength at elements is minimum of C (ϕr_v) and C (ϕr_{vb}).

Single-Angle Connections: Bolted

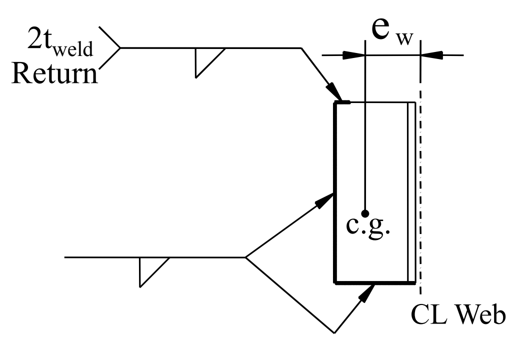
Recommended Minimum Angle Thicknesses

d_b (in.)	t_{min}(in.)
3/4	3/8
7/8	3/8
1	1/2

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

Single-Angle Connections: Welded

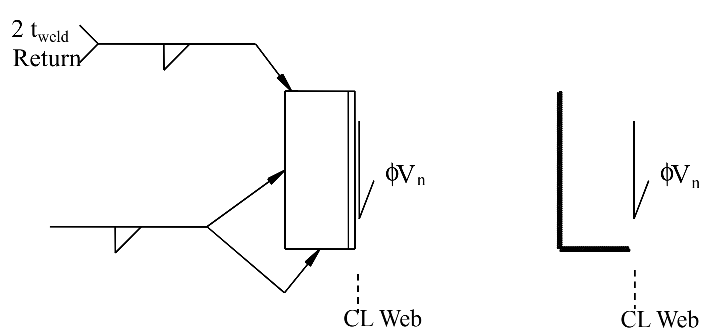
Eccentricity Assumptions for OSL:



Welded Connection

Single-Angle Connections: Welded

Eccentric Shear Strength of Weld



Manual Table 8-10

Single-Angle Connections: Welded

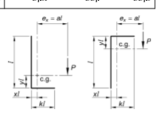
Table 8-10
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

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

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

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



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.86	2.04	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
0.50	1.15	1.29	1.44	1.60	1.77	1.95	2.13	2.31	2.50	2.70	2.90	3.33	3.78	4.25	4.74	5.23
0.60	0.999	1.12	1.25	1.39	1.54	1.70	1.87	2.04	2.21	2.40	2.59	2.99	3.42	3.87	4.34	4.82
0.70	0.879	0.987	1.10	1.22	1.35	1.50	1.66	1.82	1.98	2.15	2.32	2.71	3.11	3.55	4.00	4.47
0.80	0.783	0.878	0.978	1.09	1.20	1.34	1.48	1.63	1.78	1.94	2.11	2.46	2.85	3.27	3.70	4.15
0.90	0.704	0.790	0.879	0.976	1.08	1.20	1.33	1.48	1.62	1.77	1.92	2.26	2.63	3.02	3.43	3.86
1.0	0.639	0.717	0.797	0.885	0.983	1.09	1.21	1.35	1.48	1.62	1.76	2.08	2.43	2.80	3.20	3.61
1.2	0.538	0.603	0.671	0.745	0.828	0.922	1.03	1.14	1.26	1.38	1.51	1.79	2.10	2.44	2.80	3.18
1.4	0.464	0.520	0.579	0.643	0.715	0.796	0.888	0.991	1.10	1.21	1.32	1.57	1.85	2.15	2.48	2.83
1.6	0.408	0.457	0.508	0.564	0.628	0.700	0.783	0.874	0.972	1.07	1.17	1.40	1.65	1.93	2.22	2.54
1.8	0.363	0.407	0.453	0.503	0.560	0.625	0.699	0.782	0.871	0.957	1.05	1.26	1.49	1.74	2.01	2.31
2.0	0.328	0.367	0.408	0.454	0.505	0.564	0.632	0.706	0.788	0.867	0.952	1.14	1.35	1.58	1.84	2.11
2.2	0.298	0.334	0.372	0.413	0.460	0.514	0.576	0.644	0.719	0.792	0.870	1.04	1.24	1.45	1.69	1.94
2.4	0.274	0.306	0.341	0.379	0.422	0.472	0.529	0.592	0.661	0.728	0.801	0.960	1.14	1.34	1.56	1.79
2.6	0.253	0.283	0.315	0.350	0.390	0.437	0.489	0.547	0.611	0.674	0.741	0.890	1.06	1.24	1.45	1.67
2.8	0.235	0.263	0.293	0.325	0.363	0.406	0.455	0.509	0.568	0.628	0.690	0.829	0.986	1.16	1.35	1.56
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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Manual Table 8-10

$R_n = CC_1Dl$

$\phi = 0.75$

Parameters:

$C_1 = Exx/70$

$k \Rightarrow x$

$x \ \& \ a \Rightarrow C$

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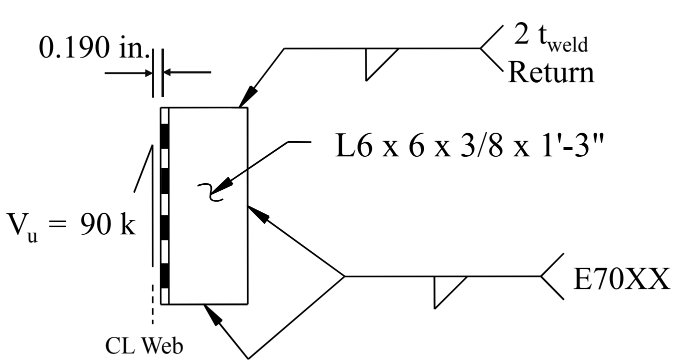
Single-Angle Connection Example

Example: Determine required weld size.

Beam W21x50 A992 $t_w = 0.380$ in.

Column W14x90 A992 $t_f = 0.710$ in.



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Single-Angle Connection Example

Manual Table 8-10 with l = 15 in.

kl = 6 in.

k = 6 / 15 = 0.4 → x = 0.057

xl = 0.057 x 15 = 0.855 in.

al = 0.38/2 + 6 – 0.855

= 5.34 in.

a = 5.34 /15 = 0.356

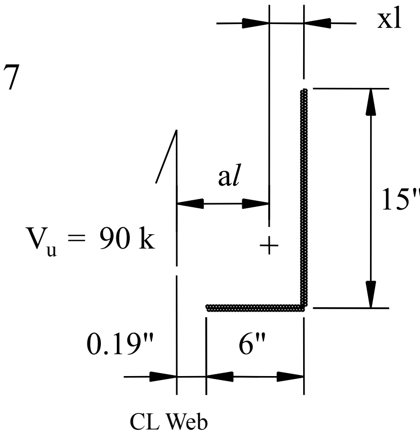


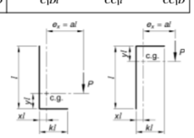
Table 8-10
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

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

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

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



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.86	2.04	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
0.50	1.15	1.29	1.44	1.60	1.77	1.95	2.13	2.31	2.50	2.70	2.90	3.33	3.78	4.25	4.74	5.23
0.60	0.999	1.12	1.25	1.39	1.54	1.70	1.87	2.04	2.21	2.40	2.59	2.99	3.42	3.87	4.34	4.82
0.70	0.879	0.987	1.10	1.22	1.35	1.50	1.66	1.82	1.98	2.15	2.32	2.71	3.11	3.55	4.00	4.47
0.80	0.783	0.878	0.978	1.09	1.20	1.34	1.48	1.63	1.78	1.94	2.11	2.46	2.85	3.27	3.70	4.15
0.90	0.704	0.790	0.879	0.976	1.08	1.20	1.33	1.48	1.62	1.77	1.92	2.26	2.63	3.02	3.43	3.86
1.0	0.639	0.717	0.797	0.885	0.983	1.09	1.21	1.35	1.48	1.62	1.76	2.08	2.43	2.80	3.20	3.61
1.2	0.538	0.603	0.671	0.745	0.828	0.922	1.03	1.14	1.26	1.38	1.51	1.79	2.10	2.44	2.80	3.18
1.4	0.464	0.520	0.579	0.643	0.715	0.796	0.888	0.991	1.10	1.21	1.32	1.57	1.85	2.15	2.48	2.83
1.6	0.408	0.457	0.508	0.564	0.628	0.700	0.783	0.874	0.972	1.07	1.17	1.40	1.65	1.93	2.22	2.54
1.8	0.363	0.407	0.453	0.503	0.560	0.625	0.699	0.782	0.871	0.957	1.05	1.26	1.49	1.74	2.01	2.31
2.0	0.328	0.367	0.408	0.454	0.505	0.564	0.632	0.706	0.788	0.867	0.952	1.14	1.35	1.58	1.84	2.11
2.2	0.298	0.334	0.372	0.413	0.460	0.514	0.576	0.644	0.719	0.792	0.870	1.04	1.24	1.45	1.69	1.94
2.4	0.274	0.306	0.341	0.379	0.422	0.472	0.529	0.592	0.661	0.728	0.801	0.960	1.14	1.34	1.56	1.79
2.6	0.253	0.283	0.315	0.350	0.390	0.437	0.489	0.547	0.611	0.674	0.741	0.890	1.06	1.24	1.45	1.67
2.8	0.235	0.263	0.293	0.325	0.363	0.406	0.455	0.509	0.568	0.628	0.690	0.829	0.986	1.16	1.35	1.56
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

