

## AISC Live Webinars

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Connection Design for Moment Frames and Braced Frames  
Session 2: Moment Connections, Part 2  
February 26, 2020



## AISC Live Webinars

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## AISC Live Webinars

### Course Description

Moment Connections, Part 2  
February 26, 2020

This session will review the prying phenomenon, and its effect on certain moment connections. End-plate and tee-stub moment connections will be discussed. Design examples will be presented to demonstrate concepts.



## AISC Live Webinars

### Learning Objectives

- Explain the prying action phenomenon.
- Identify the design parameters that can eliminate or reduce prying action force.
- List the design steps of bolted tee stub moment connections.
- Describe the design methodologies in AISC Design Guide 16, *Flush and Extended Multiple-Row Moment End-Plate Connections*.



## Connection Design for Moment Frames and Braced Frames

**Session 2: Moment Connections – Part II**  
February 26, 2020



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Associate Professor, University of Kentucky  
Owner, Davis Structural Engineering



## SCHEDULE

- |                     |                                     |
|---------------------|-------------------------------------|
| • February 19, 2020 | Moment Connections Part I           |
| • February 26, 2020 | Moment Connections Part II          |
| • March 4, 2020     | Introduction to Seismic Connections |
| • March 11, 2020    | Bracing Connections                 |



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# MOMENT CONNECTIONS PART II



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## TOPICS

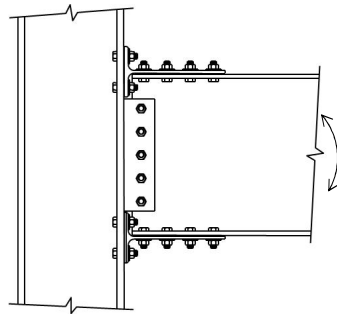
### FR Moment Connections

- Connections with Prying Forces
- Basics of Prying
- Tee-Stub (Bolted) Connections
- End-Plate Moment Connections
- Design Examples

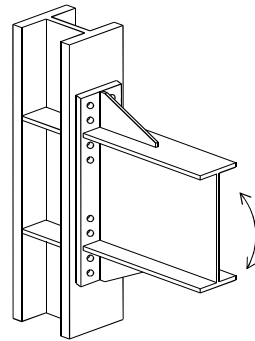


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## CONNECTIONS WITH PRYING FORCES



Tee-Stub/Web Bolted

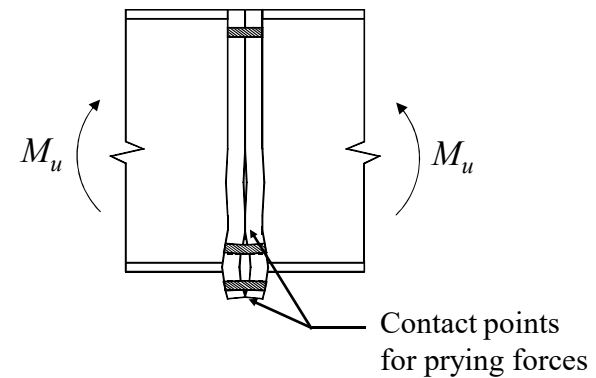


End-Plate Moment



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## Prying Forces



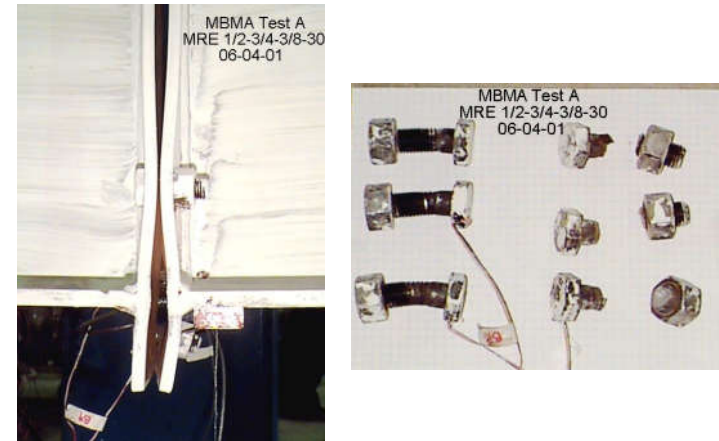
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## Prying Forces



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## Prying Forces



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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).

