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Course Description

Session 2: October 10, 2017 -Fundamental Concepts, Part II

This live webinar discusses eccentric bolted and welded connections, direct loaded tension connections, block shear, the Whitmore Section, and light bracing connections. Beam bearing and column base plate design are also discussed. Design examples are presented to demonstrate concepts.

Learning Objectives

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

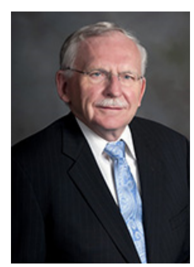
- List the steps in designing an eccentric bolted and welded connection.
- List the limit states in designing a light bracing connection.
- Describe the Whitmore Section concept.
- List the limit states in designing a beam bearing plate connection.

There's always a solution in steel.

Fundamentals of Connection Design

Session 2: Fundamental Concepts Part 2

October 10, 2017



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

SCHEDULE

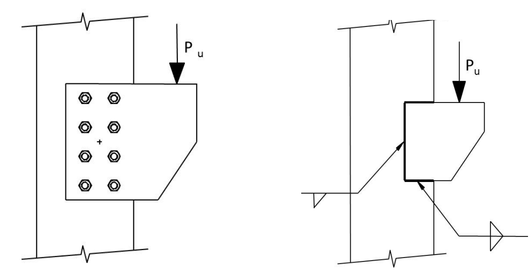
- October 03, 2017 Fundamental Concepts Part I
- October 10, 2017 Fundamental Concepts Part II
- October 17, 2017 Shear Connections Part I
- October 24, 2017 Shear Connections Part II
- November 07, 2017 Moment Connections Part I
- November 14, 2017 Moment Connections Part II
- November 28, 2017 Introduction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

FUNDAMENTAL CONCEPTS PART II

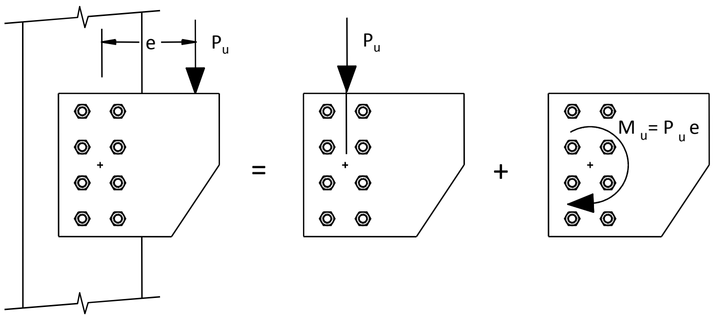
TOPICS

- Eccentric Bolted and Welded Connections
- Direct Loaded Tension Connections
- Light Bracing Connection Example
- Beam Bearing Plate Design
- Column Base Plate Design

ECCENTRIC BOLTED AND WELDED CONNECTIONS

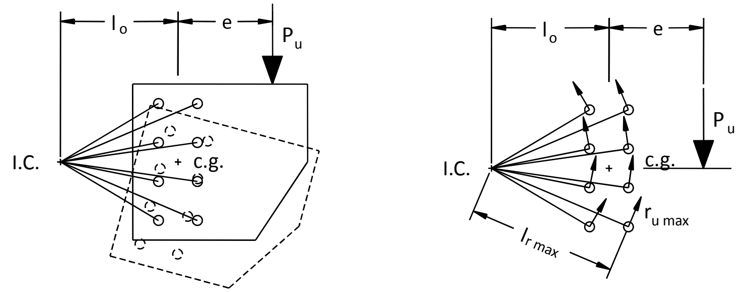


Bolts: Eccentric Connections



Elastic Method

Bolts: Eccentric Connections



Instantaneous Center of Rotation Method

Bolts: Eccentric Connections

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

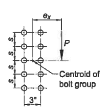
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

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

$\phi P_n = C \times \phi r_v$
$\phi = 0.75$

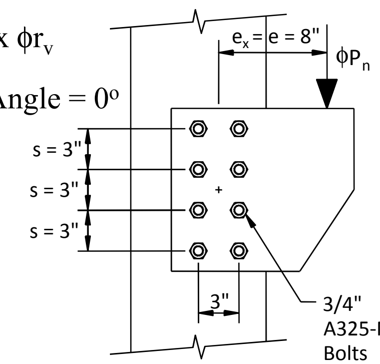
Example: Eccentric Bolted Connection

Determine ϕP_n

$\phi P_n = C$ (from Table 7-7) x ϕr_v

From *Manual* Table 7-7, Angle = 0°
with e = 8 in. and n=4
C = 2.93

From *Manual* Table 7-1,
for ¾ in. A325-N:
$\phi r_v = \phi F_{nv} A_b = 17.9$ k/bolt



$\phi P_n = 2.93 \times 17.9 = 52.4$ k

Example: Eccentric Bolted Connection

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

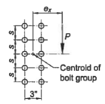
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

or

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

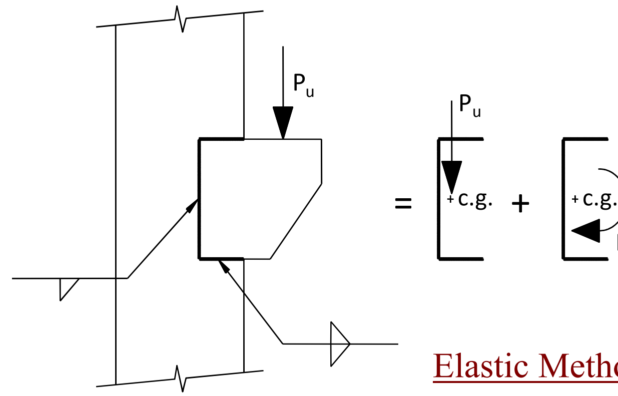
where
- P = required force, P_u or P_a, kips
- r_n = nominal strength per bolt, kips
- e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
- s = bolt spacing, in.
- C = coefficient tabulated below



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

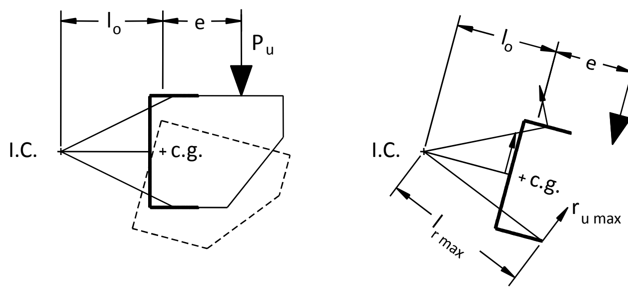
n = 4
e_x = 8 in.
C = 2.93

Welds: Eccentric Connections



Elastic Method

Welds: Eccentric Connections



Instantaneous Center of Rotation Method

Welds: Eccentric Connections

Eccentric Weld Strength

$$R_{nx} = \Sigma F_{nwix} A_{wei}$$

$$R_{ny} = \Sigma F_{nwiy} A_{wei}$$

$$M_n = \Sigma \left[F_{nwiy} A_{wei} (x_i) - F_{nwix} A_{wei} (y_i) \right]$$

where
- A_{wei} = effective area of weld throat of the ith weld element, in.^{2} (mm^2)
- F_{nwi} = $0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta_i) f(p_i)$
- $f(p_i)$ = $[p_i (1.9 - 0.9 p_i)]^{0.3}$
- F_{nwi} = nominal stress in the ith weld element, ksi (MPa)
- F_{nwix} = x-component of nominal stress, F_{nwi}, ksi (MPa)
- F_{nwiy} = y-component of nominal stress, F_{nwi}, ksi (MPa)
- p_i = Δ_i / Δ_{mi}, ratio of element i deformation to its deformation at maximum stress
- r_{cr} = distance from instantaneous center of rotation to weld element with minimum Δ_{ui}/r_i ratio, in. (mm)
- r_i = distance from instantaneous center of rotation to ith weld element, in. (mm)

