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Connection Design for Moment Frames and Braced Frames

Session 2: Moment Connections, Part 2

February 26, 2020

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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.

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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*.

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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

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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

<u>FR Moment Connections</u>

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

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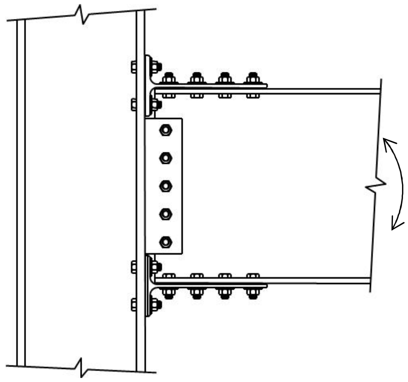
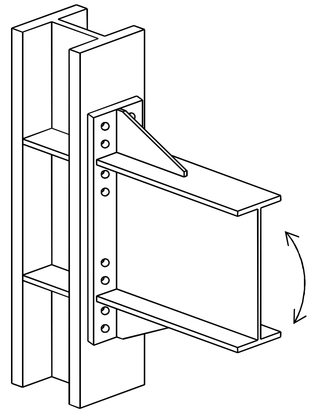
CONNECTIONS WITH PRYING FORCES



Tee-Stub/Web Bolted



End-Plate Moment

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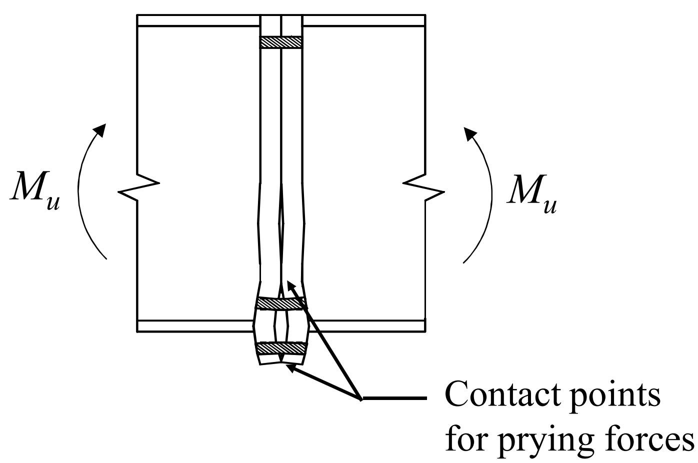
Prying Forces



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Prying Forces

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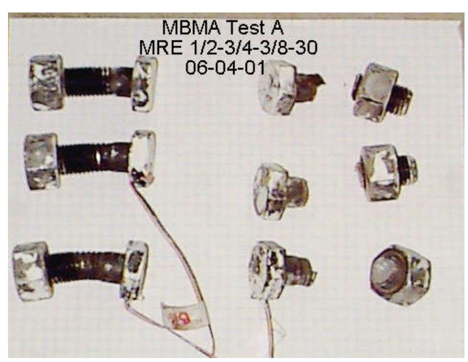
Prying Forces



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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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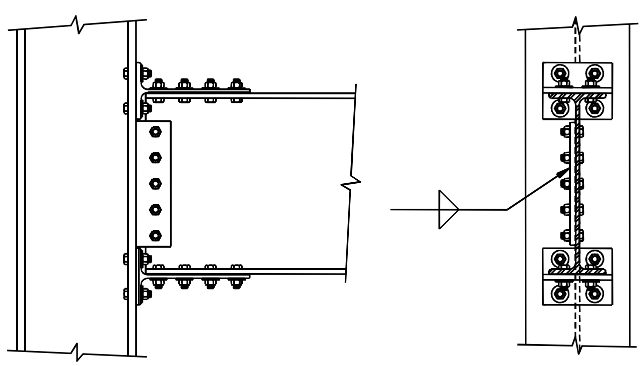
Prying Forces

Flange Tee-Stub Bolted/Web Bolted

- Tee Stub → Bolted Hanger Connection



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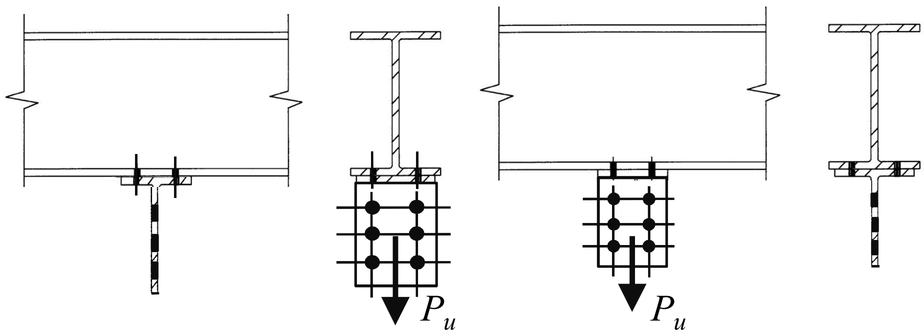
Bolted Hanger Connections