Single-Angle Connection Example

Manual Table 8-10

$R_n = CC_1Dl$

$\phi = 0.75$

$D_{req'd} = V_u / (\phi C\ C_1\ l)$

Parameters:

C_1 = Exx/70 = 1.0

k = 0.4 ⇒ x = 0.057

a = 0.356

x & a ⇒ C = 2.18

Single-Angle Connection Example

From *Manual* Table 8-10:

$C = 2.18$ by interpolation $\quad C_1 = 1.0$

$D_{req'd} = V_u / (\phi C\, C_1\, l)$

$= 90 / (0.75 \times 2.18 \times 1.0 \times 15)$

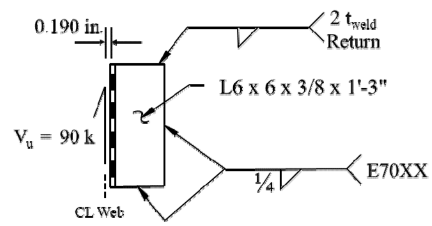
$= 3.66 \rightarrow 4/16 = 1/4$ in.

Min. weld = 3/16 in. (Angle t_a = 3/8 in.)

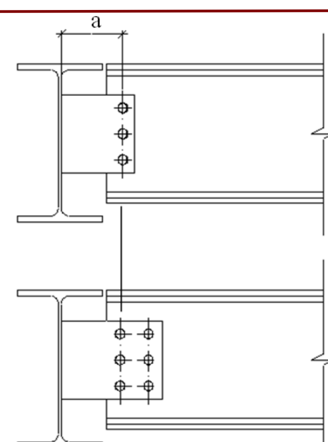
Max. weld = (3/8 - 1/16)

= 5/16 in.

<u>Use 1/4 in. Fillet weld</u>



SINGLE-PLATE (SHEAR TAB) CONNECTIONS



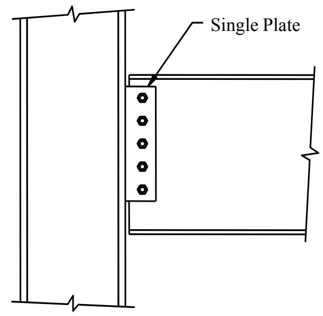
Single-Plate Connections

Advantages:

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

Disadvantages:

- Stiffer than Other Types
- Requires Careful Design



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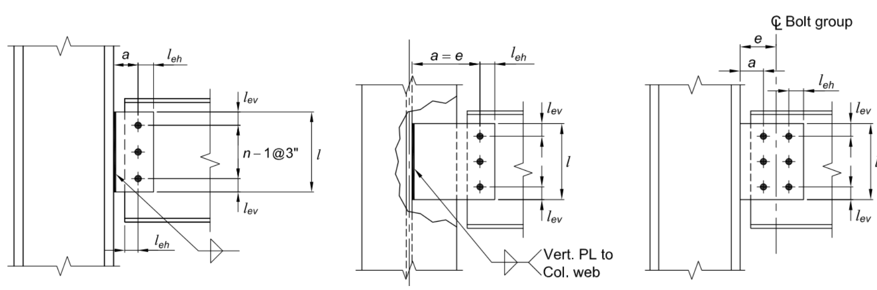
Single-Plate Connections



(Also used to eliminate beam copes)

Conventional Single-Plate

Maximum plate thickness
a $\leq$ 3 ½ in.
2 $\leq$ n = no. of bolts $\leq$ 12
L_{eh}= $\geq 2d_b$
$L_{ev} \geq$ limits in *Spec.* Table J3.4

Extended Single-Plates

Maximum plate thickness
L_{eh} and L_{ev} satisfy *Spec.* Table J3.4

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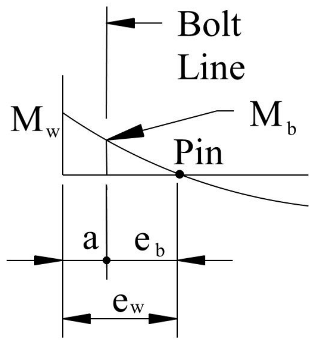
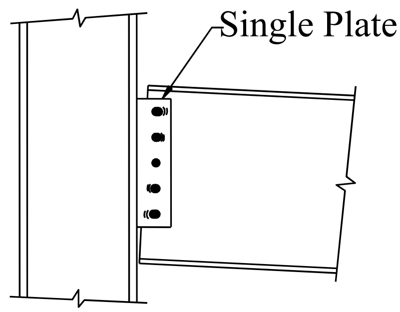
Single-Plate Connections

Plate Rigidity

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





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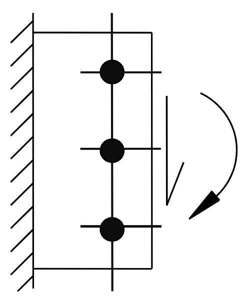
Single-Plate Connections

Plate Limit States

- Shear Yielding
- Shear Rupture
- Block Shear
- Shear Transfer at Elements
- Plate Buckling
- Eccentric Bolt Shear



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Conventional Single-Plate Connections

Conventional Connection Geometric Limitations

Maximum t_p or t_w from *Manual* Table 10-9

$L_{eh} \geq 2d_b$ for the plate or beam web.

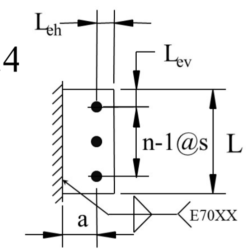
$L_{ev} \geq$ limits in *Specification* Table J3.4

$a \leq 3\frac{1}{2}$ in.

$2 \leq n =$ No. of Bolts ≤ 12

$L > T/2$

$t_{weld} \geq 5/8\ t_p$ on both sides of plate



Note: $t_{weld} \geq 5/8 t_p$ is sufficient to develop the plate; no strength calculations are required.

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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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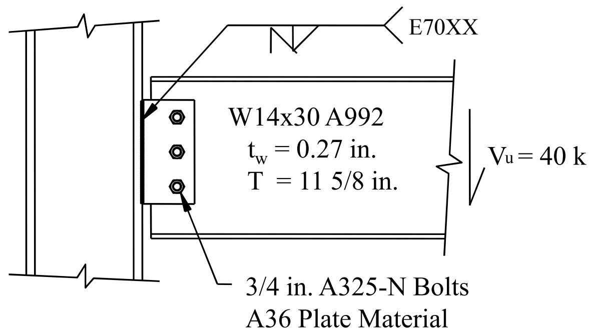
Conventional Single-Plate Connection Ex.

Example: Determine required number of bolts and plate and weld sizes, for the conventional single plate connection shown.



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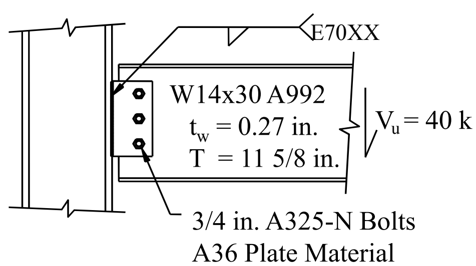
Conventional Single-Plate Connection Ex.

Bolt Shear Rupture

Try 3 – 3/4 in. A325-N Bolts

$$\phi V_n = (\phi F_{nv} A_b)n$$
$$= (0.75 \times 54 \times 0.4418)(3) = 17.9 \times 3$$
$$= 53.7 \text{ k} > V_u = 40 \text{ k}$$

OK but need to check eccentric shear strength.