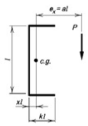
Welds: Eccentric Connections

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

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

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

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

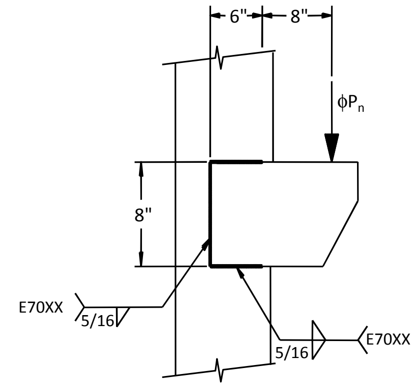


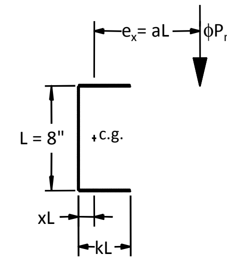
Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Sections J2.4,

a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	8.26	9.37	10.5	11.6	12.7
0.10	1.86	2.28	2.78	3.30	3.83	4.37	4.92	5.46	6.01	6.56	7.11	8.22	9.32	10.4	11.5	12.7
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.41	6.94	8.02	9.11	10.2	11.3	12.4
0.20	1.76	2.18	2.63	3.11	3.60	4.11	4.61	5.13	5.64	6.16	6.68	7.72	8.77	9.83	10.9	12.0
0.25	1.66	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37	5.86	6.36	7.37	8.39	9.42	10.5	11.5
2.6	0.253	0.320	0.396	0.481	0.576	0.680	0.788	0.901	1.02	1.15	1.28	1.57	1.90	2.25	2.64	3.05
2.8	0.235	0.297	0.368	0.447	0.535	0.632	0.734	0.839	0.950	1.07	1.19	1.47	1.77	2.10	2.46	2.85
3.0	0.219	0.278	0.343	0.417	0.500	0.591	0.686	0.784	0.889	1.00	1.12	1.37	1.66	1.97	2.31	2.68
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

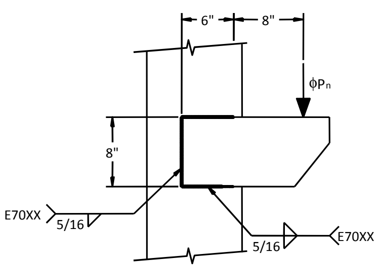
$R_n = CC_1DL$
$\phi = 0.75$
Parameters:
$C_1 = Exx/70$
$k \Rightarrow$ x (at bottom of Table)
x & a $\Rightarrow$ C

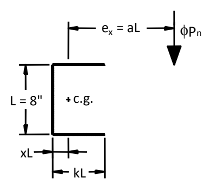
Example: Determine ϕP_n





Example: Determine ϕP_n





kL = 6 in. $\Rightarrow$ k = 6 /8 = 0.75
Using *Manual* Table 8-8: x = 0.225
xL = 0.225 x 8 = 1.8 in. (location of c.g.)

Example: Determine ϕP_n

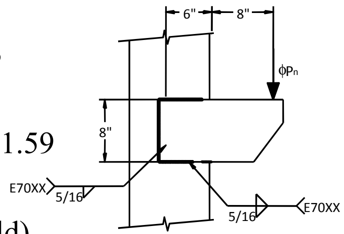
Manual Table 8-8

a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	8.26	9.37	10.5	11.6	12.7
0.10	1.86	2.28	2.78	3.30	3.83	4.37	4.92	5.46	6.01	6.56	7.11	8.22	9.32	10.4	11.5	12.7
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.41	6.94	8.02	9.11	10.2	11.3	12.4
0.20	1.76	2.18	2.63	3.11	3.60	4.11	4.61	5.13	5.64	6.16	6.68	7.72	8.77	9.83	10.9	12.0
0.25	1.66	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37	5.86	6.36	7.37	8.39	9.42	10.5	11.5
2.6	0.253	0.320	0.396	0.481	0.576	0.680	0.788	0.901	1.02	1.15	1.28	1.57	1.90	2.25	2.64	3.05
2.8	0.235	0.297	0.368	0.447	0.535	0.632	0.734	0.839	0.950	1.07	1.19	1.47	1.77	2.10	2.46	2.85
3.0	0.219	0.278	0.343	0.417	0.500	0.591	0.686	0.784	0.889	1.00	1.12	1.37	1.66	1.97	2.31	2.68
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Example: Determine ϕP_n

With $e_x = aL$

$a = (6.0+8.0-1.8)/8.0 = 1.53$



Using *Manual* Table 8-8: C = 1.59

D = 5 (since 5/16" weld)

$C_1 = 1.0$ (since E70XX weld)

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

$= 0.75 \times 1.59 \times 1.0 \times 5 \times 8.0$

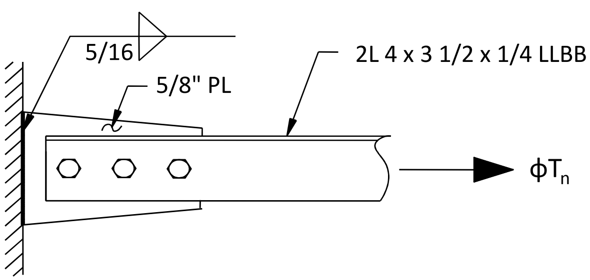
$= \underline{47.7\ k}$

Example: Determine ϕP_n

Manual Table 8-8

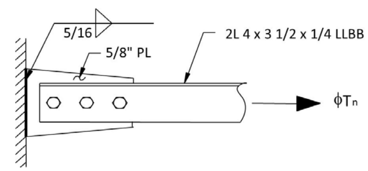
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
1.4	0.464	0.589	0.729	0.883	1.05	1.23	1.42	1.61	1.82	2.04	2.27	2.77	3.31	3.89	4.50	5.15
1.6	0.408	0.517	0.640	0.775	0.924	1.09	1.25	1.43	1.61	1.81	2.02	2.46	2.95	3.48	4.04	4.64
1.8	0.363	0.461	0.570	0.691	0.825	0.970	1.12	1.28	1.45	1.62	1.81	2.22	2.66	3.14	3.66	4.21
2.0	0.328	0.415	0.514	0.623	0.744	0.877	1.01	1.16	1.31	1.47	1.64	2.01	2.42	2.86	3.34	3.85
2.2	0.298	0.378	0.468	0.567	0.678	0.800	0.926	1.06	1.20	1.35	1.50	1.84	2.22	2.62	3.07	3.54
2.4	0.274	0.347	0.429	0.521	0.623	0.735	0.852	0.973	1.10	1.24	1.38	1.70	2.04	2.42	2.84	3.28
2.6	0.253	0.320	0.396	0.481	0.576	0.680	0.788	0.901	1.02	1.15	1.28	1.57	1.90	2.25	2.64	3.05
2.8	0.235	0.297	0.368	0.447	0.535	0.632	0.734	0.839	0.950	1.07	1.19	1.47	1.77	2.10	2.46	2.85
3.0	0.219	0.278	0.343	0.417	0.500	0.591	0.686	0.784	0.889	1.00	1.12	1.37	1.66	1.97	2.31	2.68
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

DIRECT LOADED TENSION CONNECTIONS



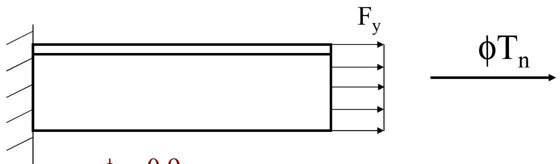
Applicable Limit States

- Tension Yielding
- Tension Rupture
- Block Shear
- Bearing and Tear Out
- Bolt Rupture
- Weld Rupture
- Whitmore Section Considerations
- For Design: $T_u \leq \phi T_n$



Limit State: Tension Rupture

Specification D2
Tension Yielding

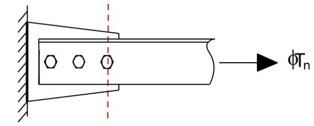


$\phi = 0.9$
$T_n = F_y A_g$ (*Spec.* D2-1)

Note: Tension yielding is actually a member limit state.