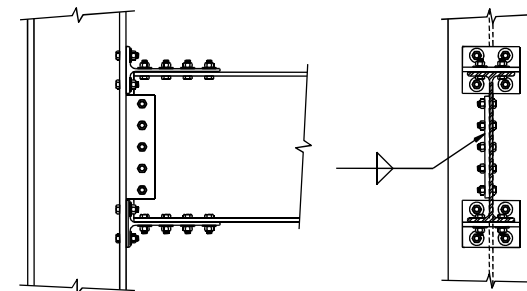


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## Prying Forces

### Flange Tee-Stub Bolted/Web Bolted

- Tee Stub → Bolted Hanger Connection



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### Bolted Hanger Connections

$P_u$

$P_u$

**Limit States from *Manual* Part 9**

- Tee Flange Bending
- Bolt Tension Rupture

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### Bolted Hanger Connections

- Tee Flange Bending  
 $d_b =$  bolt diameter  
 $b' = b - d_b/2$   
 $a' = a + d_b/2 \leq 1.2b + d_b/2$   
 $\rho = b'/a'$

Design model is for one row of bolts  
 $p =$  length of tee per bolt row

Min of  $1.75b$  and  $s/2$

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### Bolted Hanger Connections

- Bolt Strength Criterion  
 For LRFD ( $T_r$  and  $q_r \Rightarrow T_u$  and  $q_u$ )  
 $2T_u =$  required connection strength per bolt row  
 $q_u =$  prying force  
 Required Bolt Strength  $= T_u + q_u$   
 $T_u + q_u \leq \phi r_n$  where  $\phi = 0.75$

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### Bolted Hanger Connections

- “Thick” Plate Behavior  
 No Prying Force:  $q_u = 0$   
 $M_u = T_u b' \leq \phi M_p, \phi = 0.9$   
 $M_p = F_u Z_x = F_u (pt^2/4)$   
 On substitution, the required no prying flange thickness is:
 

$t_{np} = \sqrt{\frac{4T_u b'}{\phi F_u}}$  (Manual Eq. 9-17a)  
 Flange flexural strength is adequate if  $t_f \geq t_{np}$

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### BOLTED HANGER CONNECTION EX.

Example: Determine if the hanger connection shown is adequate for  $P_u = 80$  kips. Emphasizing flange-to-flange connection limit states.

W24x94 A992  
4 in.  
5-1/2 in.  
WT8x28.5 x 0'-8" A992  
3/4 in. Gr. A325-N Bolts  
 $P_u = 80$  kips

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### 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 Bolts

Bolts

8. Tension Rupture

Beam

9. Flange Bending
10. Web Local Yielding

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### Bolted Hanger Connection Example

#### Properties

W24x94	$t_f = 0.875$ in.	A992 $F_y = 50$ ksi
	$b_f = 9.07$ in.	$F_u = 65$ ksi
	$t_w = 0.515$ in.	Gr. A325 Bolts
	$k_{des} = 1.38$ in.	$F_{mt} = 90$ ksi

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

3/4 in. Gr. A325 Bolt

$$\phi r_n = \phi F_{mt} A_b$$

$$= (0.75)(90)(0.4418) = 29.8 \text{ kips}$$

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### Bolted Hanger Connection Example

#### Tee-Stub Parameters

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

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

$L =$  length of tee = 8.0 in.  
 $s =$  bolt spacing = 5.5 in.  
 $s/2 = 5.5/2 = 2.75 \text{ in.} < 1.75b = 1.75(1.79) = 3.13 \text{ in.}$   
 (Use 2.75 in.)

$p =$  flange length / bolt

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

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### Bolted Hanger Connection Example

W24x94 A992

4"

5-1/2"

WT8x28.5 x 0'-8" A992  
3/4 in. Gr. A325-N Bolts

$P_u = 80$  kips

WT flange thick enough.  
W24 flange thick enough.  
Bolts strong enough.  
Other limit states OK.

**The Connection is Adequate**

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### FLANGE TEE-STUB (BOLTED) MOMENT CONNECTIONS

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### Flange Tee-Stub (Bolted) Connections

Limit States

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

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### 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.

Custom Tee from W16x57.  $M_u = 100$  kip-ft

W24x94 A992

W18x35 A992

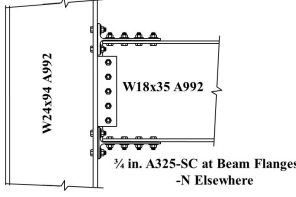
3/4 in. A325-SC at Beam Flanges  
-N Elsewhere

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### Flange Tee-Stub (Bolted) Example

**1. Limit States**

Beam	Reduced Flexural Strength Flange Block Shear
Beam Flange Bolts	Bolt Slip (-SC Bolts) Shear Transfer
Tee-Stub	Stem Block Shear Stem Tensile Rupture Stem Tensile Yielding Flange Shear Yielding Flange Shear Rupture Flange Bending / Prying Action
Tee-Stub Flange Bolts	Tensile Rupture



W24x94 A992  
W18x35 A992  
¾ in. A325-SC at Beam Flanges  
-N Elsewhere



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### Flange Tee-Stub (Bolted) Example

