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Course Description

Session 6: November 14, 2017 – Moment Connections Part II.

This live webinar covers the basics of prying as needed for the design of connections where prying forces are a concern. Tee-stub-web bolted moment connections and end-plate moment connections are discussed. Column side limit states for these connections are presented. Design examples are included.



Learning Objectives

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

- Describe the characteristics of prying action.
- Describe the design steps of bolted tee stub moment connections.
- List the column side limit states for tee-stub-web moment connections.
- Describe the design methodologies in Design Guide 16: Flush and Extended Multiple-Row Moment End-Plate Connections.



There's always a solution in steel.

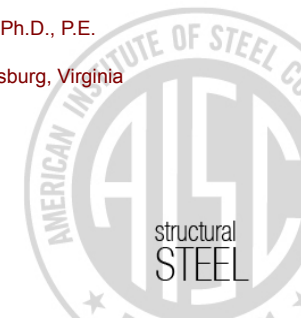
Fundamentals of Connection Design

Session 6: Moment Connections – Part II

November 14, 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**

MOMENT CONNECTIONS PART II

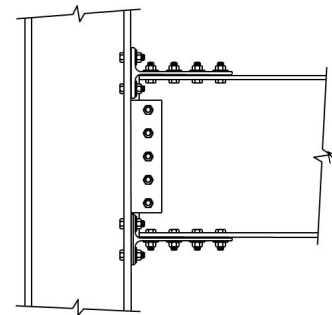


TOPICS

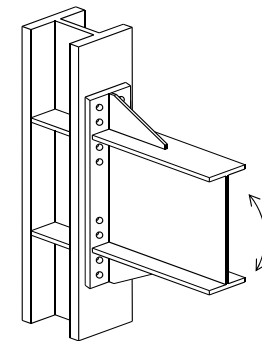
FR Moment Connections

- Connections with Prying Forces
- Basics of Prying
- Tee-Stub / Web Bolted Connections
- Column Side Limit States
- End-Plate Moment Connections
- Design Examples

CONNECTIONS WITH PRYING FORCES





Tee-Stub/Web Bolted





End-Plate Moment

Prying Forces


Contact points
for prying forces


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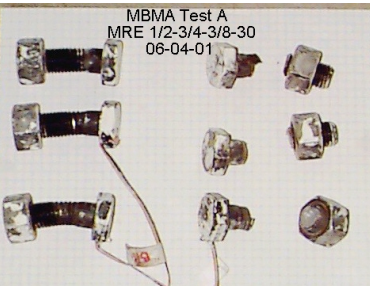
Prying Forces


 14



Prying Forces



MBMA Test A
MRE 1/2-3/4-3/8-30
06-04-01





MBMA Test A
MRE 1/2-3/4-3/8-30
06-04-01


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Prying Forces

Design Models

- “Prying Action” in *Manual* pages 9-10 to 9-14.
- “Kennedy Split-Tee” used for End-Plate Moment Connections.
- Both are basically the same but details differ.
- Nomenclature is entirely different.
- “Thick” Plate Behavior is recommended for Tee-Stub Connections (no prying force).


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Prying Forces

Flange Tee-Stub Bolted/Web Bolted

- Tee Stub → Bolted Hanger Connection

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Bolted Hanger Connections

New Limit States from Manual Design Model

- Tee Flange Bending
- Bolt Tension Rupture

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Bolted Hanger Connections

- Tee Flange Bending

$d_b = \text{bolt diameter}$
 $a' = a + d_b/2$
 $b' = b - d_b/2$
 $\rho(\text{rho}) = b'/a'$

Design model is for one row of bolts
 $p = \text{length of tee per bolt row}$

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Bolted Hanger Connections

- Tee Flange Bending

For LRFD (T_r and $q_r \Leftrightarrow T_u$ and q_u)

$2T_u = 2r_{ut} = \text{Required Strength per bolt row}$
 $q_u = \text{Prying Force}$
 $b_u = T_u + q_u = r_{ut} + q_u = \text{Required Bolt Force}$
 $b_u \leq \phi r_{nt} = 0.75r_{nt}$

For “thick” plate behavior
 (no prying force) $q_u = 0$.

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Bolted Hanger Connections

- Tee Flange Bending

“Thick” Plate Behavior

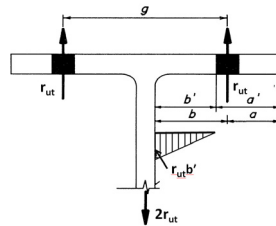
No Prying Force: $q_u = 0$

$$M_u = r_{ut} b' \leq \phi_b M_p$$

$$\phi = 0.9$$

$$M_p = F_u Z_x = F_u (p t_f^2/4)$$

On substitution, the required no prying flange thickness is:



$$t_{np} = \sqrt{\frac{4 r_{ut} b'}{\phi_b p F_u}}$$

Note: F_u not F_y

Bolted Hanger Connections

- Tee Flange Bending w/o Prying Summary

Bolts:

$$\phi_t = 0.75$$

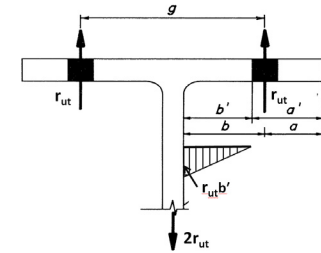
$$q_u = 0$$

$$b_u = r_{ut} \leq \phi_t r_{nt}$$

Flange:

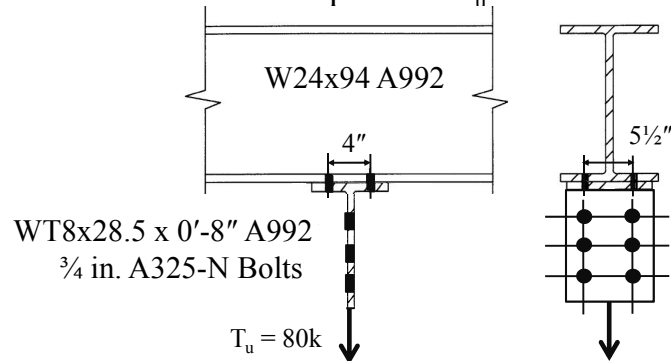
$$\phi_b = 0.9$$

$$t_{np} = \sqrt{\frac{4 r_{ut} b'}{f_b p F_u}}$$



BOLTED HANGER CONNECTION EX.