Limit State: Tension Rupture

Specification D2
Tension Rupture



$\phi = 0.75$
$\phi T_n = 0.75\, F_u A_e$ (*Spec.* D2-2)

F_u = tensile strength
= 58 ksi for A36; 65 ksi for A992 & Gr50

A_e = effective net area = U A_n

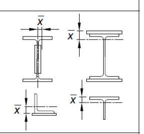
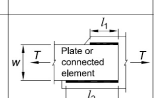
U = reduction or shear lag coefficient

A_n = net area

Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

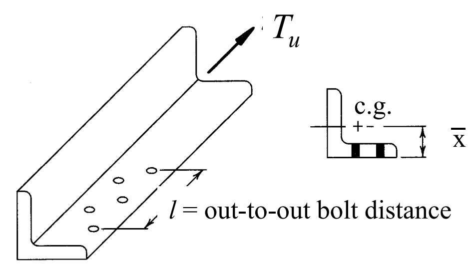
Case	Description of Element	Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).	$U = 1.0$	–
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)	$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and A_n = area of the directly connected elements	–
4[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.	$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$	

Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

Ex. *Specification* Table D3.1, Case 2

$U = 1 - \bar{x}/l$

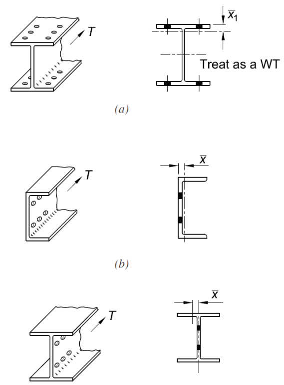


Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

$U = 1 - \bar{x}/l$

l = out-out bolt distance

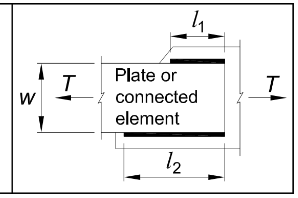


Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

Ex. *Specification* Table D3.1 Case 4

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$$



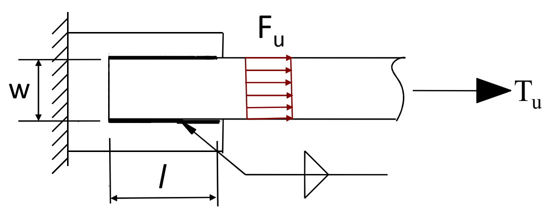
where $l = \dfrac{l_1 + l_2}{2}$

Note: l_1 and l_2 not less than 4 times fillet weld size.

Limit State: Tension Rupture

Reduction or Shear Lag Coefficient

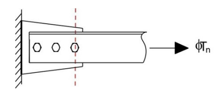
Ex. Common Connection



$$U = \frac{3l^2}{3l^2 + w^2}$$

Limit State: Tension Rupture

Net Area



A_n = Net Area = $A_g - \Sigma A_{h'} + \Sigma$ Stagger

A_g = gross area of cross-section

$A_{h'}$ = effective area of hole

= (hole diameter + 1/16 in.) t_p

Stagger = $(s^2/4g)t_p$ (*Specification* B4.3b)

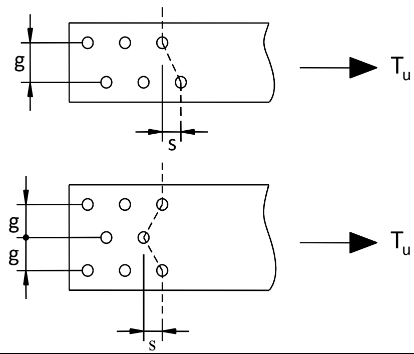
~~Note: $A_n \leq 0.85 A_g$ for Tension Splice Plates~~

~~(Rule does not apply to members.)~~

Limit State: Tension Rupture

Net Area

Stagger Term = $(s^2/4g)t_p$



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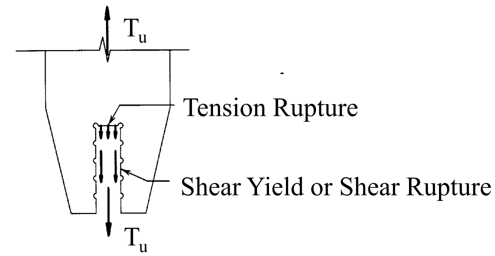
Limit State: Block Shear Strength

Specification J4.3
Block Shear Strength



- Failure occurs when shear forces reach the smaller of shear yield and shear rupture.
- The tension area is at rupture when failure occurs.

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Limit State: Block Shear

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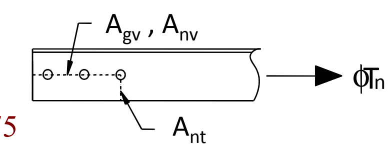
Limit State: Block Shear Strength

Specification J4.3
Block Shear Strength



$\phi = 0.75$

$$R_n = [0.6F_uA_{nv}+U_{bs}F_uA_{nt}] \leq [0.6F_yA_{gv} +U_{bs}\,F_uA_{nt}] \quad \text{(J4-5)}$$

Or,

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

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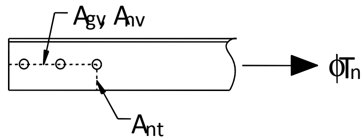
Limit State: Block Shear Strength

Specification J4.3
Block Shear Strength



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

- Shear Rupture $= 0.6F_uA_{nv}$ (net shear area)
- Shear Yield $= 0.6F_yA_{gv}$ (gross shear area)
- Tension Rupture $= F_uA_{nt}$ (net tension area)
- $U_{bs} = 1.0$ for Direct Loaded Connections

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Limit State: Block Shear Strength

Example: $A_{nv} = 2.53\ \text{in}^2$ $A_{gv} = 3.625\ \text{in}^2$
$A_{nt} = 0.781\ \text{in}^2$ $U_{bs} = 1.0$
A36 Steel: $F_y = 36$ ksi $F_u = 58$ ksi

$$R_n = \min\left|\begin{array}{l}0.6\text{x}58\text{x}2.53 = 88.0\\ 0.6\text{x}36\text{x}3.625 = \underline{78.3}\end{array}\right. + 1.0\text{x}58.0\text{x}0.781$$

$$= 78.3 + 45.3 = 123.6\ \text{k}$$

$$\phi R_n = 0.75\text{x}123.6 = \underline{92.7\ \text{k}}$$

(Note: See following example for area calculations)