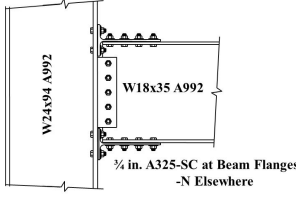
**1. Limit States**

**Web Plate**


- Shear Rupture
- Shear Yielding
- Block Shear
- Weld Rupture
- Shear Transfer

**Column**

- Flange Bending / Prying Action. Replaces flange local bending from *Spec. J10*.
- Web Local Yielding
- Web Local Crippling



W24x94 A992  
W18x35 A992  
¾ in. A325-SC at Beam Flanges  
-N Elsewhere



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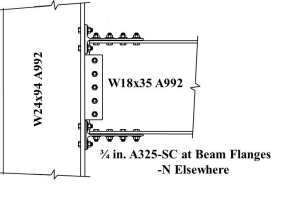
### Flange Tee-Stub (Bolted) Example

**2. Check Strength of Tee-Stub Flange Connection**

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


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

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

$$= 66.2 \text{ kips}$$


W24x94 A992  
W18x35 A992  
¾ in. A325-SC at Beam Flanges  
-N Elsewhere

From previous example:  
Tee Flange Connection  $\phi R_n > 80 \text{ kips} > 66.2 \text{ kips}$ , OK



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### Flange Tee-Stub (Bolted) Example

**3. Check Beam Flange Bolt Slip Resistance**

8 – 3/4 in. A325-SC Bolts

$T_b = 28$  kips (*Spec. Table 3.1*)

$\phi = 1.0$  (standard holes)

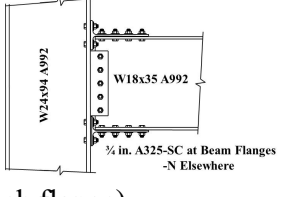
$\mu = 0.30$  (Class A)

$h_f = 1.0$  (no fillers or one filler at each flange)


$n_s = 1$  (slip plane)

$$\phi R_n = \phi \mu D_u h_f T_b n_s \text{ (Spec. Eq. J3-4)}$$

$$= (1.0)(0.30)(1.13)(1.0)(28)(1)(8 \text{ bolts}) = 75.9 \text{ kips}$$

$$R_u = M_u / d = (100)(12) / 17.7 = 67.8 \text{ kips} < 75.9 \text{ kips}, \text{ OK}$$


W24x94 A992  
W18x35 A992  
¾ in. A325-SC at Beam Flanges  
-N Elsewhere



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## END-PLATE MOMENT CONNECTIONS


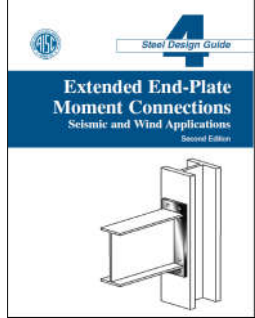






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## End-Plate Moment Connections

### AISC MOMENT END-PLATE DESIGN GUIDES





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## End-Plate Moment Connections

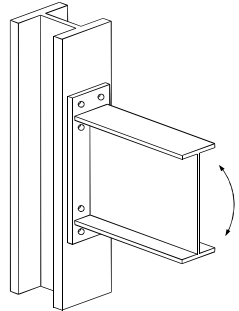
### TODAY'S TOPICS

- 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*

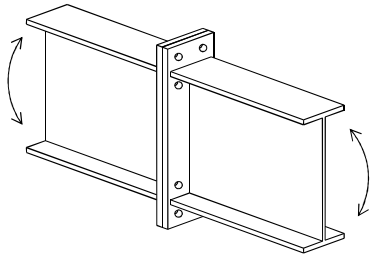



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## End-Plate Moment Connections