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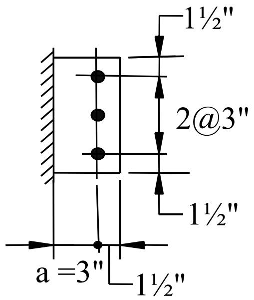
Conventional Single-Plate Connection Ex.

Assume Plate Geometry



Conventional Plate Limitations:

$2 \leq n \leq 12$

a = 3 in. < 3 ½ in.

$2d_b$= 2x3/4 =1.5 in. ≤ L_{eh}=1.5 in.

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

L = 9 in. > T/2 = 11.625/2 = 5.81 in.

All OK

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Conventional Single-Plate Connection Ex.

Max. Plate Thickness *Manual* Table 10-9)

2-5 Bolts

t_{max} = d/2 + 1/16 = (3/4)/2 + 1/16 = 0.4375 in.

Try 1/4 in. plate < 0.4375 in. (Note: t_w = 0.27 in.)

Eccentricity from *Manual* Table 10-9

e = a/2 = 3.0/2 = 1.5 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

ϕr_v = 17.9 k (3/4 in. A325-N Bolt, *Manual* Table 7-1)

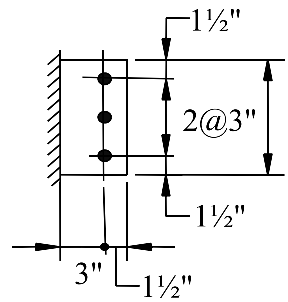
n = 3 e_x = 1.5 in.

From *Manual* Table 7-6, C= 2.47

$\phi V_n = C\ \phi r_n$

$= 2.47 \times 17.9$

$= 44.2$ k > 40 k OK



Conventional Single-Plate Connection Ex.

Check Eccentric Bolt Shear Strength

Table 7-6
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

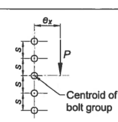
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$R_n = C \times r_n$

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*										
		2	3	4	5	6	7	8	9	10	11	12
3	1	1.63	2.71	3.75	4.77	5.77	6.77	7.76	8.75	9.74	10.7	11.7
	2	1.18	2.23	3.32	4.39	5.45	6.48	7.51	8.52	9.53	10.5	11.5
	3	0.88	1.75	2.81	3.90	4.98	6.06	7.12	8.17	9.21	10.2	11.3
	4	0.69	1.40	2.36	3.40	4.47	5.56	6.64	7.72	8.78	9.84	10.9
	5	0.56	1.15	2.01	2.96	3.98	5.05	6.13	7.22	8.30	9.38	10.4
	6	0.48	0.97	1.73	2.59	3.55	4.57	5.63	6.70	7.79	8.87	9.96
	7	0.41	0.83	1.51	2.28	3.17	4.13	5.15	6.20	7.28	8.36	9.44
	8	0.36	0.73	1.34	2.04	2.85	3.75	4.72	5.73	6.78	7.85	8.93
	9	0.32	0.65	1.21	1.83	2.59	3.42	4.34	5.31	6.32	7.36	8.42
	10	0.29	0.59	1.09	1.66	2.36	3.14	4.00	4.92	5.89	6.90	7.94
	12	0.24	0.49	0.92	1.40	2.00	2.68	3.44	4.27	5.15	6.09	7.06
	14	0.21	0.42	0.79	1.21	1.74	2.33	3.01	3.75	4.55	5.41	6.31
	16	0.18	0.37	0.70	1.06	1.53	2.06	2.67	3.33	4.06	4.85	5.68
	18	0.16	0.33	0.62	0.95	1.37	1.84	2.39	3.00	3.66	4.38	5.15
	20	0.15	0.29	0.56	0.85	1.24	1.67	2.16	2.72	3.33	3.99	4.70
	24	0.12	0.25	0.47	0.71	1.03	1.40	1.82	2.29	2.81	3.37	3.99
	28	0.11	0.21	0.40	0.61	0.89	1.20	1.57	1.97	2.42	2.92	3.45
	32	0.09	0.18	0.35	0.54	0.78	1.05	1.37	1.73	2.13	2.57	3.04
	36	0.08	0.16	0.31	0.48	0.69	0.94	1.22	1.54	1.90	2.29	2.72
	C', in.	2.94	5.89	11.3	17.1	25.1	33.8	44.4	55.9	69.2	83.5	100
	1	1.86	2.88	3.88	4.87	5.86	6.84	7.83	8.81	9.80	10.8	11.8

Manual Table 7-6

$R_n = C \times r_n$

$\phi = 0.75$

Parameters:

s =3 in.

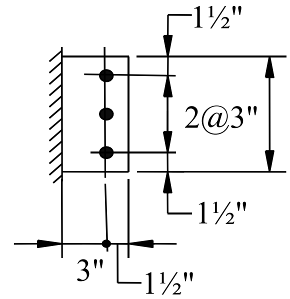
n = 3 & e_x = 1.5 in.

$\Rightarrow C = 2.47$

Conventional Single-Plate Connection Ex.

PLATE LIMIT STATES

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



Shear Yielding

$$\phi V_n = 1.0\,(0.6\,F_y)\,A_g$$
$$= 1.0\,(0.6 \times 36)\,(0.25 \times 9)$$
$$= 48.6 \text{ k} > 40 \text{ k OK}$$

Shear Rupture

$$\phi V_n = 0.75\,(0.6\,F_u)\,A_n$$
$$= 0.75(0.6 \times 58)(9 - 3 \times 7/8)(1/4)$$
$$= 41.6 \text{ k} > 40 \text{ k OK}$$

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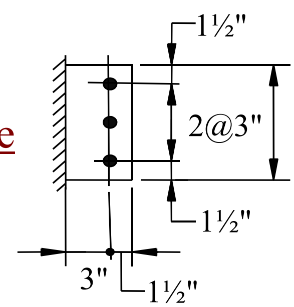
Conventional Single-Plate Connection Ex.

Block Shear

$\phi V_n = 41.9$ k > 40 k OK



Shear Transfer Btw. Web & Plate

Plate: Bearing = 26.1 k
Edge T.O. = 19.0 k
Other T.O. = 38.0 k

Bolt Shear Rupture = 23.9 k

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

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Conventional Single-Plate Connection Ex.

Note: Beam web (t_w = 0.27 in. A992 steel) is stronger than plate (t_p = 0.25 in. A36 steel) and will not control bearing and tear-out strengths.

Required Fillet Weld Size

$t_{weld} = 5/8t_p = 5/8(¼) = 5/32$ in. $\Longrightarrow$ 3/16 in. B.S.

No other calculations needed.

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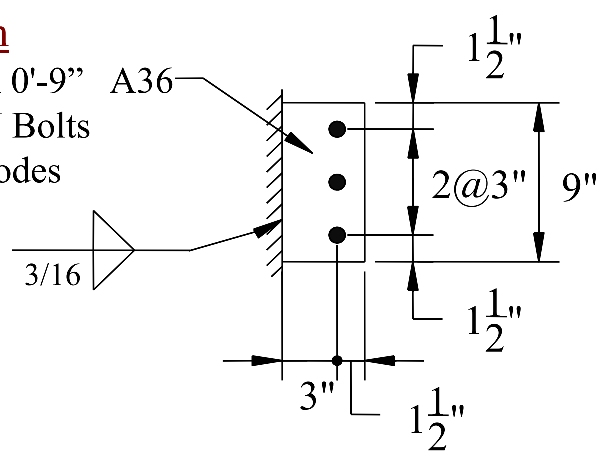
Conventional Single-Plate Connection Ex.

Final Design

PL ¼ x 4-½ x 0'-9" A36

¾ in. A325-N Bolts

E70xx Electrodes



$V_u = 40 \text{ k} \leq \phi V_n = 41.6 \text{ k}$ (Shear Rupture)

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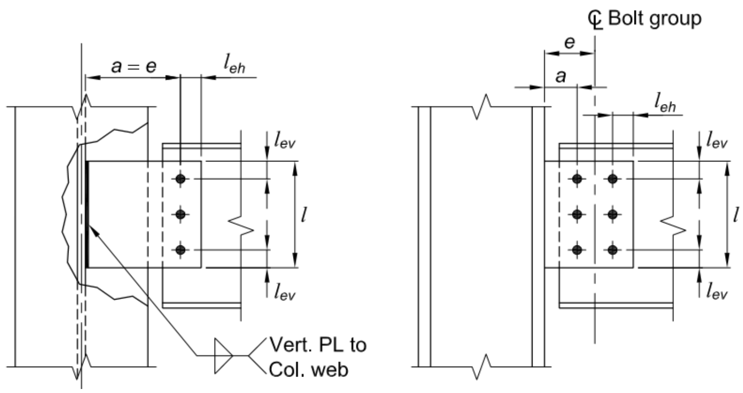
Extended Single-Plate Connections



a is to first column of bolts
e is to center of bolt group

Extended Single-Plate Connections

Limitations

- No limit on a-distance
- 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.