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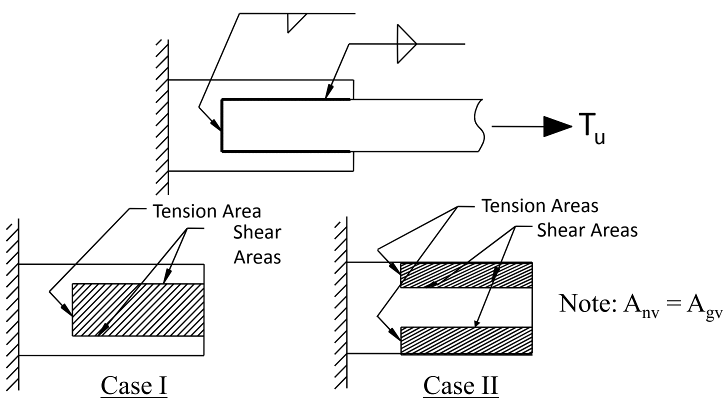
Limit State: Block Shear Strength

Welded Connections



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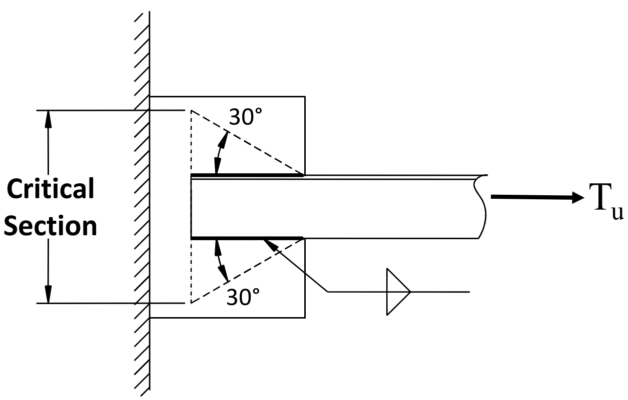
Whitmore Section -- Welds



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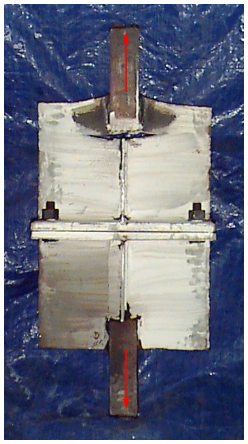
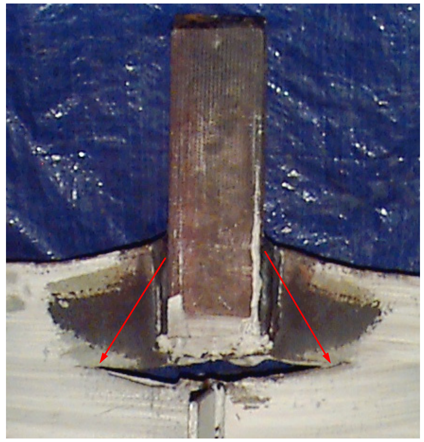
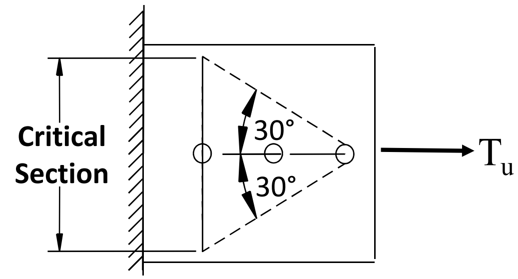
Whitmore Section -- Welds





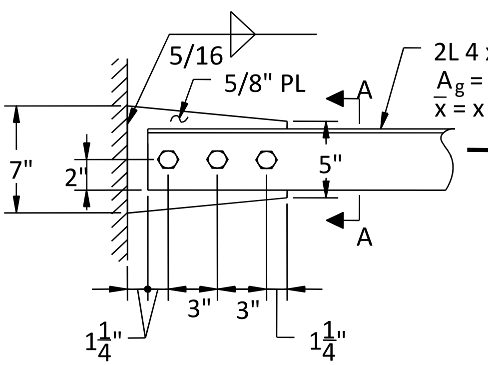
Whitmore Section -- Bolts

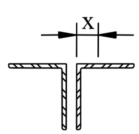


Light Bracing Connection

Example. Determine ϕT_n.

A36 Steel 3/4 in. A325-N Bolts



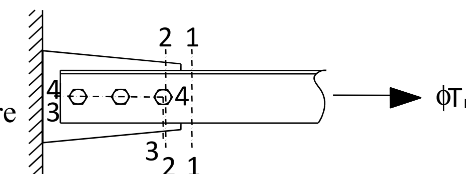

Section A-A

Light Bracing Connection

Limit States:

Angles:
- 1-1 Tension Yield
- 2-2 Tension Rupture
- 3-3 Block Shear

Shear Transfer Between Elements:
- 4-4 Angle Bearing and Tear Out
- 4-4 Bolt Shear Rupture
- 4-4 Plate Bearing and Tear Out



Light Bracing Connection

Limit States:

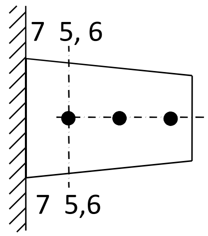
Plate:

5-5 Tension Yield

6-6 Tension Rupture

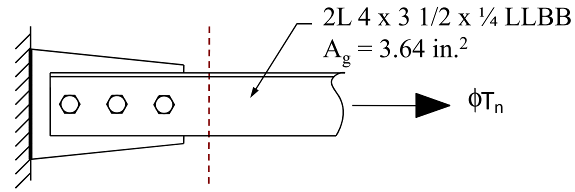
Weld:

7-7 Weld Rupture



Light Bracing Connection

Angle Yielding



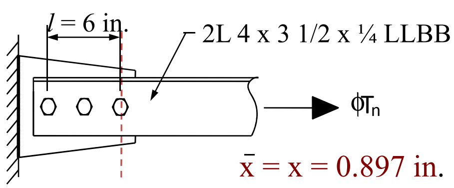
$\phi T_n = 0.9\, F_y\, A_g$

$= 0.9 \times 36 \times 3.64 = \underline{118\text{ k}}$

Light Bracing Connection

Angle Rupture

3/4 in. A325-N Bolts



$\phi T_n = 0.75\, F_u\, A_e = 0.75\, F_u U A_n$

$A_n = A_g - A_h' = 3.64 - (0.5)\,(3/4 + 1/16 + 1/16)$

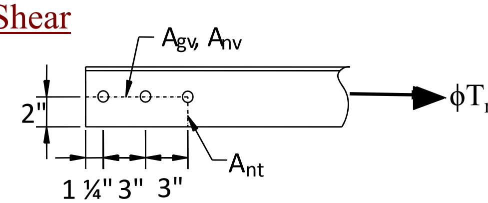
$= 3.18\text{ in}^2$

$U = 1 - \bar{x}/l = 1 - 0.897/6 = 0.850$

$\phi T_n = 0.75 \times 58 \times 0.850 \times 3.18 = \underline{118\text{ k}}$

Light Bracing Connection

Angle Block Shear



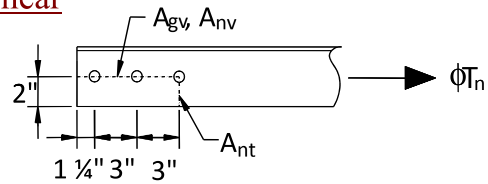
$A_{nv} = 0.5\,[7.25 - (2.5 \times 7/8)] = 2.53\text{ in}^2$