Example: Determine if the hanger connection shown is adequate for $P_u = 80k$.

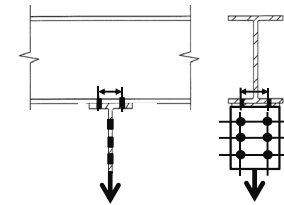


Bolted Hanger Connection Example

Limit States

WT:

1. Flange Bending,
2. Flange Shear Yielding
3. Flange Shear Rupture
4. Web Tension Yielding
5. Tension Rupture
6. Block Shear
7. Shear Transfer at Elements



Bolts:

8. Tension Rupture

Beam:

9. Flange Bending
10. Web Local Yielding

Bolted Hanger Connection Example

Properties

W24x94 $t_f = 0.875$ in.
 $b_f = 9.07$ in.
 $t_w = 0.515$ in.
 $k_{design} = 1.38$ in.

A992 $F_y = 50$ ksi
 $F_u = 65$ ksi
 A325 Bolts
 $F_{nt} = 90$ ksi

WT8x28.5 $b_f = 7.12$ in.
 $t_f = 0.715$ in.
 $t_w = 0.430$ in.

3/4 in. A325 Bolt

$$\phi r_u = 0.75 \times 90 \times 0.4418 = \underline{29.8k}$$

Bolted Hanger Connection Example

Tee-Stub Parameters

$$b = (g - t_w)/2 = (4.0 - 0.430)/2 = 1.79 \text{ in.}$$

$$a = (b_f - g)/2 = (7.12 - 4)/2 = 1.56 \text{ in.}$$

$$b' = b - d_b/2 = 1.79 - 0.75/2 = 1.42 \text{ in.}$$

$$a' = a + d_b/2 = 1.56 + 0.75/2 = 1.94 \text{ in.}$$

L = length of tee = 8.0 in.

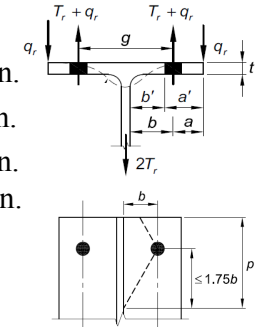
s = bolt spacing = 5.5 in.

$$s/2 = 5.5/2$$

$$= 2.75 \text{ in.} < 1.75b = 1.75(1.79) = 3.13 \text{ in. OK}$$

p = flange length/bolt

$$= s/2 + l_{ch} = 2.75 + (8.0 - 5.5)/2 = 4.0 \text{ in.}$$



Bolted Hanger Connection Example

Required Tee Flange Thickness

$$t_{np} = \sqrt{\frac{4 r_{ut} b'}{\phi_b p F_u}} \quad \text{No Prying}$$

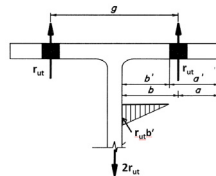
$$= \sqrt{\frac{4 \times 20.0 \times 1.42}{0.9 \times 4.0 \times 65}}$$

$$= 0.697 \text{ in.} < 0.715 \text{ in. for the WT8x28.5 OK}$$

Check Bolt Strength

$$B_u = r_{ut} = 80/4$$

$$= 20k \leq \phi r_n = 29.8k \quad \text{OK}$$



Bolted Hanger Connection Example

Tee Flange Strength Checks

- Shear Yielding of Flange with $t_f = 0.715$ in.

$$\phi R_n = 1.0(0.6F_y)A_{gv}$$

$$= 1.0(0.6 \times 50) (2 \times 8.0 \times 0.715)$$

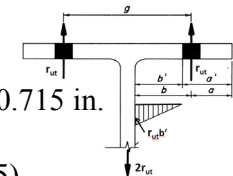
$$= 343 \text{ k} > 80 \text{ k} \quad \text{OK}$$

- Shear Rupture of Flange with $d_h' = 7/8$ in.

$$\phi R_n = 0.75(0.6F_u)A_{nv}$$

$$= 0.75 (0.6 \times 65) (2) (8.0 - 2 \times 0.875) (0.715)$$

$$= 261 \text{ k} > 80 \text{ k} \quad \text{OK}$$



Bolted Hanger Connection Example

Required Beam Flange Thickness

W24x94 $b_f = 9.07''$ $t_f = 0.875''$ $t_w = 0.515''$

$$b = (5.50 - 0.515)/2 = 2.49''$$

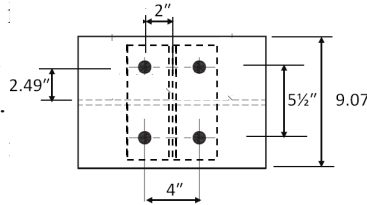
$$b' = 2.49 - 0.75/2 = 2.12''$$

$$a = (9.07 - 5.5)/2 = 1.78''$$

$$p = 1.75(2.49) + 2.0 = 6.36''$$

Assuming no prying:

$$t_{req} = \sqrt{\frac{4 r_{ut} b'}{\phi_b p F_u}} = \sqrt{\frac{4 \times 20 \times 2.12}{0.9 \times 6.36 \times 65}} = 0.675'' < t_f = 0.875'' \quad \text{OK}$$



Bolted Hanger Connection Example

Beam Flange Strength Checks

Shear Yielding and Shear Rupture are not beam flange limit states.