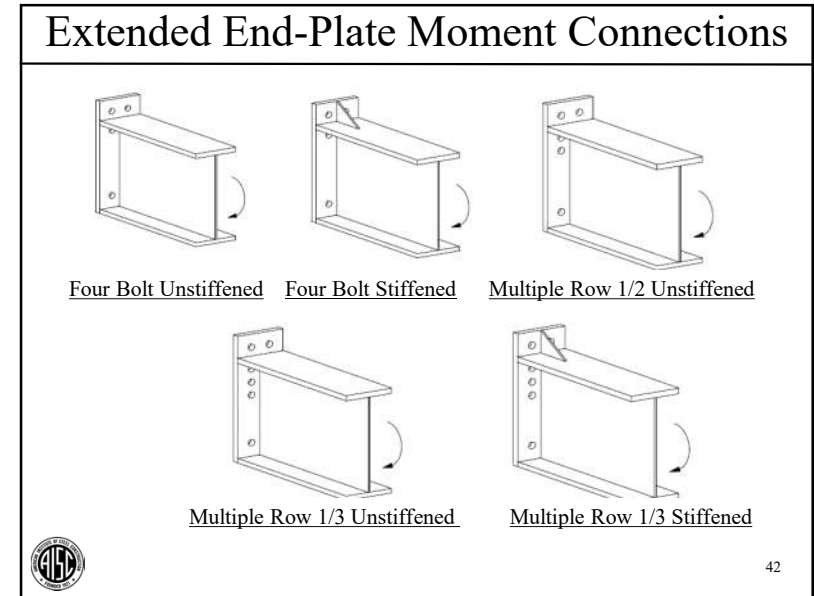
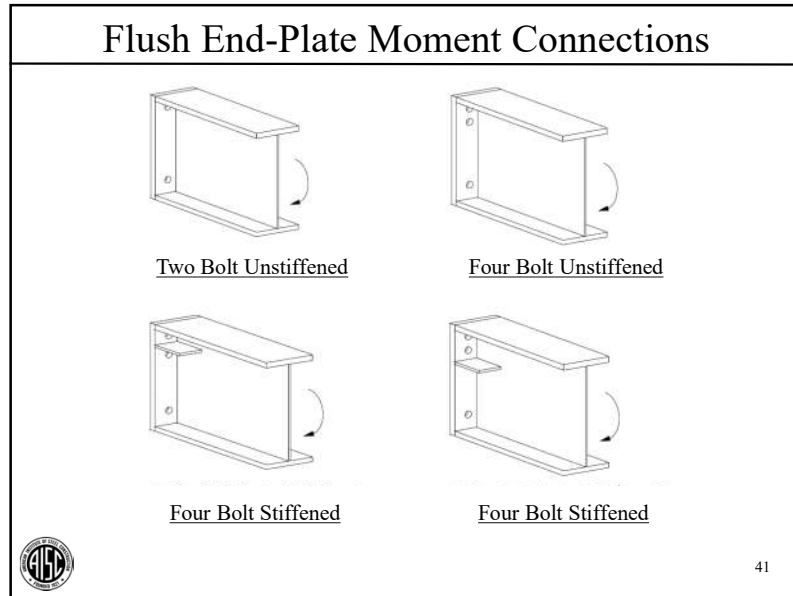
Beam-to-Column



Beam-to-Beam



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### 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

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### End-Plate Connection Classification

Stiffness Criterion

- Extended End-Plate Connections Fully Restrained (FR)
- Flush End-Plate Connections Fully Restrained (FR)

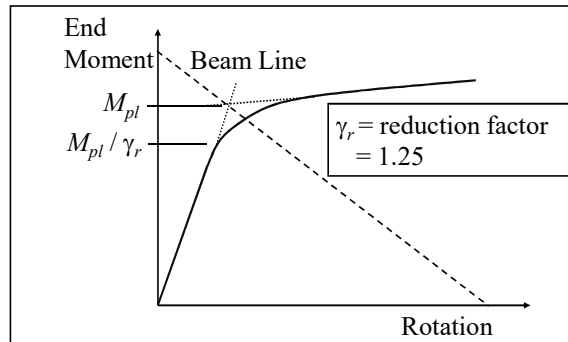
Useable end-plate strength is reduced to satisfy stiffness criterion.

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## End-Plate Connection Classification

### Stiffness Criterion

- FR Flush End-Plate Connections



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## End-Plate Flexural Strength

### End-Plate Flexural Strength: Yield-Line Analysis

- Flexural Strength of Plate

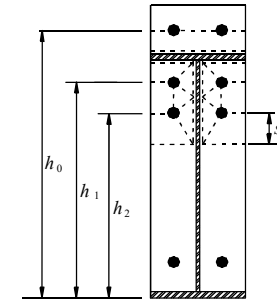
$$M_{pl} = F_y t_p^2 Y$$

where

$F_y$  = end-plate material yield stress

$t_p$  = end-plate thickness

$Y$  = geometric yield-line parameter



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## End-Plate Strength

### End-Plate Strength from Yield-Line Analysis

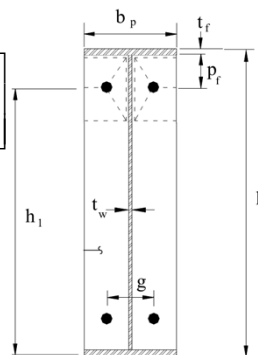
- Internal Work Example

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

where  $m_p = F_y Z = F_y (1") t_p^2 / 4$

- External Work Example

$$W_e = M_u \theta = M_u \frac{1}{h}$$



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## End-Plate Strength

### End-Plate Strength from Yield-Line Analysis

- Equating  $W_i = W_e$

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

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

Note: Yield-line solutions are upper bound solutions and therefore the least strength for any  $s$  is the ultimate strength.