- Length of plate ≥ T/2

Extended Single-Plate Connections

Limitation: Maximum Plate Thickness

Plate Flexural Strength ≤ Bolt Group Flexural Strength

$$F_y S_g \leq M_{max} = F_{nv} A_b C' / 0.90$$

F_y = Plate yield stress
S_g = Plate elastic section modulus = $t_p L^2/6$
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)
t_p = Plate thickness
L = Depth of plate

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Extended Single-Plate Connections

Limitation: Maximum Plate Thickness

Thus, the maximum single-plate thickness is:

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

Or the "plowing rules" as for conventional connections can be used. For single column of bolts:

t_p or t_w ≤ thickness from *Manual* Table 10-9

$l_{eh} = \geq 2d_b$ for both plate and web

For connections with double columns of bolts both the plate and beam web must satisfy both rules above.

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Extended Single-Plate Connections

Limitation: Maximum Plate Thickness

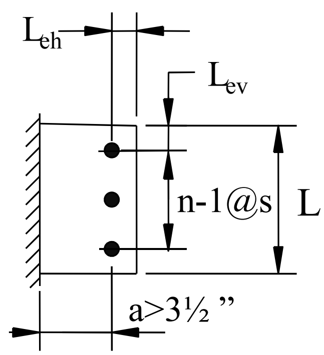
Table 10-9
Design Values for Conventional Single-Plate Shear Connections

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*	$d/2 - 1/16$

Extended Single-Plate Connections

Plate Limit States

- Shear Yielding
- Shear Rupture
- Block Shear
- Combined Shear + Moment Strength
- Plate Buckling



Extended Single-Plate Connections

Combined Shear + Moment Strength Using Von Mises Yield Criterion

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

$\phi_v = 1.0 \quad \phi_b = 0.9$

V_u = Required shear strength

$V_n = 0.6\, F_y A_g$

M_u = Required flexural strength = $V_u a$

$M_n = F_y Z_{pl} = F_y\,(t_p L^2/4)$

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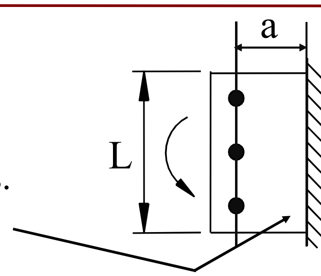
Extended Single-Plate Connections

Plate Buckling

Buckling check required for extended connections.
a > 3½ in. or w/ 2+ cols.



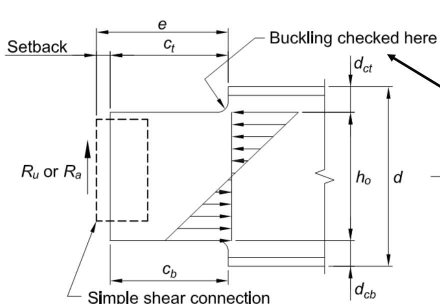


Plate Buckling is checked assuming the extended plate is equivalent to the web of a double coped beam.

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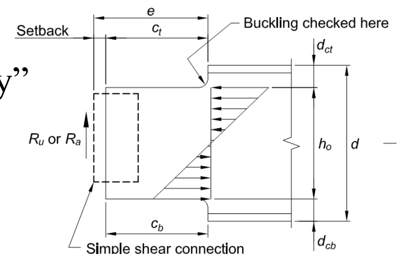
Extended Single-Plate Connections

Plate Buckling

Use "double coped beam theory" as described in Session 3 and *Manual* pp. 9-9 and 9-10.



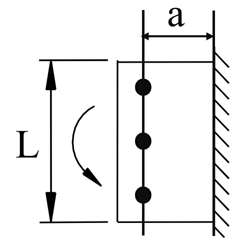
$$\phi M_n = 0.9 M_n$$

where M_n is from *Spec.* F11

$L_b = a$

$d = L$, length of plate

$C_b \geq 1.84$ (Errata)



Extended Single-Plate Connections

Plate Buckling (Modified *Spec.* F11 Eqns.)

$\lambda \leq \lambda_p$

$M_n = M_p = F_y Z_p$ (*Spec.* Eqn. F11-1)

$\lambda_p < \lambda \leq \lambda_r$

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

$\lambda > \lambda_r$

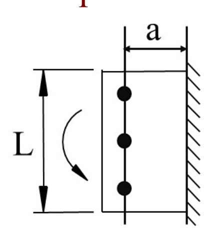
$M_n = C_b(1.9E/\lambda)S_x \leq M_p$ (*Spec.* Eqn. F11-3)

where

$\lambda = aL/t_p^2$ $\quad Z_p = t_p L^2/4$

$\lambda_p = 0.08E/F_y$

$\lambda_r = 1.9E/F_y$



Extended Single-Plate Connections

Other Limit States:

Eccentric Bolt Shear Strength

Shear Transfer at Elements

Fillet Weld Strength: 5/8 x t_p on each side of plate.

Note: 5/8 x t_p is sufficient to develop the plate in shear plus tension. A strength check is not needed.

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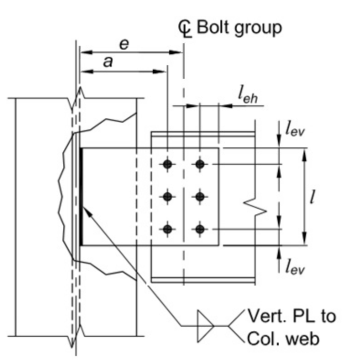
Extended Single-Plate Connection Example

Example: Determine if the extended single plate connection shown is adequate if V_u = 21 kips.



Beam: W14x43 A992
t_w = 0.305 in.

Column: W14x90 A992
t_w = 0.440 in.

Plate: A36

7/8 in. A325-N Bolts

E70xx Electrode

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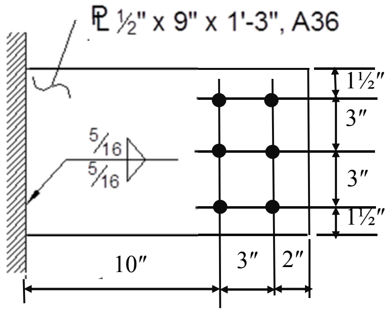
Extended Single-Plate Connection Example

Plate Geometry



$a = 10$ in.
$e = 11.5$ in.

Limitation Checks

1. Check Min. Edge Distances

$L_{eh} = 2$ in.

$L_{ev} = 1\frac{1}{2}$ in.

From *Specification* Table J3.4 for 7/8 in. bolts:

$L_{edge,\,min} = 1\text{-}1/8$ in. OK

Use $L_{eh} = 2$ in. at beam end.

$L > T/2$ by inspection.

Extended Single-Plate Connection Example

2. Check Maximum Plate Thickness

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

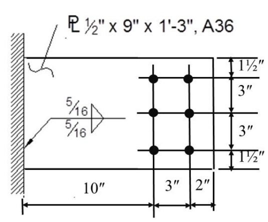
7/8 in. A325-N Bolt:

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

$A_b = 0.601$ in.^{2}

Plate: $F_y = 36$ ksi $L = 9$ in.

C' from *Manual* Table 7-7 with $s = 3$ in. $n = 3$ in.