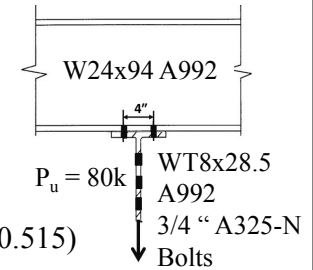
Beam Local Web Yielding

W24x94: $t_w = 0.515''$

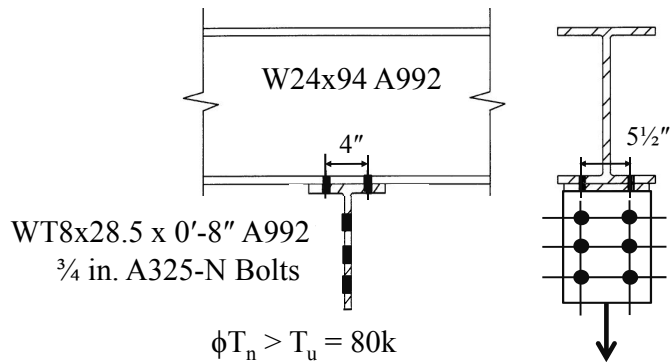
$k_{design} = 1.38''$

$\phi = 1.0$ $l_b = 4''$

$$\begin{aligned} \phi R_n &= 1.0 (5k_{design} + l_b) F_{yw} t_w \\ &= 1.0 (5 \times 1.38 + 4.0) (50 \times 0.515) \\ &= 281k > R_u = 80k \quad \text{OK} \end{aligned}$$

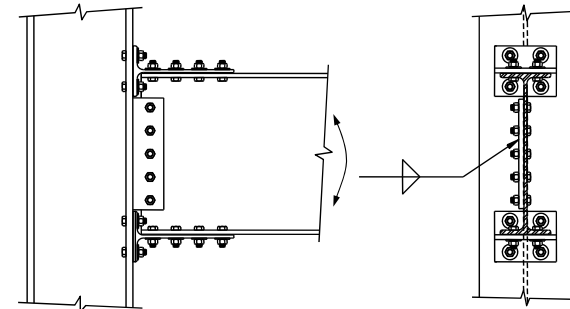


Bolted Hanger Connection Example



The Connection is Adequate

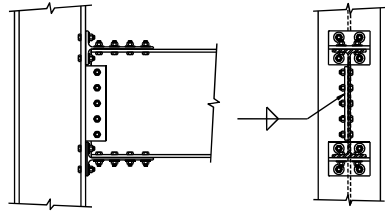
FLANGE TEE-STUB BOLTED / WEB BOLTED



Flange Tee-Stub Bolted / Web Bolted

Limit States

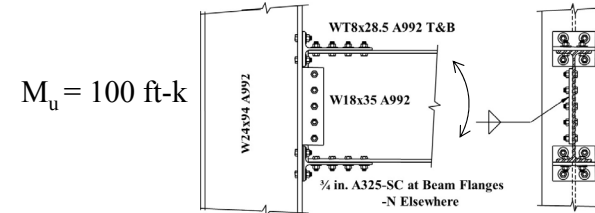
- Tee Stub → Bolted Hanger Connection Design
- Girder → Same as Flange Plate Bolted
- Web Plate → Same as previous (No moment)



Flange Tee-Stub Bolted Example

Example: For the connection shown:

1. List all limit states.
2. Determine if the tee flange-to-column flange connection is adequate.
3. Determine if the bolt slip resistance of the flange bolts is adequate. Class A surface preparation.



Flange Tee-Stub Bolted Example

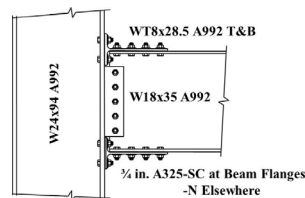
1. Limit States

Beam: Reduced Flexural Strength
 Flange Block Shear
 Web Shear Yielding

Flange Bolts: Bolt Slip (-SC Bolts)

Tee-Stub: Stem Block Shear
 Stem Tension Rupture
 Stem Tension Yielding
 Flange Shear Yielding
 Flange Shear Rupture
 Flange Bending

Tee-Stub Flange Bolts: Tension Rupture



Flange Tee-Stub Bolted Example

1. Limit States

Shear Transfer at Elements:

Flange Plate-to-Beam Flange

Single Plate:

Bolt Shear Rupture

Shear Rupture

Shear Yielding

Weld Rupture

Shear Transfer at Elements:

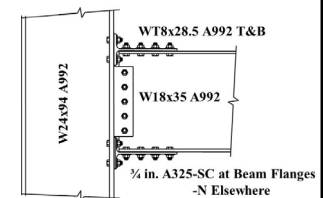
Single Plate-to-Beam Web

Column: Flange Bending (Note: Replaces Flange Local Bending)

Web Local Yielding

Web Local Crippling

Flange Shear Rupture Strength at Weld



Flange Tee-Stub Bolted Example

2. Check Strength of Tee-Stub Flange Connection

W18x35 A992 $d = 17.7$ in. WT8x28.5 $t_w = 0.430$ in.
 Assume same geometry as in previous tee-hanger
 example with one additional row of bolts at stem.

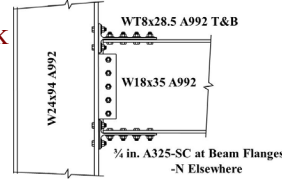
From previous example:

Tee Flange Connection $\phi T_n > 80$ k

$$T_u = M_u / (d + t_w)$$

$$= (100 \times 12) / (17.7 + 0.430)$$

$$= 66.2 \text{ k} < 80 \text{ k} \text{ OK}$$



Flange Tee-Stub Bolted Example

3. Check Beam Flange Bolt Slip Resistance

8 - 3/4 in. A325-SC Bolts $T_b = 28$ k (*Spec.* Table 3.1)

$\phi_{sc} = 1.0$ $\mu = 0.30$ (Class A) $h_f = 1.0$ $n_s = 1$

$\phi_{sc} r_{sc} = \phi_{sc} \mu D_u h_f T_b n_s$ (*Spec.* Eqn. J3-4)