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### Bolt Forces/Prying Forces

Bolt Force Predictions

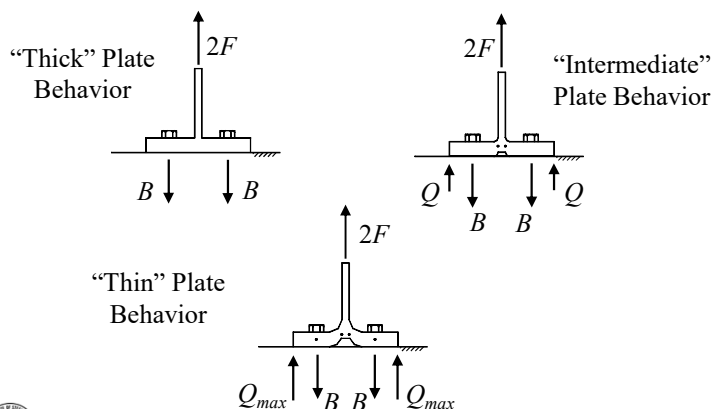

- Split-tee analogy
- Connection fails when first bolt reaches its tensile strength  
 $\phi r_n = \phi_t F_{nt} A_b$  with  $\phi_t = 0.75$
- Three stages of plate behavior possible.
- Prying forces in bolts considered.



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### Bolt Forces/Prying Forces

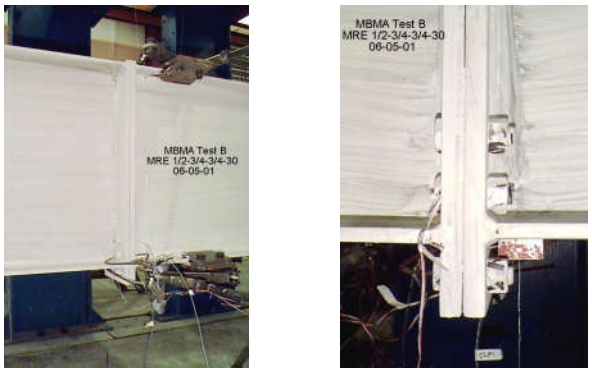

Split-Tee Analogy

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### Bolt Forces/Prying Forces



Thick Plate Response

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### Bolt Forces/Prying Forces

Thick Plate Response

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## Bolt Forces/Prying Forces

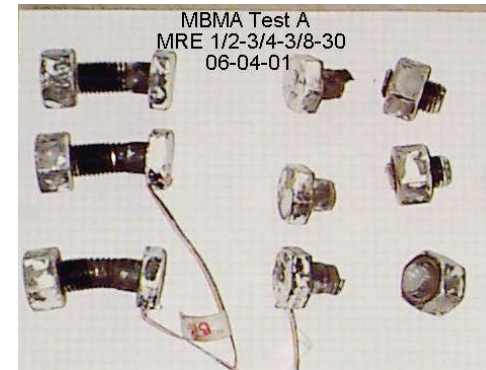
### Thin Plate Response



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## Bolt Forces/Prying Forces

### Thin Plate Response



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## AISC Design Guide 16

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



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## 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



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### 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

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### 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:  $\phi_b M_{pl}/\gamma_r \geq \phi_t 1.11 M_{np}$  (end-plate stronger than the bolts)
- $M_{np}$  = no prying flexural strength of bolts
- Design procedure is straight forward.

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### 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:  $\phi_b M_{pl}/\gamma_r < \phi_t 1.11 M_{np}$  (bolts stronger than end-plate)
- Design procedure is complicated.

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### AISC Design Guide 16

“Thick” Plate and “Thin” Plate Design Options

“Thick” Plate  
 $\phi_b M_{pl}/\gamma_r \geq 1.11 \phi_t M_{np}$   
 $Q = 0$

“Thin” Plate  
 $\phi_b M_{pl}/\gamma_r < 1.11 \phi_t M_{np}$   
 $Q = Q_{max}$

$M_{pl}$  is the flexural strength of the plate  
 $M_{np}$  is the “no prying” flexural strength of the bolts

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### 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	$\phi M_n = \phi_b M_{pl} = \phi_b F_y t_p^2 Y$ $Y = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_{f,i}} + \frac{1}{s} \right) + h_0 \left( \frac{1}{p_{f,o}} - \frac{1}{2} \right) + \frac{2}{g} h_1 (p_{f,i} + s) \right]$ <p style="font-size: small;">Note: Use <math>p_{f,i} &gt; s</math>, if <math>p_{f,i} &gt; s</math></p> $s = \frac{1}{2} \sqrt{b_p g} \quad \phi_b = 0.90$		
Bolt Rupture w/Prying Action	$\phi M_n = \phi M_u = \frac{\phi [2(P_i - Q_{max,i})d_o + 2(P_i - Q_{max,i})d_i]}{\max \left\{ \frac{\phi [2(P_i - Q_{max,i})d_o + 2(T_b)(d_o)]}{\phi [2(T_b)(d_o + d_i)]} \right\}}$ $\phi = 0.75$		
Bolt Rupture No Prying Action	$\phi M_n = \phi M_u = \phi [2(P_i)(d_o + d_i)]$ $\phi = 0.75$		