$A_{gv} = 0.5 \times 7.25 = 3.625\text{ in}^2$

$A_{nt} = 0.5\,[2.0 - (0.5 \times 7/8)] = 0.781\text{ in}^2$

Light Bracing Connection

Angle Block Shear



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

$$= \min \left| \begin{array}{l} 0.6 \text{x} 58 \text{x} 2.53 = 88.0 \\ 0.6 \text{x} 36 \text{x} 3.625 = \underline{78.3} \end{array} \right. + 1.0 \text{x} 58 \text{x} 0.781$$

$$\phi R_n = 0.75\ (78.3 + 45.3) = \underline{92.7\ \text{k}}$$

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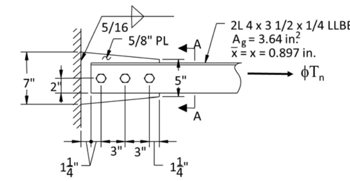
Light Bracing Connection

Shear Transfer Between Elements

Specification Section J3.6 User Note



Strength at Each Bolt/Hole is the minimum of:

- Angle Bearing / Tear Out
- Bolt Shear Rupture
- Plate Bearing / Tear Out

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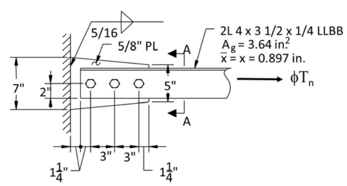
Light Bracing Connection

Shear Transfer Between Elements

Angle Bearing / Tear Out:



Bearing: $2.4F_u td = (2.4 \text{ x } 58)\ (0.5 \text{ x } 3/4) = 52.2\text{k}$

Edge: $1.2F_u l_c t = (1.2 \text{ x } 58)\ (1.25 - 13/32)\ (0.5)$
$= \underline{29.4\ \text{k}} < 52.2$ k (Tear-Out Controls)

Other: $1.2F_u l_c t = (1.2 \text{ x } 58)\ (3.0 - 13/16)\ (0.50)$
$= 76.1\ \text{k} > \underline{52.2\ \text{k}}$ (Bearing Controls)

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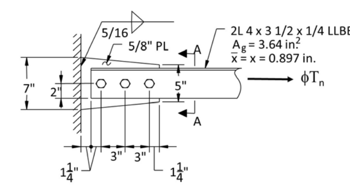
Light Bracing Connection

Shear Transfer Between Elements

Bolt Shear Rupture:



¾ in. A325-N Bolts in Double Shear
With F_{nv} =54.0 ksi from *Specification* Table J3.2

$r_v = F_{nv} A_b n_s = 54 \text{ x } 0.4418 \text{ x } 2$
$= 47.7$ k/bolt (nominal strength)

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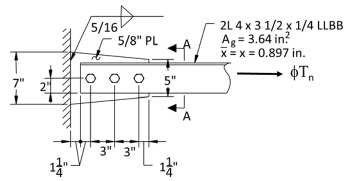
Light Bracing Connection

Shear Transfer Between Elements

Plate Bearing / Tear Out:



Bearing: $2.4F_u td = (2.4 \times 58)(0.625 \times 3/4) = 65.3$ k

Edge: $1.2F_u L_c t = (1.2 \times 58)(1.25 - 13/32)(0.625)$
$= 36.8$ k < 65.3 k (Tear-Out Controls)

Other: $1.2F_u L_c t = (1.2 \times 58)(3.0 - 13/16)(0.625)$
$= 95.1$ k > 65.3 k (Bearing Controls)

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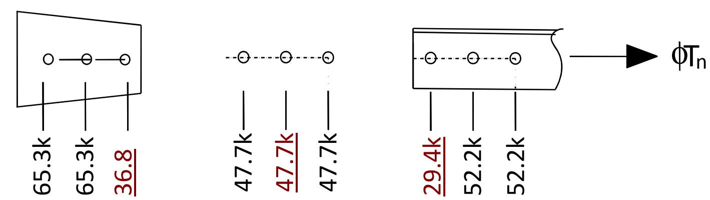
Light Bracing Connection

Shear Transfer Between Elements

Plate Brg./T.O. Bolt Rupture Angles Brg./T.O.



$\phi T_n = 0.75(36.8 + 47.7 + 29.4) = 85.4$ k

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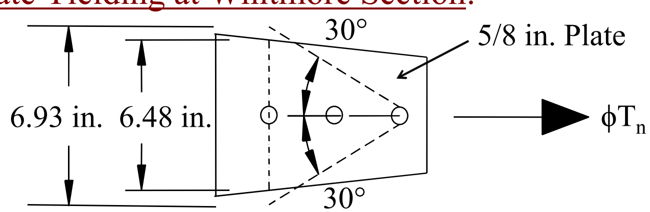
Light Bracing Connection

Plate Yielding at Whitmore Section:



$A_g = 0.625 \times 6.48 = 4.05 \text{ in}^2$

$\phi T_n = 0.9 F_y A_g$
$= (0.9 \times 36)(4.05) = 131$ k

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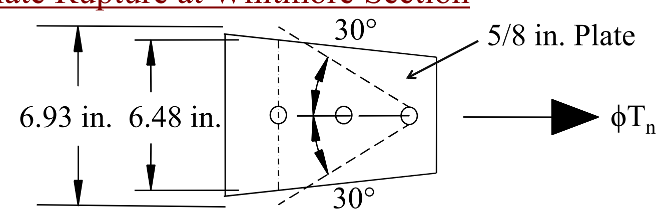
Light Bracing Connection

Plate Rupture at Whitmore Section



$A_e = UA_n$ U = 1.0 for plates

$A_n = (6.48 - 0.875)(0.625)$
$= 3.50 \text{ in}^2$

$\phi T_n = 0.75 F_u A_e$
$= (0.75 \times 58)(1.0 \times 3.50) = 152$ k

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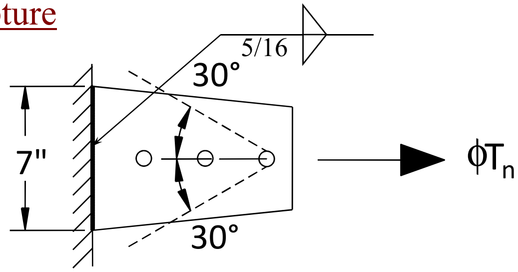
Light Bracing Connection

Weld Rupture



$T_n = 1.392\ (1.5)\ D\ L_{weld}$

$= 1.392\ (1.5)\ (2 \times 5)\ 7.0 =$ **146 k**

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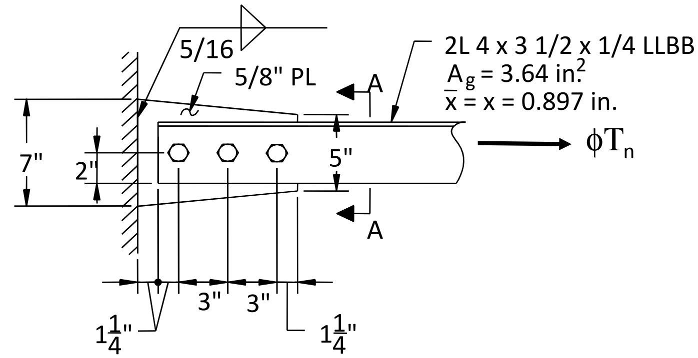
Light Bracing Connection