Extended Single-Plate Connection Example

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

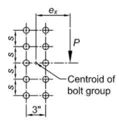
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$R_n = Cr_n$

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



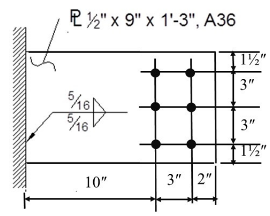
s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	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	$\boxed{15.8}$	26.0	38.7	54.2	72.2	93.1	117	143	172	204
	2	0.84	3.24	5.39	7.47	9.51	11.5	13.5	15.5	17.5	19.5	21.5	23.4
	3	0.65	2.79	4.93	7.08	9.17	11.2	13.3	15.3	17.3	19.3	21.3	23.3

Manual Table 7-7

Parameters:

s =3 in. & n = 3

$\Rightarrow C' = 15.8$

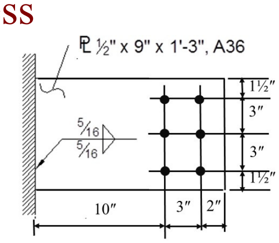


Extended Single-Plate Connection Example

2. Check Maximum Plate Thickness

$$t_{max} = \frac{6F_{nv}A_bC'}{0.9F_yL^2} = \frac{6(54)(0.601)(15.8)}{0.9(36)(9.0^2)}$$

$$= 1.18\text{in.} > t_p = 0.5\text{ in.}\quad \text{OK}$$



Alternatively for 2 columns of bolts:

t_p and $t_w \leq d_b/2 + 1/16$ in. (*Manual* Table 10-9)

$t_p = 0.5$ in. $< 0.875/2 + 1/16 = 0.5$ in. OK

$t_w = 0.305$ in. < 0.5 in. OK

$l_{eh} = 2$ in. $\geq 2d_b = 2\ (0.875) = 1.75$in for both plate and web. OK

½ in. Plate is OK

Extended Single-Plate Connection Example

Limit State Checks

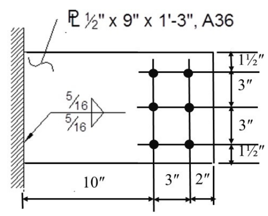
1. Eccentric Bolt Shear Rupture

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

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

C = 1.30

$\phi R_v = 1.30\ (24.3)$
$= 31.6 \text{ k} > 21 \text{ k}$ OK



Extended Single-Plate Connection Example

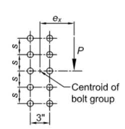
Table 7-7
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

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

$R_n = Cr_n$
or

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

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



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n: 1	2	3	4	5	6	7	8	9	10	11	12
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
	2	0.84	3.24	5.39	7.47	9.51	11.5	13.5	15.5	17.5	19.5	21.5	23.4
	3	0.65	2.79	4.93	7.08	9.17	11.2	13.3	15.3	17.3	19.3	21.3	23.3

Manual Table 7-7

$R_n = C \times r_n$
$\phi = 0.75$

Parameters:
s =3 in.
n = 3 & $e_x = 11.5$ in.
$\Rightarrow C = 1.30$

$\phi R_n = 1.30 \times 24.3$
$= 31.6$ k

Extended Single-Plate Connection Example

2. Shear Transfer Between Elements

<u>Plate:</u>

Brg. (2.4x58)(0.875x0.5) = 60.9 k

Edge T.O. (1.2x58)(1.5-15/32)(0.5) = <u>35.9</u> < 60.9 k

Other T.O. (1.2x58)(3.0-15/16)(0.5) = 71.8 k > <u>60.9 k</u>

<u>Bolt</u>

r_v = 54(0.601) = 32.4 k

<u>Web</u> (Not coped; Edge Tear Out is not a Limit State)

Brg. (0.305/0.5)(60.9) = 37.1 k

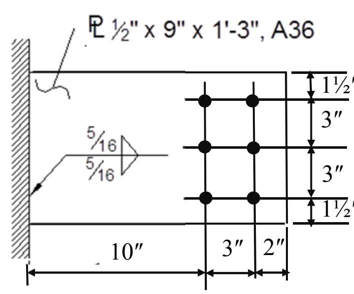
Other T.O. (0.305/0.5)(71.8) = 43.8 k > <u>37.1 K</u>

Bolt Shear Rupture Strength Controls Transfer Strength

ϕR_v = = 31.6 k > 21 k OK (as previous)

Extended Single-Plate Connection Example

3. Shear Yielding



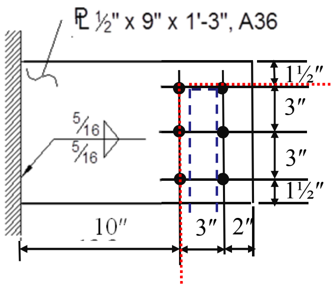
$$\phi R_n = 1.0(0.6xF_y)A_g$$
$$= 1.0(0.6x36)(0.5x9)$$
$$= 97.2\text{ k} > 21\text{ k}$$ <u>OK</u>

4. Shear Rupture

$$\phi R_n = 0.75(0.6F_u)A_{nv}$$
$$= 0.75(0.6x58)(9-3x1)(0.5)$$
$$= 78.3\text{ k} > 21\text{ k}$$ <u>OK</u>

Extended Single-Plate Connection Example

5. Block Shear



Controls by inspection

$$R_n = \min\left| \begin{matrix} 0.6F_uA_{nv} \\ 0.6F_yA_{gv} \end{matrix} \right. + U_{bs}F_uA_{nt}$$

$A_{gv} = 7.5x0.5 = 3.75 \text{ in}^2$

$A_{nv} = (7.5 - 2.5x1.0)(0.5) = 2.50 \text{ in}^2$

$U_{bs} = 0.5 \rightarrow$ *Specification* Fig. C – J4.2

$A_{nt} = (5 - 1.5x1.0)(0.5) = 1.75 \text{ in}^2$

$$R_n = \min\left| \begin{matrix} (0.6x58)(2.50) = 87.0 \\ (0.6x36)(3.75) = 81.0 \end{matrix} \right. + 0.5x58x1.75$$

$\phi R_n = 0.75(81.0 + 50.8) = 98.8 \text{kips} > 21 \text{ kips}$

OK

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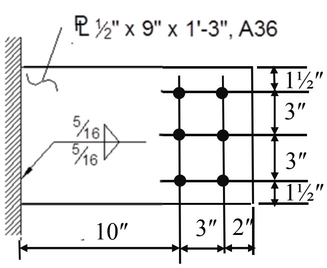
Extended Single-Plate Connection Example

6. Plate Buckling



$$\phi V_n = \frac{\phi M_n}{a}$$

$$\lambda = \frac{aL}{t^2} = \frac{(10)(9)}{0.5^2} = 360$$

$$\lambda_p = 0.08\frac{E}{F_y} = 0.08\frac{29000}{36} = 64.4$$

$$\lambda_r = 1.9\frac{E}{F_y} = 1.9\frac{29000}{36} = 1531$$

$\lambda_p < \lambda < \lambda_r$, so inelastic buckling

$M_y = F_yS_x = F_y(tL^2/6) = 36\,[0.5(9.0^{2)})/6] = 243 \text{ kip-in.}$

$M_p = F_yZ_x = F_y(tL^2/4) = 36\,[0.5(9.0^{2)})/4] = 365 \text{ kip-in.}$

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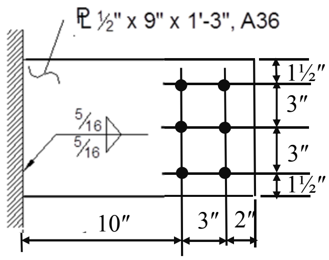
Extended Single-Plate Connection Example

6. Plate Buckling



Conservatively: $C_b = 1.84$

$$M_n = C_b\left(1.52 - 0.274\lambda\frac{F_y}{E}\right)M_y \le M_p$$

$$= 1.84\left(1.52 - 0.274(360)\frac{36}{29000}\right)243$$

$$= 625 \text{ kip-in.} > M_p = 360 \text{ kip-in.}$$

$$\phi V_n = \phi M_n / a = 0.9(360)/10$$

$$= 32.4 \text{kips} > 21 \text{kips}$$ OK

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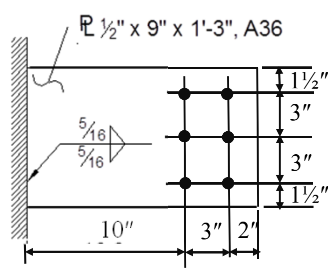
Extended Single-Plate Connection Example

7. Von Mises Yield Criterion

Maximum $M_u = V_u a \quad a = 10$ in.