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### AISC Design Guide 16

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

- Beam Flange to End-Plate Weld

$$T_u = \max \left\{ \begin{array}{l} M_u / (d - t_f) \\ 0.6F_y b_f t_f \end{array} \right.$$

- Beam Web to End-Plate Weld

Zone 1: Max. of calculated web tensile stress and  $0.6F_y t_w$ , kips/in.

Zone 2:  $V_u \leq \phi V_n$

$$L = \min \left\{ \begin{array}{l} h_c - p_{f,i} - 2d_b \\ h_c / 2 \end{array} \right.$$

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### AISC Design Guide 16

#### Example: 4-Bolt Extended Unstiffened “Thick” Plate

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

$\phi_b = 0.9$

$M_{pl}$  = plate nominal flexural strength

$$= F_y t_p^2 Y$$

$F_y$  = end-plate yield stress

$t_p$  = end-plate thickness

$Y$  = yield line parameter

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### AISC Design Guide 16

#### Ex.4-Bolt Extended Unstiffened “Thick” Plate

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

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

where

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

Note: if  $p_{f,i} > s$ , then use  $p_{f,i} = s$

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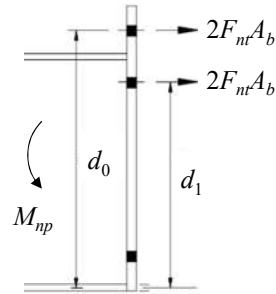
## AISC Design Guide 16

### Ex.4-Bolt Extended Unstiffened “Thick” Plate

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

$$\begin{aligned}\phi_t M_{np} &= \sum_{i=0}^N 2\phi_t F_{nt} A_b d_i \\ &= 2\phi_t F_{nt} A_b \sum_{i=0}^N d_i\end{aligned}$$

where  $\phi_t = 0.75$   
 $F_{nt} = 90$  ksi for Group A  
 $= 113$  ksi for Group B  
 (Spec. Table J3.2)



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## AISC Design Guide 16

### Ex.4-Bolt Extended Unstiffened “Thick” Plate

- For “Thick” Plate Design

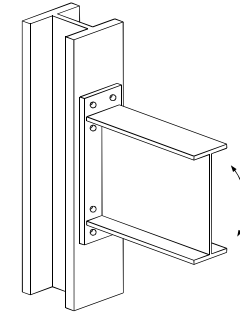
$$\phi_b = 0.90 \quad \phi_t = 0.75$$

To avoid prying:

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

Required strength:

$$\begin{aligned}M_u &\leq \phi_b M_{pl}/\gamma_r \\ &\text{with } \gamma_r = 1.0 \text{ (Extended)} \\ &\quad 1.25 \text{ (Flush)}\end{aligned}$$



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## AISC Design Guide 16

### Assumption and Limitations for All Configurations

- Assumption:
  - Compression side bolts resist all vertical shear.
- Limitations:
  - Bolt diameter  $\leq 1\text{-}1/2$  in.
  - $F_y \leq 50$  ksi
  - Width of end-plate,  $b_p$ , 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
  - $p_f \geq d_b + 1/2$  in. if  $d_b \leq 1$  in.
  - $p_f \geq d_b + 3/4$  in. if  $d_b > 1$  in.



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## 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.

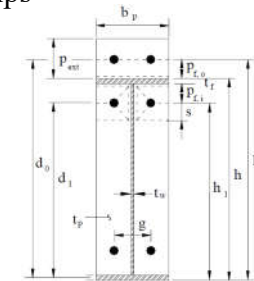
$$M_u = 2,000 \text{ kip-in.} \quad V_u = 33.0 \text{ kips}$$

End-Plate: A572 Gr 50

Bolts: Gr. A325-N

Geometry

$$\begin{aligned}b_p &= b_f = 8 \text{ in.} & t_f &= 3/8 \text{ in.} \\ t_w &= 1/4 \text{ in.} & p_{f,i} &= 2 \text{ in.} \\ h &= 18 \text{ in.} & p_{f,o} &= 2\text{-}1/2 \text{ in.} \\ g &= 3\text{-}1/2 \text{ in.} & p_{ext} &= 3\text{-}1/2 \text{ in.}\end{aligned}$$

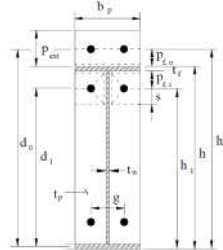


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### 4E Moment End-Plate Design Example

#### 4-Bolt Extended Unstiffened “Thick” End-Plate

Geometry:  $t_f = 3/8$  in.  
 $h = 18$  in.  
 $p_{f,i} = 2$  in.  
 $p_{f,o} = 2-1/2$  in.  
 $p_{ext} = 3-1/2$  in.