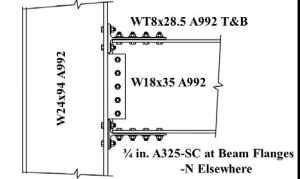
$$= 1.0 \times 0.30 \times 1.13 \times 1.0 \times 28 \times 1$$

$$= 9.49 \text{ k}$$

$\phi_{sc} M_n = \sum \phi_{sc} r_{sc} \times d$

$$= (8 \times 9.49) (17.7 / 12)$$

$$= 112 \text{ ft-kips} > M_u = 100 \text{ ft-kips} \text{ OK}$$

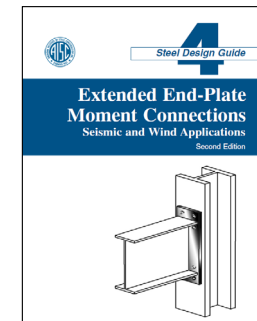


END-PLATE MOMENT CONNECTIONS



End-Plate Moment Connections

AISC MOMENT END-PLATE DESIGN GUIDES

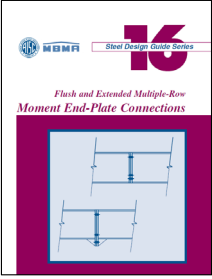



Coming in 2018 Design Guide 4 + 16

End-Plate Moment Connections

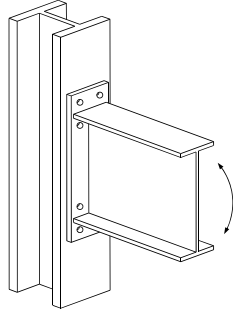
TODAY'S TOPICS

- Types of Moment End-Plate Connections for Wind and Low Seismic
- Basis of AISC Design Guide Procedures
- AISC Design Guide 16
Wind and Low Seismic Moment End-Plate Connections

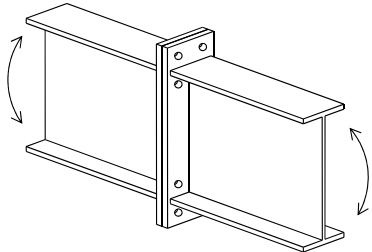


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End-Plate Moment Connections



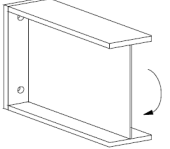
Beam-to-Column



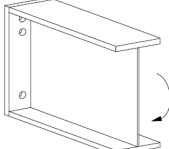
Beam-to-Beam

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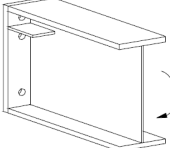
Flush End-Plate Moment Connections



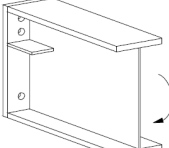
Two Bolt Unstiffened




Four Bolt Unstiffened



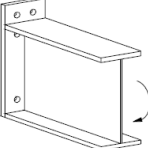
Four Bolt Stiffened



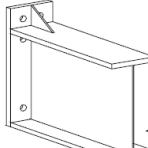
Four Bolt Stiffened

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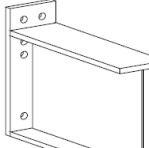
Extended End-Plate Moment Connections



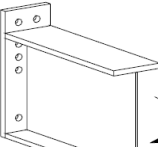
Four Bolt Unstiffened



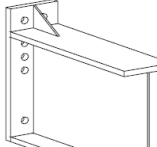
Four Bolt Stiffened




Multiple Row 1/2 Unstiffened



Multiple Row 1/3 Unstiffened



Multiple Row 1/3 Stiffened

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Basis of End-Plate Design Procedures

Stiffness Criterion

- Fully Restrained (FR) Construction

End-Plate Strength

- Yield Line Theory

Bolt Force Prediction

- Pretensioned and Snug-Tightened Bolts
- Including Prying Action

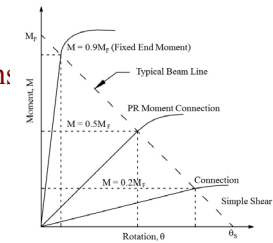
End-Plate Connection Classification

Stiffness Criterion

- Extended End-Plate Connection:
Fully Restrained (FR)

- Flush End-Plate Connections
Fully Restrained (FR)

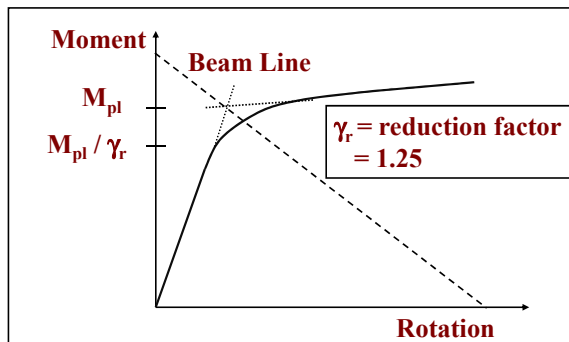
Useable end-plate strength is reduced to satisfy stiffness criterion.



End-Plate Connection Classification

Stiffness Criterion

- FR Flush End-Plate Connections



End-Plate Flexural Strength

End-Plate Flexural Strength: Yield-Line Analysis

- Flexural Strength of Plate:

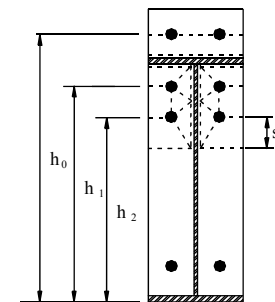
$$M_{pl} = F_{py} t_p^2 Y$$

where:

F_{py} = end-plate material yield stress

t_p = end-plate thickness

Y = geometric yield-line parameter



End-Plate Strength

End-Plate Strength from Yield-Line Analysis

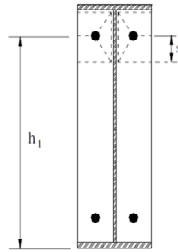
- Internal Work Ex.

$$W_i = \frac{4m_p}{h} (h_1) \left[\frac{b_p}{2} \left(\frac{1}{p_f} + \frac{1}{s} \right) + \frac{2}{g} (p_f + s) \right]$$

where $m_p = F_{py} Z = F_{py}(1) t_p^2 / 4$
 (m_p is the plastic moment per unit length)

- External Work Ex.

$$W_e = M_u \theta = M_u \left(\frac{1}{h_1} \right)$$



End-Plate Strength

End-Plate Strength from Yield-Line Analysis

- Equating $W_i = W_e$

$$M_n = M_{pl} = F_{py} t_p^2 (h_1) \left[\frac{b_p}{2} \left(\frac{1}{p_f} + \frac{1}{s} \right) + \frac{2}{g} (p_f + s) \right]$$

with $s = \frac{1}{2} \sqrt{b_p g}$ from $\frac{dW_i}{ds} = 0$

Note: Yield-line solutions are upper bound solutions and therefore the least internal work is the least upper bound.