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

$$\left(\frac{21.0}{1.0(0.6x36)(0.5x9.0)}\right)^2 + \left(\frac{21.0 \text{ x } 10.0}{0.9(36)(0.5x9.0^2)/4}\right)^2 \le 1.0$$

$$0.05 + 0.41 = 0.46 \le 1.0$$ OK

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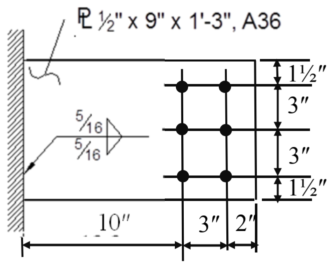
Extended Single-Plate Connection Example

8. Weld Strength



$$t_w \geq \tfrac{5}{8}\, t_p$$

$$\tfrac{5}{8}\, t_p = \tfrac{5}{8} \cdot \tfrac{1}{2}$$

$$= \tfrac{5}{16} \text{ in.}$$

$$t_{min} = \tfrac{3}{16} \text{ in.}$$

<u>5/16 in. E70xx Welds are Adequate</u>

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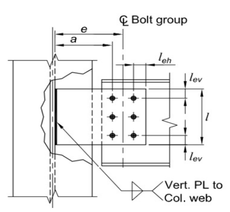
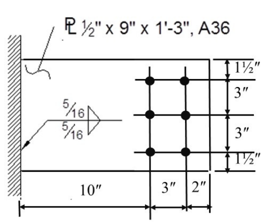
Extended Single-Plate Connection Example





Beam: W14x43 A992
$t_w = 0.305$ in.

Column: W14x90 A992
$t_w = 0.440$ in.

Plate: A36

7/8 in. A325-N Bolts
E70xx Electrode

<u>Connection is Adequate for V_u 21.0 kips</u>

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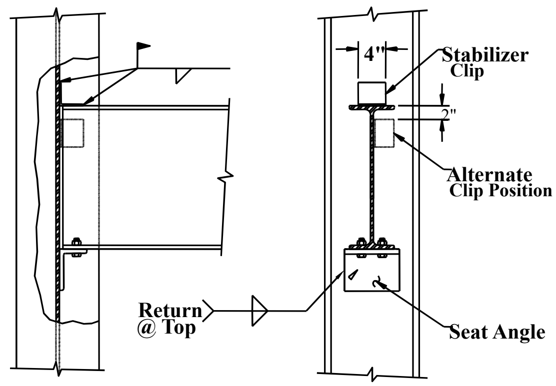
WELDED UNSTIFFENED SEATED CONNECTIONS



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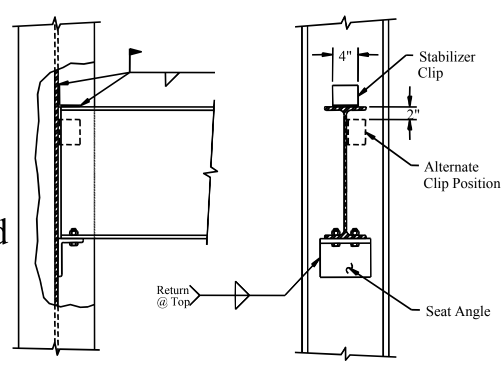
Welded Unstiffened Seated Connections

Advantages:

- Few Parts
- Few Bolts

Disadvantages:

- Requires Field Welded Stability Angle
- Limited strength
- Cannot Resist Axial Force



Comment:

Commonly used to connect to the web of a column.

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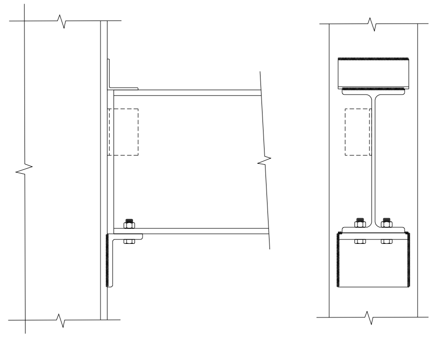
Welded Unstiffened Seated Connections

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



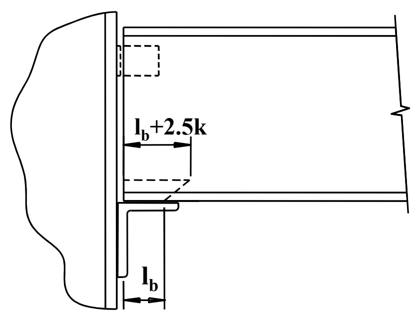
Welded Unstiffened Seated Connections

Beam Local Web Yielding

Specification Section J10.2

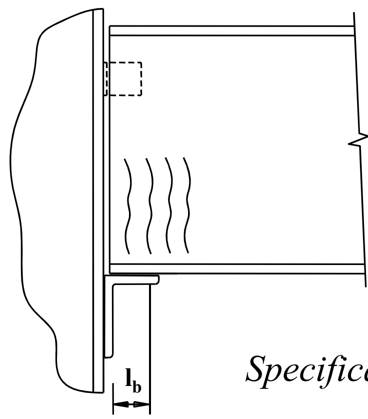
$\phi = 1.0$

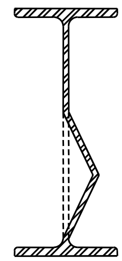
$R_n = (2.5k_{design} + l_b)F_{yw}\, t_w$ (*Spec.* Eqn. J10-3)



Welded Unstiffened Seated Connections

Beam Local Web Crippling
Specification Section J10.3





$\phi = 0.75$
Specification Eqns. J10-5a and -5b

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Welded Unstiffened Seated Connections

Beam Local Web Crippling

$l_b/d \le 0.2$ (*Spec.* Eqn. J10-5a)

$$R_n = 0.40\, t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E\, F_y\, t_f}{t_w}}\, Q$$

$l_b/d > 0.2$ (*Spec.* Eqn. J10-5b)

$$R_n = 0.40\, t_w^2 \left[1 + \left(\frac{4\, l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E\, F_y\, t_f}{t_w}}\, Q$$

Note: Q = 1.0 for wide flange sections.

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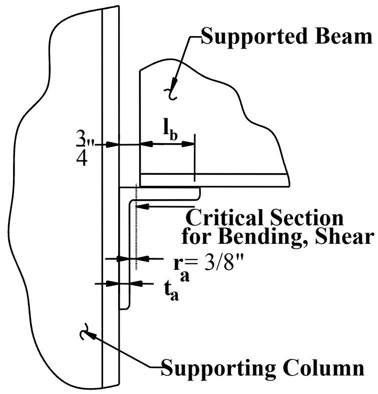
Welded Unstiffened Seated Connections

Design Model for Angle Flexure

The l_b-distance is determined from the limit states of web yielding and web crippling, but not less than k_{design} and l_b + setback not greater than OSL length.



Welded Unstiffened Seated Connections

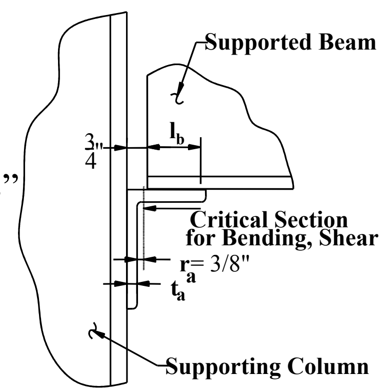
Design Model for Angle Flexure

Setback = 1/2 in.

Beam Tolerance = 1/4 in.

Use 3/4 in. setback
in calculations.

Assume distance from face of
angle leg to end of fillet = 3/8"



Welded Unstiffened Seated Connections

Design Model for Angle Flexure

The maximum value of l_b is then used to determine the eccentricity:

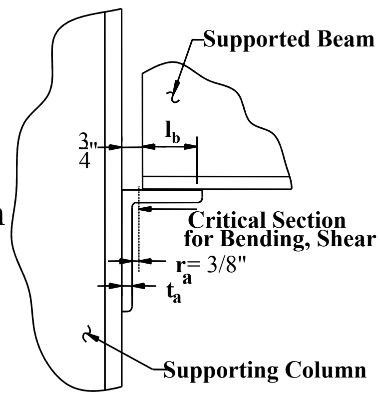
$l_b = \max(l_{b,yielding}, l_{b,crippling})$

but $\geq k_{design}$

and $l_b + ¾$ in. $\leq$ OSL length

$e = l_b/2 + (3/4 – 3/8) – t_a$

$= l_b/2 + 3/8 - t_a$



Welded Unstiffened Seated Connections

Design Model for Angle Flexure

For the Limit State of Web Yielding

$$l_{b,yielding} = \frac{R_u}{1.0\, F_y\, t_w} - 2.5\, k_{design}$$

Welded Unstiffened Seated Connections

Design Model for Angle Flexure

For the Limit State of Web Crippling

when $l_b/d \le 0.2$

$$l_{b,crippling} = \frac{d}{3}\left[\frac{R_u}{0.75\,(0.40\,t_w^2)}\sqrt{\frac{t_w}{E\,F_y t_f}} - 1\right]\left(\frac{t_f}{t_w}\right)^{1.5} Q$$

when $l_b/d > 0.2$

$$l_{b,crippling} = \frac{d}{4}\left\{\left[\frac{R_u}{0.75\,(0.40\,t_w^2)}\sqrt{\frac{t_w}{E\,F_y t_f}} - 1\right]\left(\frac{t_f}{t_w}\right)^{1.5} + 0.2\right\} Q$$