Calculate:

$$h_0 = 18 + 2.5 = 20.5 \text{ in.}$$

$$h_1 = 15.4 + 0.375/2 = 15.6 \text{ in.}$$

$$d_0 = 18 + 2.5 - 0.375/2 = 20.3 \text{ in.}$$

$$d_1 = 18.0 - 0.375 - 2.0 - 0.375/2 = 15.4 \text{ in.}$$

$$\sum d_i = 20.3 + 15.4 = 35.8 \text{ in.}$$



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### 4E Moment End-Plate Design Example

#### Required Bolt Diameter

Will ensure plate is thick enough to prevent prying, so:

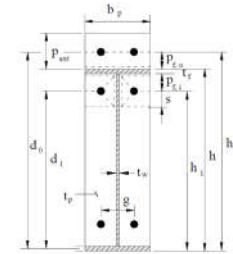
$$M_u \leq \phi_t M_{np} = 2\phi_t F_{nt} A_b \sum_{i=0}^N d_i$$

$$M_u = 2\phi_t F_{nt} A_{b,req} \sum_{i=0}^N d_i$$

$$2000 = 2(0.75)(90)(A_{b,req})(35.8)$$

$$A_{b,req} = 0.414 \text{ in.}^2$$

Use 3/4 in. bolts ( $A_b = 0.442 \text{ in.}^2$ )



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### 4E Moment End-Plate Design Example

#### Required End-Plate Thickness to Prevent Prying

$$\phi_b M_{pt} / \gamma_r \geq \phi_t 1.11 M_{np} \rightarrow t_{p,req} = \sqrt{\frac{1.11 \gamma_r \phi_t M_{np}}{\phi_b F_y Y}}$$

$$\phi_b F_y t_p^2 Y / \gamma_r \geq \phi_t 1.11 M_{np}$$

End-Plate Yield Line Parameter, Y:

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

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



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### 4E Moment End-Plate Design Example

#### Required End-Plate Thickness

$$\phi_t M_{np} = 2\phi_t F_{nt} A_b \sum_{i=0}^N d_i$$

$$= (2)(0.75)(90)(0.4418)(35.8) = 2,130 \text{ kip-in.}$$

$$t_{p,req} = \sqrt{\frac{1.11 \gamma_r \phi_t M_{np}}{\phi_b F_y Y}}$$

$$= \sqrt{\frac{(1.11)(1.0)(2130)}{(0.9)(50)(127)}}$$

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

Try 3/4 in. end-plate.



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### 4E Moment End-Plate Design Example

#### Required End-Plate Thickness

- Check Shear Yielding of Extended Portion of End-Plate

PL3/4 x 8 A572 Gr. 50

$$V_u = (M_u/h)/2 = (2,000/18.0)/2 = 55.5 \text{ kips}$$

$$\phi V_n = 1.0(0.6)(50)(0.75)(8.0) = 180 \text{ kips} > V_u, \text{ OK}$$

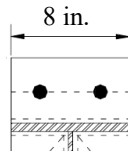
- Check Shear Rupture of Extended Portion of End-Plate

3/4 in. A325-N Bolts → effective hole dia. = 7/8 in.

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

$$= 137 \text{ kips} > V_u, \text{ OK}$$

Use 3/4 in. End-Plate



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### 4E Moment End-Plate Design Example

#### Compression Side Bolts

- Check Shear Transfer

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

PL 3/4 x 8 A572 Gr50

2 – 3/4 in. Gr. A325-N Bolts

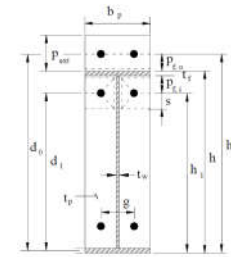
$$r_{n,bearing} = 2.4F_u d t_p \text{ (Spec. Eq. J3-6a)}$$

$$= (2.4)(65)(0.75)(0.75) = 87.5 \text{ kips/bolt}$$

Tearout (Spec. Eq. J3-6c) will not control by inspection

$$r_{n,bolt} = F_{nv} A_b = (54)(0.442) = 23.9 \text{ kips (Spec. Eq. J3-1)}$$

$$\phi V_n = 0.75(23.9)(2 \text{ bolts}) = 35.8 \text{ kips} > 33 \text{ kips, OK}$$



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### 4E Moment End-Plate Design Example

#### Design of Flange Welds

- Beam Flange to End-Plate Weld

$$R_u = \max \left\{ \begin{array}{l} M_u / (d - t_f) = 2,000 / (18 - 3/8) = 113 \text{ kips} \\ 0.6F_y b_f t_f = (0.6)(50)(8)(3/8) = 90 \text{ kips} \end{array} \right.$$

Try E70xx fillet welds:

$$R_u \leq \phi R_n = 1.392(1.5)(D)(l) \text{ (Manual Eq. 8-2)}$$

$$D_{req} = 113 \text{ kips} / [(1.392)(1.5)(8.0 + 8.0 - 0.25)] = 3.44 \text{ sixteenths}$$

Minimum from Spec. Table J2.4 is 3/16 in.