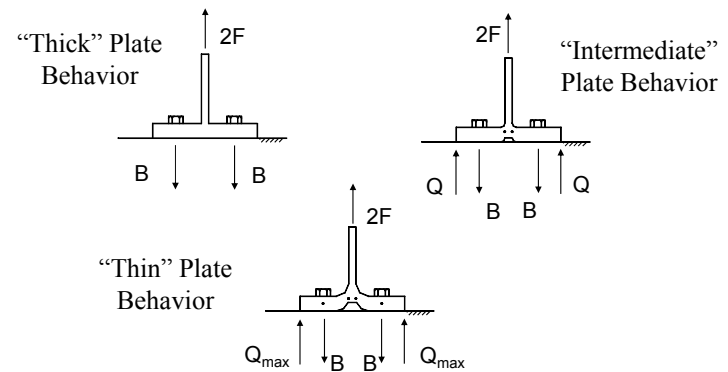
Bolt Forces/Prying Forces

Bolt Force Predictions

- Split-tee analogy
- Connection fails when first bolt reaches its tensile strength
- $\phi r_{nt} = 0.75 F_{nt} A_b$
- Three stages of plate behavior possible.
- Prying forces in bolts considered.

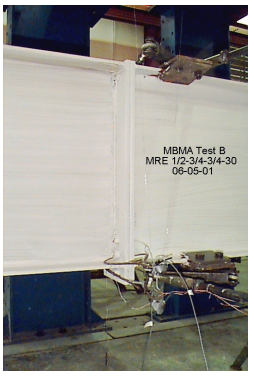
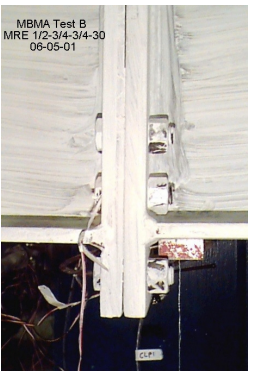
Bolt Forces/Prying Forces



Split-Tee Analogy



Bolt Forces/Prying Forces

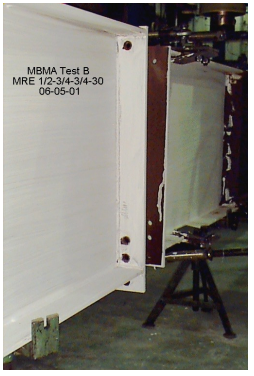
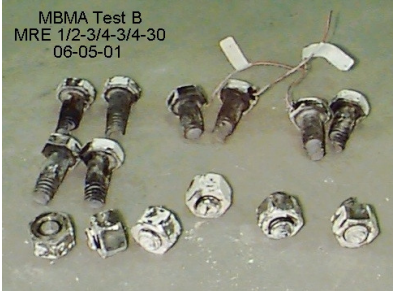
Thick Plate Response








53

Bolt Forces/Prying Forces



Thick Plate Response








54

Bolt Forces/Prying Forces

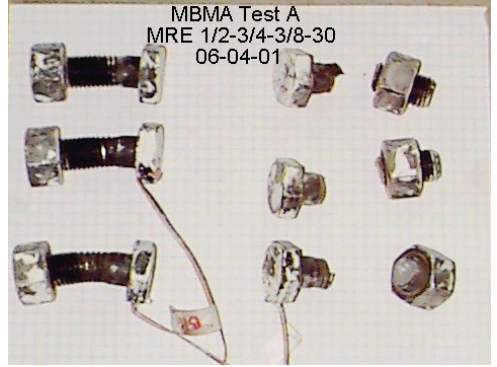
Thin Plate Response








55

Bolt Forces/Prying Forces

Thin Plate Response





56



AISC Design Guide 16

For Wind and Low
 Seismic Applications
 ($R \leq 3$ Designs)



AISC Design Guide 16

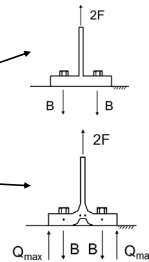
Overview of Design Guide 16

- Background
- Flush End-Plate Design Procedures
 2-Bolt, 4-Bolt, and 4-Bolt Stiffened
- Extended End-Plate Design Procedures
 4-Bolt, 4-Bolt Stiffened, MRE 1/2, MRE 1/3, and
 MRE 1/3 Stiffened
- Gable Frame Panel Zone Design

AISC Design Guide 16

Overview of Design Guide 16

- Two Design Options:
 - “Thick” end-plate option
 (thicker plate, smaller bolts)
 - “Thin” end-plate option
 (thinner plate, larger bolts)
- Two Bolt Pretension Levels:
 - Fully Pretensioned (AISC Table J3.1)
 (Note: Not Slip-Critical)
 - Snug Tightened



AISC Design Guide 16

“Thick” Plate Design Option

- Plate yielding does not occur prior to bolt rupture.
- Minimum diameter bolts
- Thicker end-plate
- Prying forces are negligible
- Design requirement: $\phi_b M_{pl} \geq \phi_t 1.11 M_{np}$
 (end-plate stronger than the bolts)
- M_{np} = no prying flexural strength of bolts
- Design procedure is straight forward.

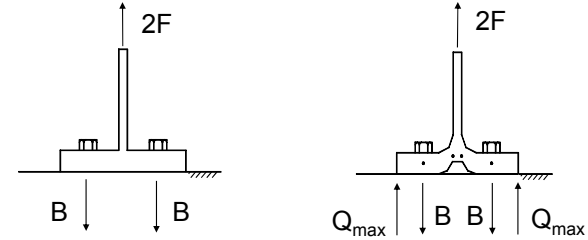
AISC Design Guide 16

“Thin” Plate Design Option

- Plate yielding occurs prior to bolt rupture.
- Minimum end-plate thickness
- Larger diameter bolts
- Prying forces assumed to be maximum
- Design requirement: $\phi_b M_{pl} < \phi_t 1.11 M_{np}$
(bolts stronger than end-plate)
- Design procedure is complicated.