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Welded Unstiffened Seated Connections

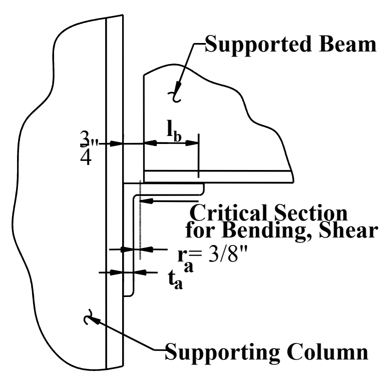
Design Model for Angle Flexure

Required angle thickness
from OSL bending

$$t_{req} = \sqrt{\frac{4\,R_u\,e}{0.9\,F_y\,L_a}}$$

with L_a = angle length

$e = l_b/2 + 3/4 - 3/8 - t_a$



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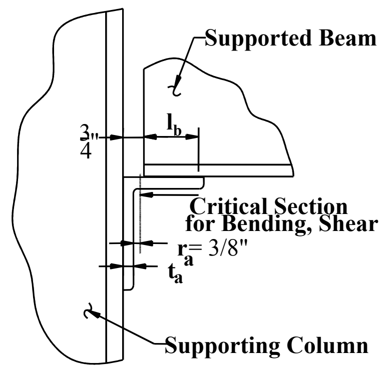
Welded Unstiffened Seated Connections

Angle Shear Yielding

$$R_u \le \phi R_n = 1.0(0.6F_y)L_a t_{angle}$$

$$t_{req} = \frac{R_u}{1.0(0.6F_y)L_a}$$



Welded Unstiffened Seated Connections

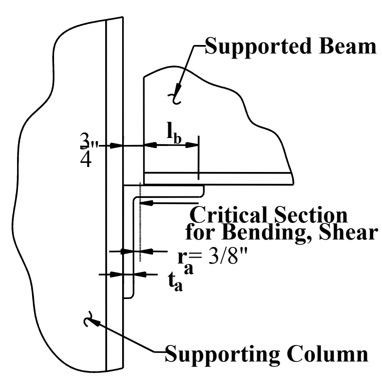
Weld Rupture Strength

Eccentric Weld Rupture Strength:
Elastic Method *Manual* p.10-72

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

L = length of vertical angle leg
= length of vertical welds

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



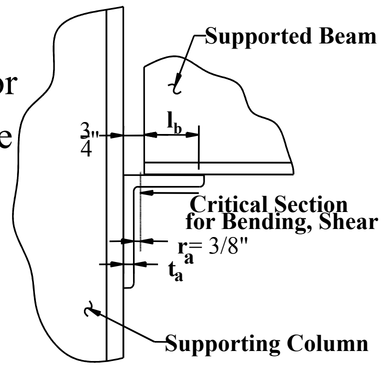
Welded Unstiffened Seated Connections

Design Model for Angle Flexure

Eccentric Weld Rupture Strength:
Instantaneous Center of Rotation Method

$\phi R_n = 0.75CC_1DL$

C is from *Manual* Table 8-4 for vertical welds or *Manual* Table 8-6 if welded all around.



Welded Unstiffened Seated Connections

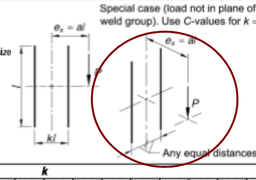
Table 8-4
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

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)

Special case (load not in plane of weld group). Use *C*-values for $k = 0$.



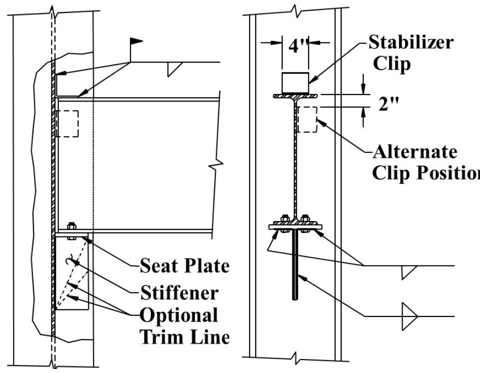
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71
0.10	3.72	3.72	3.72	3.71	3.70	3.69	3.67	3.65	3.63	3.61	3.59	3.55	3.52	3.48	3.44	3.43
0.15	3.67	3.66	3.65	3.64	3.62	3.60	3.58	3.56	3.54	3.52	3.50	3.46	3.43	3.39	3.36	3.33
0.20	3.51	3.51	3.50	3.49	3.47	3.46	3.44	3.42	3.41	3.39	3.38	3.35	3.32	3.30	3.27	3.25
0.25	3.31	3.31	3.31	3.30	3.29	3.28	3.28	3.27	3.26	3.25	3.25	3.23	3.21	3.20	3.18	3.16
0.30	3.09	3.09	3.10	3.10	3.10	3.10	3.11	3.11	3.11	3.11	3.11	3.11	3.10	3.09	3.08	3.07
0.40	2.66	2.67	2.68	2.70	2.73	2.75	2.77	2.80	2.81	2.83	2.84	2.87	2.88	2.89	2.90	2.90
0.50	2.30	2.30	2.32	2.36	2.40	2.44	2.48	2.52	2.55	2.58	2.60	2.65	2.68	2.70	2.72	2.73

Manual Table 8-4

$R_n = CC_1Dl$
$\phi = 0.75$

Parameters:
$k = 0$ & $a \Rightarrow C$

WELDED STIFFENED SEATED CONNECTIONS



Welded Stiffened Seated Connections

Advantages:

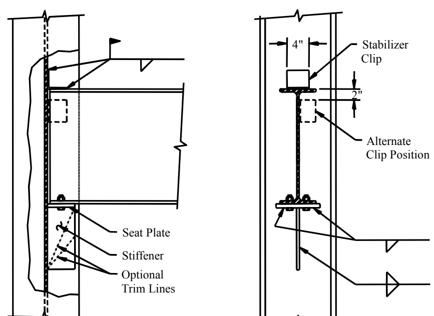
- Few Parts
- Few Bolts

Disadvantages:

- Requires Stability Angle
- Introduces Column Web Limit States

Comment:

- Commonly used to connect to the web of a column.



Welded Stiffened Seated Connections

Limit States

- Beam Web Yielding
- Beam Web Crippling
- Strength of Stiffener Plate
- Eccentric Shear of Welds
- Column Web Punching Shear Failure

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Welded Stiffened Seated Connections

Notes

- 1/2 in. setback but 3/4 in. for calculations
- Seat plate ≥ 3/8 in.
- For unstiffened beam webs, the seat stiffener thickness is a function of the beam and seat stiffener yield stresses and the weld size.

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Welded Stiffened Seated Connections

Seat Stiffener Thickness

For Unstiffened Beams (No web stiffeners):

	Seat Stiffener	
Beam, F_y	F_y	$t_{s,min}$
36	36	t_w
50	36	$1.4\ t_w$
50	50	t_w

Stiffened and Unstiffened Beams:

$$t_{s,min}\ (36\ \text{ksi}) \geq 2\ t_{weld}$$

$$t_{s,min}\ (50\ \text{ksi}) \geq 1.5\ t_{weld}$$

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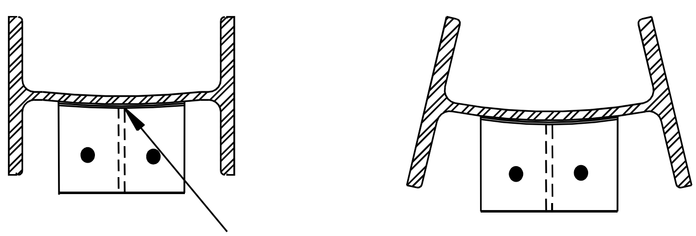
Welded Stiffened Seated Connections

Column Web Limit State



Stress Concentration which may cause weld fracture.

Need to prevent flange rotation.