Use  $\frac{1}{4}$



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### 4E Moment End-Plate Design Example

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

- Beam Web to End-Plate Weld

Zone 1:  $t_w = 0.25$  in.

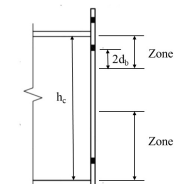
Required weld strength is max. of calculated web tension stress and  $0.6F_y t_w$ , kips/in.

Conservatively compare with yield strength of web:  $F_y t_w = (50)(0.25)(1.0) = 12.5 \text{ kips/in.}$

$$D_{req} = 12.5 / [(1.392)(1.5)(1.0)(2 \text{ welds})] = 2.99$$

Min. is 3/16 in.

Use  $\frac{3}{16}$



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### 4E Moment End-Plate Design Example

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

- Beam Web to End-Plate Weld

Zone 2:  $V_u = 33.0$  kips

$$h_c = h - 2t_f = 18.0 - 2(0.375) = 17.3 \text{ in.}$$

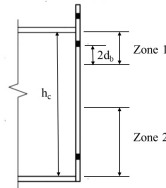
Length of weld:

$$L = \min \begin{cases} h_c - p_{f,i} - 2d_b = 13.8 \text{ in.} \\ h_c / 2 = 8.63 \text{ in.} \end{cases}$$

$$D_{req} = 33.0 / [(1.392)(8.63)(2 \text{ welds})] = 1.38$$

Min. is 3/16 in.

Use  $\frac{3}{16}$



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### 4E Moment End-Plate Design Example

#### 4-Bolt Extended Unstiffened "Thick" Plate Design

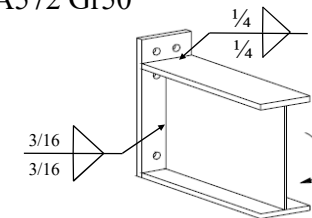
- Final Design

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

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

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

3/4 in. Gr. A325-N Bolts



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### AISC Design Guide 16

#### Other Limit States

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



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### End of Session 2

Thank You for  
Attending

Next Up



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## Next Session

- March 4, 2020 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



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- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!



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## 4-Session Registrants

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Event	Start Date
Structural Design in Steel	3/17/2020 12:00:00 AM
4-Session Package Design of Facade Attachments	5/9/2018 1:00:00 PM
101.20.8-Session Package Night School 20 - Fundamentals of Connection Design	2017/2017 7:00:00 PM
101.20.8-Session Package Night School 24 - Seismic Design in Steel	2/5/2018 7:00:00 PM
101.27.4-Session Package Night School 22 - Design of Facade Attachments	1/5/2018 7:00:00 PM
101.28.8-Session Package Night School 18 - Steel Construction: NF, To Tomorrow's Out	10/15/2018 7:00:00 PM
101.28.8-Session Package Night School 18 - Connection Design	2/4/2018 7:00:00 PM
101.27.4-Session Package Night School 20 - Classical Methods of Structural Analysis	6/3/2018 7:00:00 PM
8-Session Package Design in Steel - Concepts & Examples	1/10/2018 1:00:00 PM

## 4-Session Registrants

### Course Resources

**Design of Facade Attachments**

**4-SESSION PACKAGE RESOURCES**

Event	Date	Handouts	Video	Quiz	Attendance
41. Facade Fundamentals	N/A	Handouts	Video	Quiz	N/A
11. Facade Attachments Part 1	May 9 2018 1:30PM EDT	Handouts	Available 05/11/2018 9:00PM EDT	Available 05/11/2018 9:00 PM EDT	Pending
12. Facade Attachments Part 2	May 16 2018 1:30PM EDT	Handouts	Available 05/18/2018 9:00PM EDT	Available 05/18/2018 1:00 PM EDT	Pending
13. Facade Attachments - Building Lateral Drifts	May 23 2018 1:30PM EDT	Handouts	Available 05/25/2018 9:00PM EDT	Available 05/25/2018 5:00 PM EDT	Pending
Total Exam	N/A			Available 5/27/2018 5:00 PM EDT	

**AISC | Thank you.**