AISC Design Guide 16

“Thick” Plate and “Thin Plate Design Options



“Thick” Plate

$$\phi_b M_{pl} \geq 1.11 \phi_t M_{np}$$

$$Q = 0$$

M_{np} is the “no prying” moment strength of the bolts

“Thin” Plate

$$\phi_b M_{pl} < 1.11 \phi_t M_{np}$$

$$Q = Q_{max}$$

AISC Design Guide 16

Table 4-2 Summary of Four-Bolt Extended Unstiffened Moment End-Plate Analysis

Geometry	Yield-Line Mechanism	Bolt Force Model
End-Plate Yield	$\phi M_u = \phi M_{pl} = \phi F_y F_p s^2 Y$ $Y = \frac{b_2}{2} \left[h_1 \left(\frac{1}{P_{f,1}} + \frac{1}{s} \right) + h_2 \left(\frac{1}{P_{f,2}} - \frac{1}{2} \right) + \frac{2}{g} \left[h_1 (P_{f,1} + s) \right] \right]$ <p>Note: Use $P_{f,1} = s$, if $P_{f,1} > s$</p> $s = \frac{1}{2} \sqrt{F_y g}$ $\phi_b = 0.90$	
Bolt Rupture w/Prying Action	$\phi M_u = \phi M_{pl} = \begin{cases} \phi [2(P_1 - Q_{max,1})d_0 + 2(P_2 - Q_{max,2})d_1] \\ \phi [2(P_1 - Q_{max,1})d_0 + 2(T_1)(d_1)] \\ \phi [2(T_2)(d_0 + d_1)] \end{cases}$ $\phi = 0.75$	
Bolt Rupture No Prying Action	$\phi M_u = \phi M_{np} = \phi [2(P_1)(d_0 + d_1)]$ $\phi = 0.75$	

AISC Design Guide 16

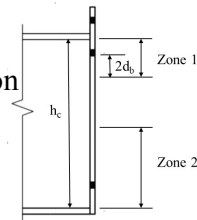
Weld Strength Requirements (*Manual* p. 12-10)

- Beam Flange to End-Plate Weld

$$T_u = \begin{cases} M_u / (d - t_f) \\ \max[0.6[F_y (b_f \times t_f)]] \quad (60\% \text{ of beam flange yield str.}) \end{cases}$$

- Beam Web to End-Plate Weld

Zone 1: Max. of calculated web tension stress and $0.6(F_y t_w)$, k/in.
 Zone 2: Min. $(h_c - 2d_b, h_c/2)$
 $V_u \leq \phi V_n$



AISC Design Guide 16

Ex.4-Bolt Extended Unstiffened "Thick" Plate

- Plate Design Strength: $\phi_b M_{pl}$

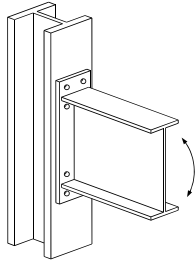
$$\phi_b = 0.9$$

M_{pl} = plate nominal flexural strength
 $= F_{py} t_p^2 Y$

F_{py} = end-plate material yield stress

t_p = end-plate thickness

Y = the yield line parameter



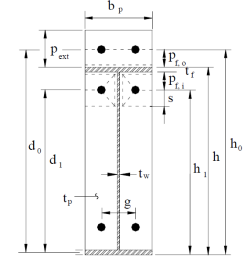
AISC Design Guide 16

Ex.4-Bolt Extended Unstiffened "Thick" Plate

- Plate Design Strength $\phi_b M_{pl} = 0.9 F_{py} t_p^2 Y$

$$Y = \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{f,i}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{f,o}} \right) - \frac{1}{2} \right] + \frac{2}{g} [h_1 (p_{f,i} + s)]$$

$$\text{where } s = \frac{1}{2} \sqrt{b_p g}$$



AISC Design Guide 16

Ex.4-Bolt Extended Unstiffened "Thick" Plate

- No Prying Bolt Moment Strength: $\phi_t M_{np}$

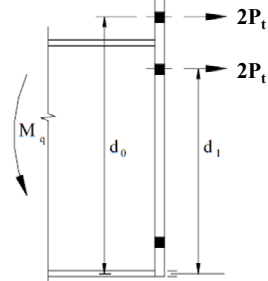
$$\phi_t M_{np} = \phi_t \left[\sum_{i=1}^{N_t} (2P_t d_i) \right]$$

where $\phi_t = 0.75$

$P_t = F_{nt} A_b$ (nominal strength)

$F_{nt} = 90$ ksi for A325 bolts

$= 113$ ksi for A490 bolts



AISC Design Guide 16

Ex.4-Bolt Extended Unstiffened "Thick" Plate

- For "Thick" Plate Design

$$\phi_b = 0.90 \quad \phi_t = 0.75$$

To avoid prying:

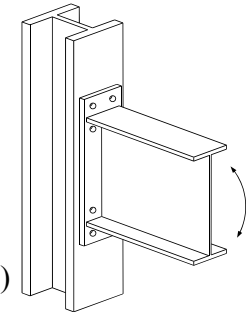
$$\phi_b M_{pl} \geq 1.11 \phi_t M_{np}$$

Required strength:

$$M_u \leq \phi_b M_{pl} / \gamma_r$$

with $\gamma_r = 1.0$ (Extended E-P)

1.25 (Flush E-P)



AISC Design Guide 16

Assumption and Limitations for All Configurations

- **Assumption:**
 - Compression side bolts resist all vertical shear.
- **Limitations:**
 - Bolt diameter $\leq 1\frac{1}{2}$ in.
 - $F_y \leq 50$ ksi
 - Width of end-plate taken in calcs to be $\leq b_f + 1$ in.
 - Bolt gage, $g \leq b_f$
 - With CJP welds, weld access holes not recommended.
- **Minimum Pitch Recommendations**
 - $p_f \geq d_b + \frac{1}{2}$ in. if $d_b \leq 1$ in. $p_f \geq d_b + \frac{3}{4}$ in. if $d_b > 1$ in.