Connection Design Strength



$\phi T_n =$ 85.4 k (Shear Transfer Between Elements)

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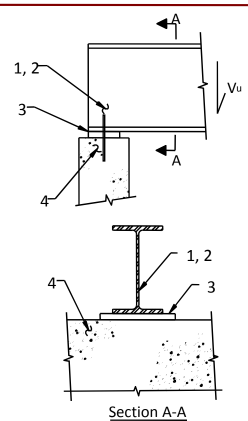
BEAM BEARING LIMIT STATES

1. Beam Web Local Yielding (*Specification* J10.2)
2. Beam Web Local Crippling (*Specification* J10.3)
3. Bearing Plate Bending
4. Bearing on Concrete (*Specification* J8**)**



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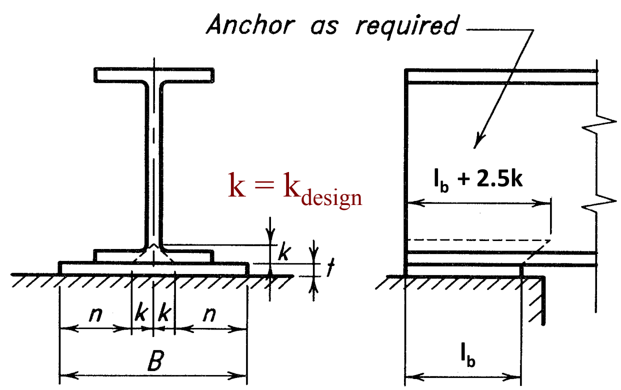
Beam Web Local Yielding



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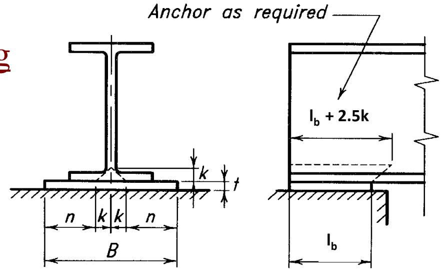
Beam Web Local Yielding

Specification J10.2
Web Local Yielding



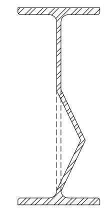
$\phi = 1.0$

$R_n = (2.5k_{design} + l_b)F_{yw}\, t_w$ at supports

$R_n = (5.0k_{design} + l_b)F_{yw}\, t_w$ interior loads

Beam Web Local Crippling

Specification J10.3
Web Local Crippling @ >d/2



$\phi = 0.75$

$$R_n = 0.8\, t_w^2 \left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}\, t_f}{t_w}}\,Q_f \quad \text{(Spec.J10-4)}$$

Note: $Q_f = 1.0$ for wide flange sections.

Beam Web Local Crippling

Specification J10.3
Web Local Crippling @ <d/2

$l_b/d \le 0.2$

$$R_n = 0.40\, t_w^2 \left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E\,F_{yw}\, t_f}{t_w}}\,Q_f \quad \text{(J10-5a)}$$

$l_b/d > 0.2$

$$R_n = 0.40\, t_w^2 \left[1+\left(\frac{4\,l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E\,F_{yw}\, t_f}{t_w}}\,Q_f \quad \text{(J10-5b)}$$

Bearing on Concrete

Specification J8
Column Bases and Bearing on Concrete

$\phi_c = 0.65$

(a) On the full area of a concrete support

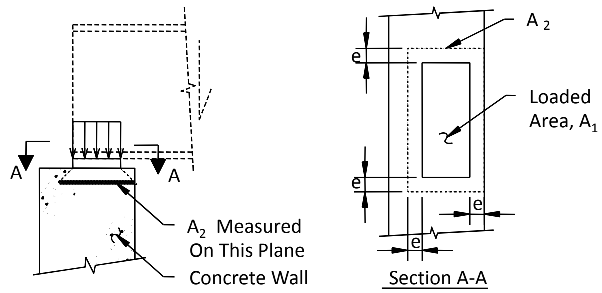
$P_p = 0.85 f_c' A_1$ (*Spec.* J8-1)

(b) On less than the full area of a concrete support

$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \le 1.7 f_c' A_1$ (*Spec.* J8-2)

Note: The limit $\le 1.7f'_c$ is equivalent to $A_2/A_1 \le 4$

Beam Bearing: Concrete Crushing

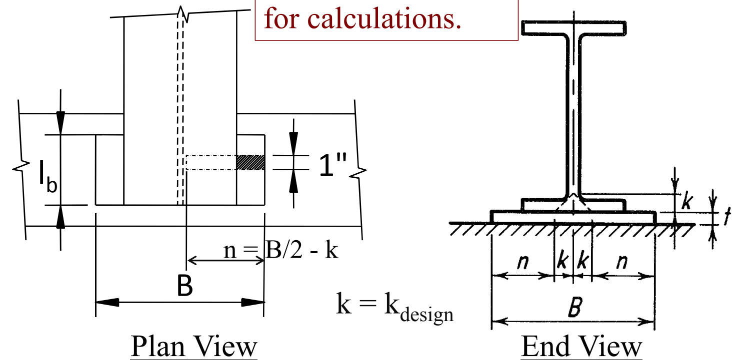


A_1 = area of steel bearing on concrete, in.^{2}

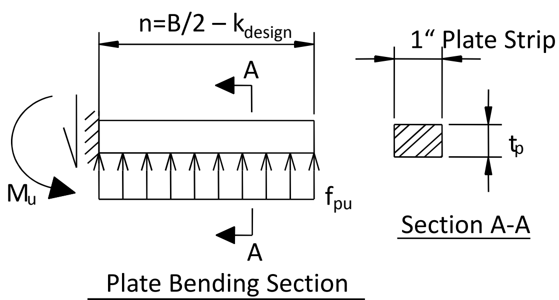
A_2 = area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2}

Beam Bearing: Plate Bending

Note: 1 in. wide plate strip is used for calculations.



Beam Bearing: Plate Bending

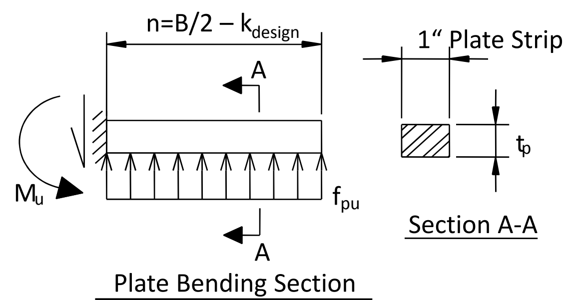


$f_{pu} = R_u / A_1$ = pressure due to reaction

$M_u = f_{pu}(1)\, n^2/2 \le \phi M_p$

$\phi M_p = 0.9\, F_y\, Z_x = 0.9 F_y (1 \times t_p^2/4)$

Beam Bearing Plate Example

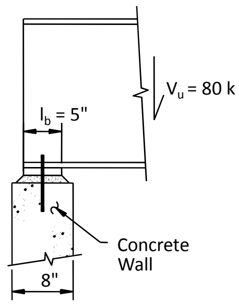


Substituting: $\boxed{t_{p,min} = \sqrt{\dfrac{2 f_{pu} n^2}{0.9 F_y}}}$

Beam Bearing Plate Example

Ex. Determine if l_b = 5 in. is adequate.
Determine required Plate Width, B.
Determine required Plate Thickness, t_p.