Solutions:

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

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Welded Stiffened Seated Connections

Column Web Limit State

Manual p.10-78 has a simplified approach for sections heavier than:

38 lb/ft for W14	($\geq$ W14x43)
35 lb/ft for W12	($\geq$ W12x40)
30 lb/ft for W10	($\geq$ W10x33)
21 lb/ft for W8	($\geq$ W8x24)

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Welded Stiffened Seated Connections

Column Web Limit State

Simplified rules:

- Beam must be bolted to seat with A325 or A490 bolts within the greater of W/2 or 2-5/8 in. from the column web.
- Special rules for W14x43.
- Beam flange is not welded to the seat plate.

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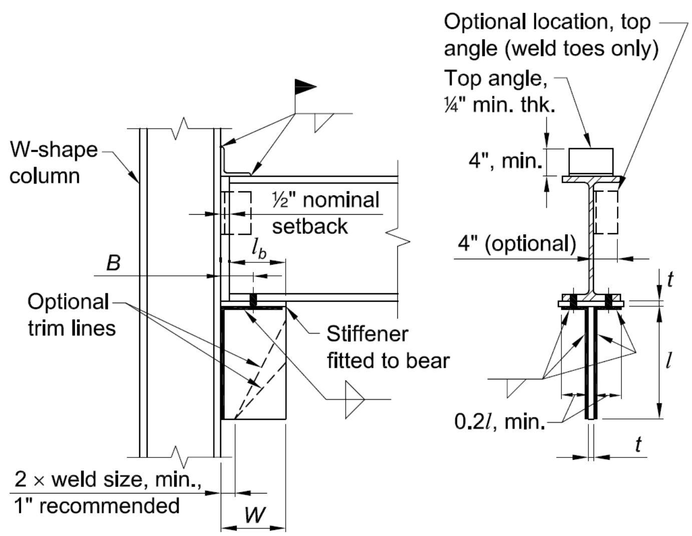
Welded Stiffened Seated Connections



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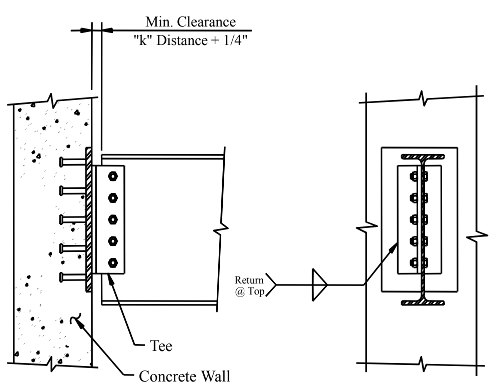
WELDED TEE AND WT CONNECTIONS



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Tee Connections

Advantages:

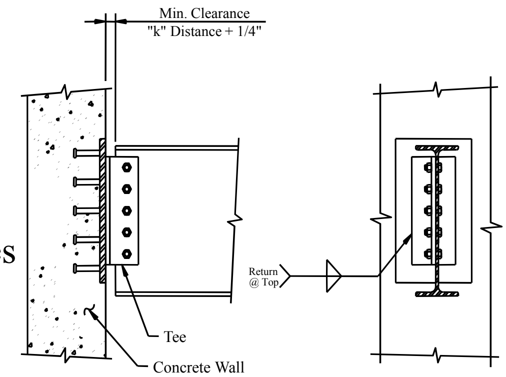
- One Sided

Disadvantages:

- Tee can be Heavy
- Stiffer than Other Types except Shear Tab

Comment:

- Sometimes used to connect to concrete wall or existing construction



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THE END
Thank You for Attending

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Next Session

November 7, 2017 Moment Connections Part I

TOPICS

- Flange Welded /Web Bolted
- Flange Plate Welded /Web Bolted
- Flange Plate Bolted/Web Bolted
- Column Side Limit States
- Design Example

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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!

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.

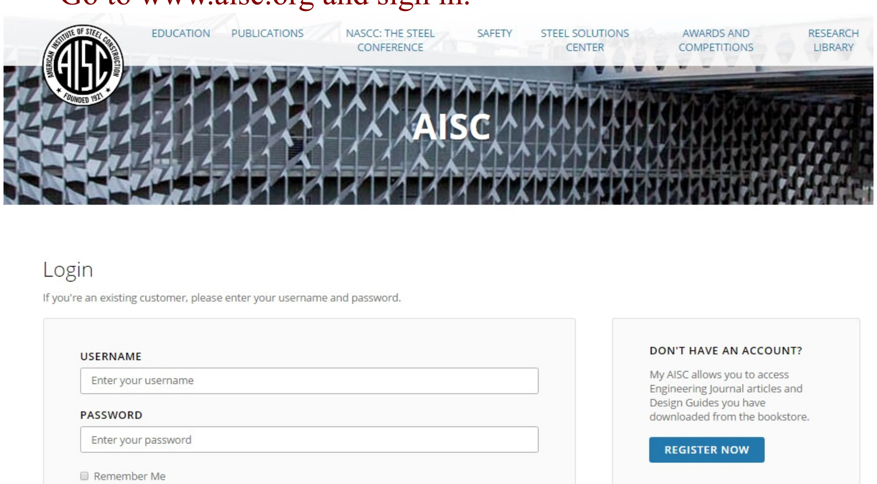
CEUs/PDHS – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHs.

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!

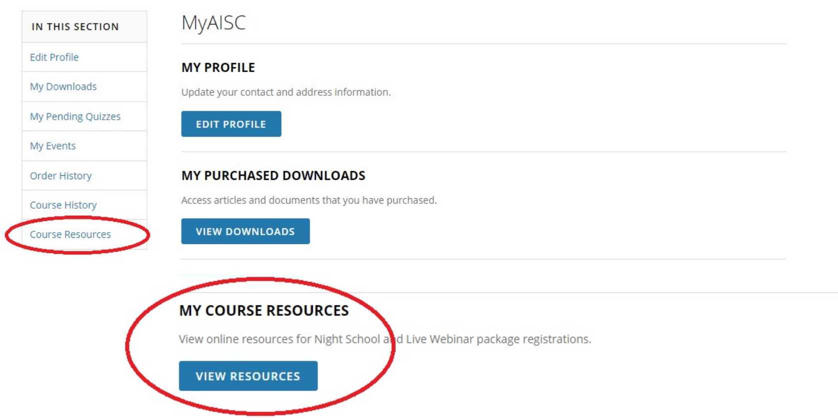
Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.

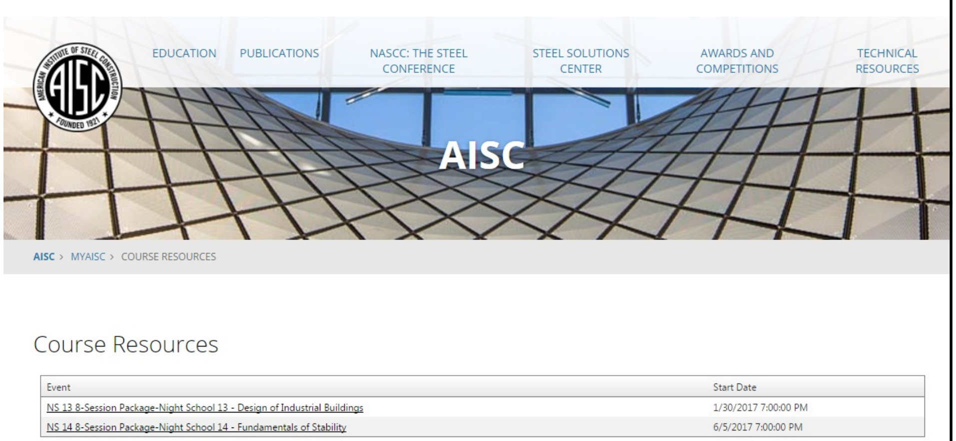


Night School Resources for 8-session package Registrants

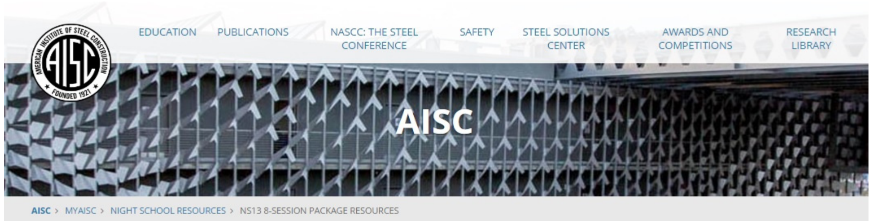
Go to www.aisc.org and sign in.



Night School Resources for 8-session package Registrants



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.

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