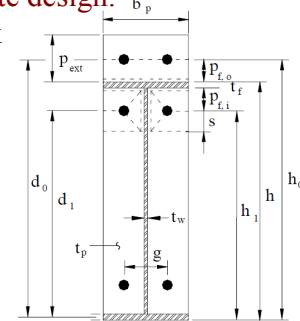
4E Moment End-Plate Design Example

Ex. Determine required bolt diameter and end-plate thickness for the 4-Bolt Extended Unstiffened (4E) shown. Use "Thick" End-Plate design.

$M_u = 2,000$ k-in. $V_u = 33.0$ k
 End-Plate: A572 Gr 50
 Bolt Grade: A325-N

Geometry:

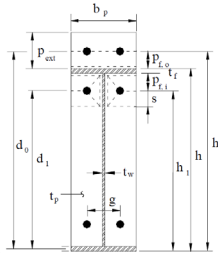
$b_p = b_f = 8$ in. $t_f = \frac{3}{8}$ in.
 $t_w = \frac{1}{4}$ in. $p_{f,i} = 2$ in.
 $h = 18$ in. $p_{f,o} = 2\frac{1}{2}$ in.
 $g = 3\frac{1}{2}$ in. $p_{ext} = 3\frac{1}{2}$ in.



4E Moment End-Plate Design Example

4-Bolt Extended Unstiffened "Thick" End-Plate

Geometry: $b_p = b_f = 8$ in.
 $t_f = \frac{3}{8}$ in. $t_w = \frac{1}{4}$ in.
 $g = 3\frac{1}{2}$ in.
 $h = 18$ in.
 $p_{f,i} = 2$ in.
 $p_{f,o} = 2\frac{1}{2}$ in.
 $p_{ext} = 3\frac{1}{2}$ in.



Calculate: $d_o = 18.0 + 2.5 - 0.375/2 = 20.3$ in.
 $d_i = 18.0 - 0.375 - 2.0 - 0.375/2 = 15.4$ in.
 $h_o = 18 + 2.5 = 20.5$ in.
 $h_i = 15.44 + 0.375/2 = 15.625$ in.
 $\sum d_n = 20.31 + 15.44 = 35.75$

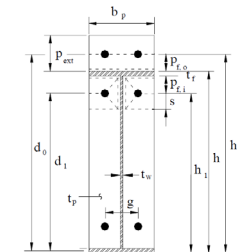
4E Moment End-Plate Design Example

Required Bolt Diameter

$$\sum d_n = 20.31 + 15.44 = 35.75$$

$$d_{b\text{ reqd}} = \sqrt{\frac{2M_u}{\pi \phi_t F_t \left(\sum d_n \right)}}$$

$$= \sqrt{\frac{2 \times 2,000}{\pi \times 0.75 \times 90 \times 35.75}} = 0.726 \text{ in.}$$



Use $\frac{3}{4}$ in. A325-N Bolts

4E Moment End-Plate Design Example

Required End-Plate Thickness

$$t_{p,reqd} = \sqrt{\frac{1.11 \gamma_r (\phi_t M_{np})}{\phi_b F_{py} Y}} \quad \text{Extended End-Plate: } \gamma_r = 1.0$$

End-Plate Yield Line Parameter, Y:

$$\text{With } s = \frac{1}{2} \sqrt{b_p g} = \frac{1}{2} \sqrt{8.0 \times 3.5} = 2.64 \text{ in.}$$

$$Y = \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{f,i}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{f,o}} - \frac{1}{2} \right) + \frac{2}{g} \left[h_1 (p_{f,i} + s) \right] \right]$$

$$= 127 \text{ in.}$$

4E Moment End-Plate Design Example

Required End-Plate Thickness

$$P_t = (\pi d_b^2 / 4) F_t = \pi (0.75)^2 (90) / 4 = 39.8 \text{ kips}$$

$$\phi_t M_{np} = \phi_t [2 P_t (\Sigma d_n)] = 0.75 [2 (39.8) (35.75)]$$

$$= 2,130 \text{ kip-in.}$$

$$t_{p,reqd} = \sqrt{\frac{1.11 \gamma_r (\phi_t M_{np})}{\phi_b F_{py} Y}} = \sqrt{\frac{1.11 \times 1.0 \times 2132}{0.9 \times 50 \times 127.2}}$$

$$= 0.643 \text{ in.}$$

Try 3/4 in. End-Plate

4E Moment End-Plate Design Example

Required End-Plate Thickness

- Check Shear Yielding of Extended Portion of End-Plate

PL 3/4 x 8 A572 Gr50

$$V_u = M_u / h = 2000 / 18.0 = 111 \text{ k}$$

$$\phi V_n = 1.0 (0.6 \times 50) (0.75 \times 8.0) = 180 \text{ k} > 111 \text{ k} \text{ OK}$$

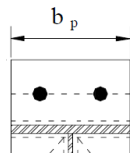
- Check Shear Rupture of Extended Portion of End-Plate

3/4 in. A325-N Bolts

$$\phi V_n = 0.75 (0.6 \times 65) (0.75) (8.0 - 2 \times 7/8)$$

$$= 137 \text{ k} > 111 \text{ k} \text{ OK}$$

Use 3/4 in. End-Plate



4E Moment End-Plate Design Example

Compression Side Bolts

- Check Shear Transfer at the Elements

$$V_u = 33 \text{ k}$$

PL 3/4 x 8 A572 Gr50

2 - 3/4 in. A325-N Bolts

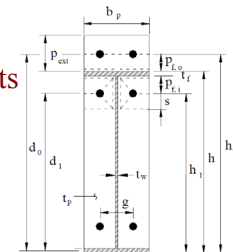
$$\text{Brg.} = 2.4 F_u d_t p$$

$$= (2.4 \times 65) (0.75 \times 0.75) = 87.5 \text{ k}$$

Tear Out will not control by inspection

$$\text{Bolt Shear Rupture } 54 \times 0.442 = 23.9 \text{ k} < 87.5 \text{ k}$$

$$\phi V_n = 0.75 (2 \times 23.9) = 35.8 \text{ k} > 33 \text{ k} \text{ OK}$$



4E Moment End-Plate Design Example

Design of Flange Welds

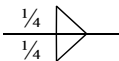
- Beam Flange to End-Plate Weld

$$F_{fu} = \begin{cases} M_u / (d - t_f) = 2,000 / (18.0 - 0.375) = 113 \text{ k} \\ \max \left\{ 0.6[F_y(b_f \times t_f)] = 0.6[50(0.375 \times 8.0)] = 90 \text{ k} \right. \end{cases}$$