R_u = 80 k
Plate F_y = 36 ksi
Concrete f_c' = 3.0 ksi



Beam Bearing Plate Example

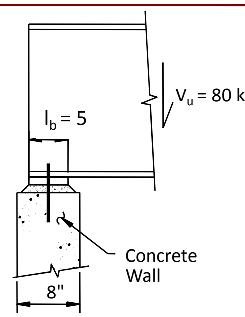
Properties:

W18x76 A992
F_y = 50 ksi
b_f = 11.0 in.
t_f = 0.680 in.
d = 18.2 in.
t_w = 0.425 in.
k = 1.08 in. (design)

Beam Bearing Plate Example

Web Local Yielding:

F_{yw} = 50 ksi
k_{design} = 1.08 in.
t_w = 0.425 in.



$$\phi R_n = 1.0\,(2.5k_{design} + l_b)F_{yw}\,t_w$$
$$= 1.0\,(2.5 \times 1.08 + 5.0)\,(50 \times 0.425)$$
$$= 164 \text{ k} > R_u = 80\text{k} \quad \underline{\text{OK}}$$

Beam Bearing Plate Example

Web Local Crippling: F_y = 50 ksi

At $\leq d/2$:

$l_b/d = 5/18.21 = 0.27 > 0.2 \quad Q_f = 1.0$

$$R_n = 0.40\,t_w^2\left[1 + \left(\frac{4\,l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E_s\,F_y\,t_f}{t_w}}\,Q_f \quad (\textit{Spec.}\ \text{J10-5b})$$

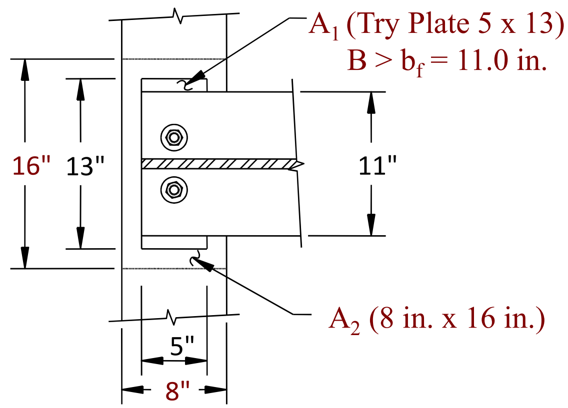
$$= 0.40\text{x}0.425^2\left[1 + \left(\frac{4 \times 5.0}{18.21} - 0.2\right)\left(\frac{0.425}{0.680}\right)^{1.5}\right]\sqrt{\frac{29000\text{x}50\text{x}0.680}{0.425}}$$

= 159 k

$\phi R_n = 0.75 \times 159 = 119 \text{ k} > R_u = 80\text{k}$ OK

l_b = 5 in. is Adequate

Beam Bearing Plate Example



Plan View

Beam Bearing Plate Example

Concrete Crushing:

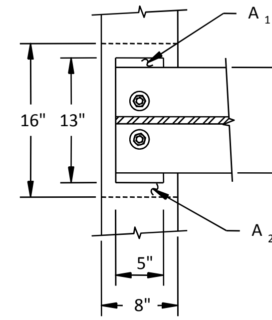
With: B = 13 in. > b_f = 11 in.

l_b = 5 in.

$A_1 = 5 \times 13 = 65$ in^2

$A_2 = 8 \times 16 = 128$ in^2

$A_2/A_1 = 1.96 < 4$



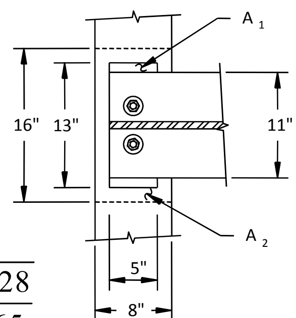
Beam Bearing Plate Example

Concrete Crushing:

$$\phi P_p = \phi\, 0.85\, f'_c\, A_1 \sqrt{\frac{A_2}{A_1}}$$

$$= 0.65\ (0.85 \times 3.0)\ (65)\sqrt{\frac{128}{65}}$$

$$= 151 \text{ k} > 80 \text{ k} \ \ \underline{\text{OK}}$$



Beam Bearing Plate Example

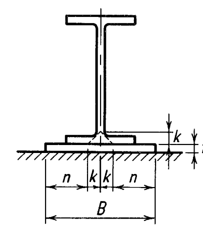
Plate Bending:

$F_y = 36$ ksi

$f_{pu} = R_u / A_1 = 80/(5 \times 13) = 1.23$ ksi

$n = B/2 - k_{design} = 13/2 - 1.08$

$= 5.42$ in.

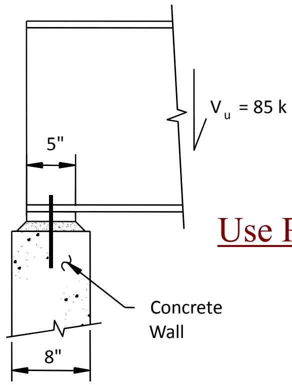


$$t_p = \sqrt{\frac{2 f_{pu} n^2}{0.9 F_y}}$$

$$= \sqrt{\frac{2 \times 1.23 \times 5.42^2}{0.9 \times 36}} = 1.49 \text{ in.}$$

Use PL 1-1/2 in.

Beam Bearing Plate Example



Use PL 1-1/2 x 5 x 1'-1" A36

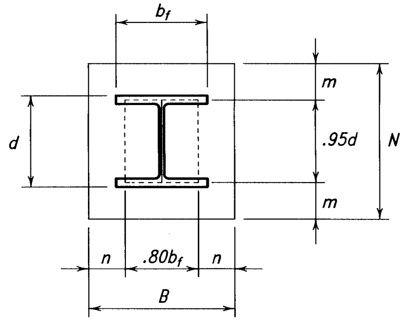
COLUMN BASE PLATE DESIGN

Column Base Plate Design

Required Column Base Plate Thickness

$f_{pu} = R_u / BN$
Let $m' = \max (m \text{ or } n)$
$M_u = f_{pu} (1)\, m'^2/2$

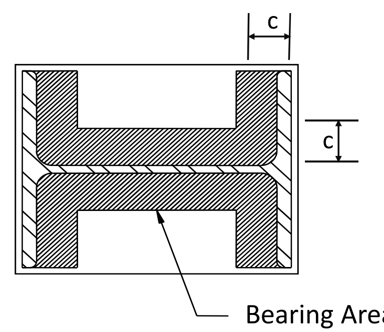
$$t_p = \sqrt{\frac{2 f_{pu} m'^2}{0.9 F_y}}$$



What is t_p if m and n are very small?

Column Base Plate Design

Lightly Loaded Column Base Plates



Column Base Plate Design

Lightly Loaded Column Base Plates

For lightly loaded base plates, replace m′ with

$l = \max\{m, n, \lambda n'\}$, where

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1.0 \qquad n' = \frac{1}{4}\sqrt{db_f}$$

and

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{P_u}{\phi P_p} \qquad \boxed{t_p = \sqrt{\frac{2 f_{pu}\ell^2}{0.9F_y}}}$$

Column Base Plate Design Example

Ex.: Determine if the column base plate shown is adequate if P_u = 250 k.

Column:

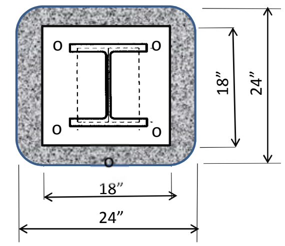
W10x33 d = 9.73 in. b_f = 7.96 in.

PL 1-½ x 18 x 1'-6" A36

Concrete Pedestal:

24 in. by 24 in

$f_{c'}$ = 3.0 ksi



Column Base Plate Design Example

Concrete Crushing

A_1 = 18 x 18 = 324 in²

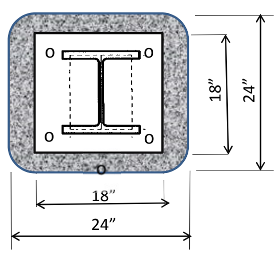
A_2 = 24 x 24 = 576 in²

A_2/A_1 = 576/324 = 1.78 < 4

$$\phi P_p = \phi\, 0.85\, f'_c\, A_1 \sqrt{\frac{A_2}{A_1}}$$

$$= 0.65\ (0.85\times 3.0)\ (324)\sqrt{1.78}$$

$$= 716\text{ k} > 250\text{ k} \ \underline{\text{OK}}$$



Column Base Plate Design Example

Plate Bending

n = 5.82 in. m = 4.38 in.

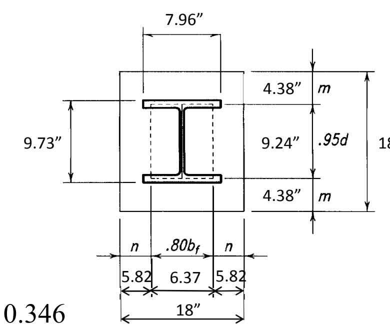
$$n' = (1/4)\sqrt{db_f}$$

$$= (1/4)\sqrt{9.73x7.96}$$

$$= 2.20 \text{ in.}$$

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{P_u}{\varphi P_p}$$

$$= \left(\frac{4x9.73x7.96}{(9.73+7.96)^2}\right)\frac{250}{716} = 0.346$$



Column Base Plate Design Example

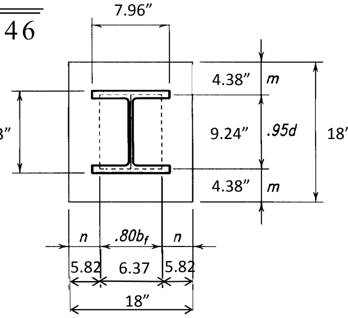
Plate Bending

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} = \frac{2\sqrt{0.346}}{1+\sqrt{1-0.346}}$$
$$= 0.650 \le 1.0$$

$$\lambda n' = 0.635x2.20 = 1.43 \text{ in.}$$

$$l = \max\{m, n, \lambda n'\}$$
$$= \max\{4.38, 5.82, 1.43\} = 5.82$$

$$f_{pu} = P_u/BN = 250/(18x18)$$
$$= 0.772 \text{ ksi}$$

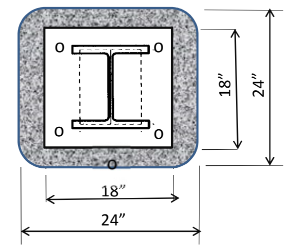


Column Base Plate Design Example

Plate Bending

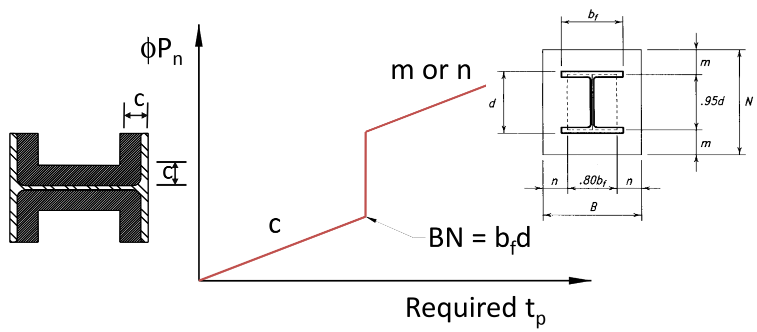
$$t_p = \sqrt{\frac{2 f_{pu}\ell^2}{0.9F_y}} = \sqrt{\frac{2x0.772x5.82^2}{0.9x36}}$$
$$= 1.27 \text{ in.} \le 1.5 \text{ in. } OK$$

PL 1-½ x 18 x 1'-6" A36
is Adequate.



Column Base Plate Design

Variation of ϕP_n with Base Plate Thickness



End of Session 2

Thank You for
Attending

Next Up

Next Session

- October 17, 2017 Shear Connections Part I

TOPICS

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

VirginiaTech Invent the Future 93

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.

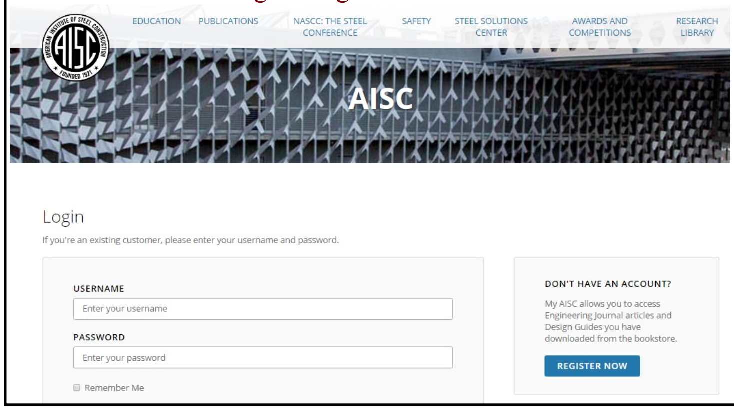
CEUs/PDHS – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHs.

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

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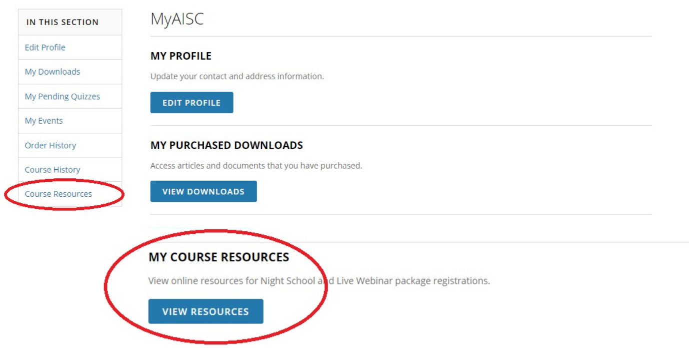
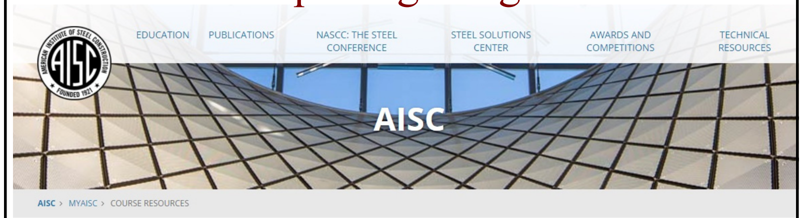
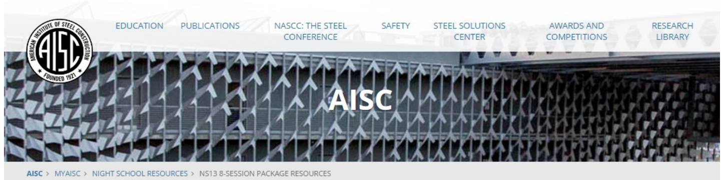


Night School Resources for 8-session package Registrants



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

Night School Resources for 8-session package Registrants



Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

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