Try E70xx fillet welds:

$$D = 113 / [(1.392 \times 1.5)(8.0 + 8.0 - 0.25)] = 3.44 \text{ 1/16's}$$

Min. 3/16

Use 

4E Moment End-Plate Design Example

Design of Web Welds (*Manual 12-9*)

- Beam Web to End-Plate Weld

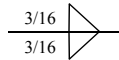
Zone 1: $t_w = 0.25 \text{ in.}$

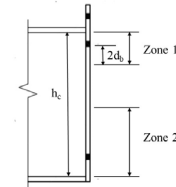
Required weld strength is max. of calculated web tension stress and $0.6(F_y t_w)$, k/in.

Conservatively compare with yield strength of web: $F_y t_w = 50 \times 0.25 \times 1.0 = 12.5 \text{ k per in. of width}$

$$D_{req'd} = 12.5 / (2 \times 1.392 \times 1.5 \times 1.0) = 2.99 \text{ 1/16's}$$

Min. 3/16 in.

Use 



4E Moment End-Plate Design Example

Design of Web Welds (*Manual 12-10*)

- Beam Web to End-Plate Weld

Zone 2: $V_u = 33.0 \text{ k}$

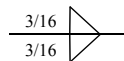
$$h_c = h - 2T_f = 18.0 - 2 \times 0.375 = 17.25 \text{ in.}$$

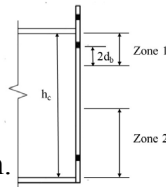
Max. length of weld:

$$\min. \begin{cases} h_c - p_{f1} - 2d_b = 17.25 - 2.0 - 2 \times 0.75 = 13.75 \text{ in.} \\ h_c / 2 = 17.25 / 2 = 8.625 \text{ in.} \end{cases}$$

$$D_{req'd} = 33.0 / (2 \times 1.392 \times 8.625) = 1.38 \text{ 1/16's}$$

Min. 3/16 in.

Use 



4E Moment End-Plate Design Example

4-Bolt Extended Unstiffened "Thick" Plate Design

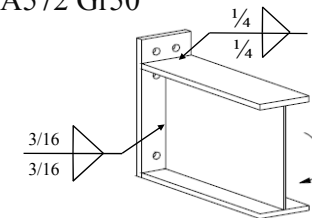
- Final Design

$$M_u = 2,000 \text{ in-kips}$$

$$V_u = 33.0 \text{ k}$$

End-Plate 3/4x8x1'-9 1/2" A572 Gr50

3/4 in. A325-N Bolts



AISC Design Guide 16

Other Limit States

- Column Side
 - Flange Bending: $M_f = F_{yf} t_f^2 Y$
 - Web Local Yielding at Compression Side
 $l_b = (6k_{design} + 2t_p + t_{fb})$ from AISC DG4
 - Web Local Crippling at Compression Side
 - Web Compression Buckling
 - Panel Zone Strength

End of Session 6
Thank You for
Attending

Next Up

Next Session

- Nov. 28, 2017 Introduction to Seismic Connections

TOPICS

- Ductile Mechanisms and Capacity Design
- Qualification Requirements for Special and Intermediate Moment Frame Connections
- Introduction to Prequalified Connections
- Requirements for Concentrically Braced Frames

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 Thursday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG

Quiz and Attendance records: Posted Wednesday mornings.
www.aisc.org/night school - 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 Thursday. 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.



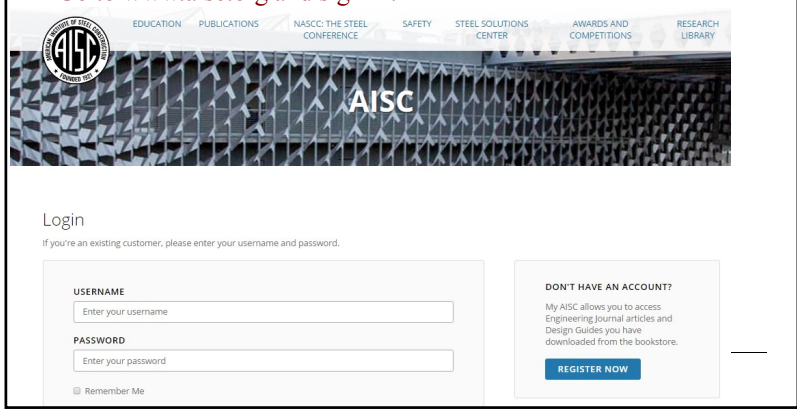
Night School Resources for 8-session package Registrants

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



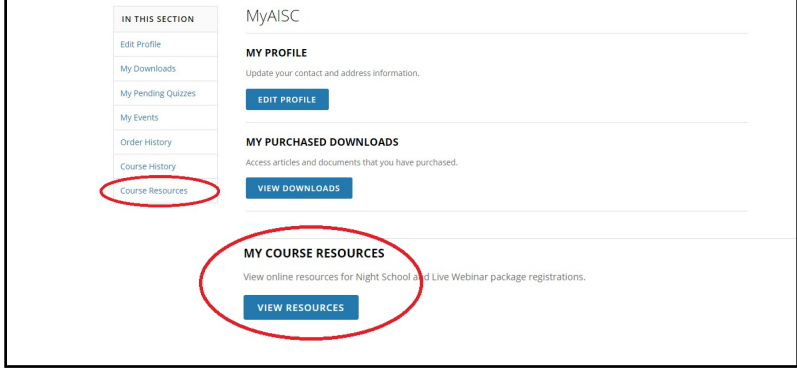
Night School Resources for 8-session package Registrants

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	End 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	Video	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/06/2017 5pm EST	Available 02/06/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/13/2017 5pm EST	Available 02/13/2017 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	4/13/2017 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.



Thank You

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

There's always a solution in steel.