<u>Limit States from *Manual* Part 9</u>

- Tee Flange Bending
- Bolt Tension Rupture

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Bolted Hanger Connections

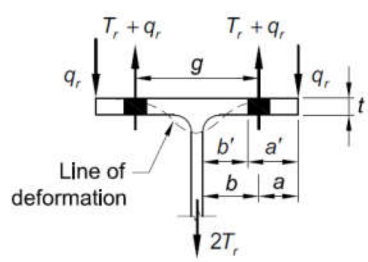
- Tee Flange Bending

  d_b = bolt diameter

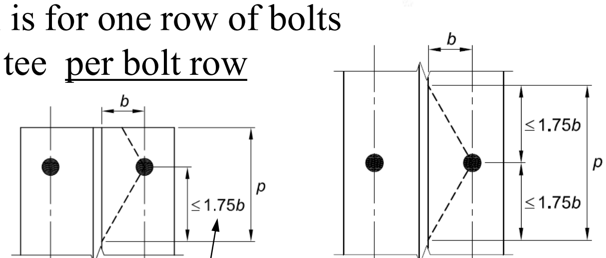
  $b' = b - d_b/2$

  $a' = a + d_b/2 \leq 1.2b + d_b/2$

  $\rho = b'/a'$



Design model is for one row of bolts
p = length of tee <u>per bolt row</u>



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

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Bolted Hanger Connections

- Bolt Strength Criterion

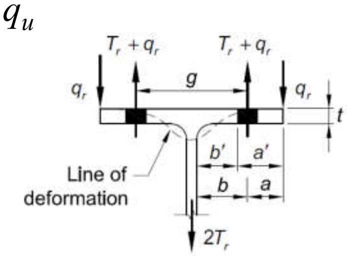
For LRFD (T_r and $q_r \Rightarrow T_u$ and q_u)

$2T_u$ = 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: $q_u = 0$

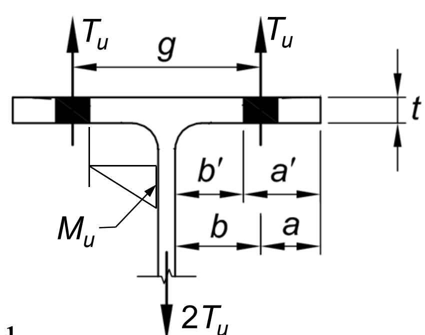
$M_u = T_u b' \leq \phi M_p$, $\phi = 0.9$

$M_p = F_u Z_x = F_u(pt^2/4)$

On substitution, the required no prying flange thickness is:

$$t_{np} = \sqrt{\frac{4T_u b'}{\phi p F_u}}$$

(*Manual* Eq. 9-17a)



Flange flexural strength is adequate if $t_f \geq t_{np}$.

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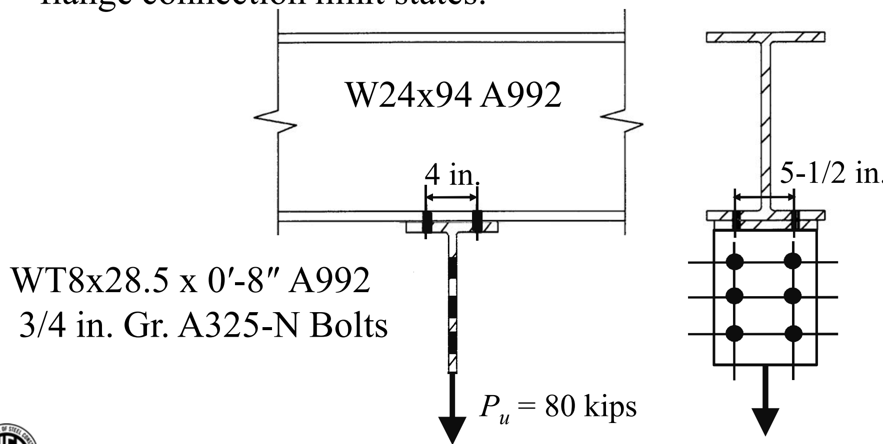
BOLTED HANGER CONNECTION EX.

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



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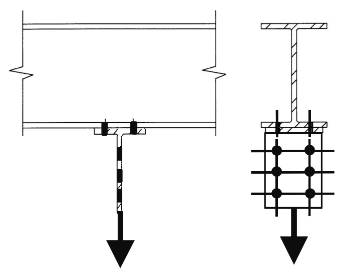
Bolted Hanger Connection Example

<u>Limit States</u>



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.
$b_f = 9.07$ in.
$t_w = 0.515$ in.
$k_{des} = 1.38$ in.

A992 $F_y = 50$ ksi
$F_u = 65$ ksi
Gr. 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. Gr. A325 Bolt

$$\phi r_n = \phi F_{nt} A_b$$
$$= (0.75)(90)(0.4418) = 29.8 \text{ kips}$$

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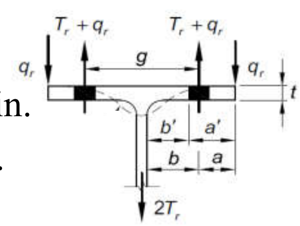
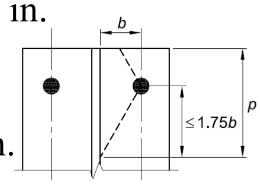
Bolted Hanger Connection Example

Tee-Stub Parameters



$b = (g - t_w)/2 = (4.0 - 0.430) / 2 = 1.79$ in.
$b' = b - d_b/2 = 1.79 - 0.75 / 2 = 1.42$ in.
L = length of tee = 8.0 in.
s = bolt spacing = 5.5 in.
$s/2 = 5.5/2$
$= 2.75 \text{ in.} < 1.75b = 1.75(1.79) = 3.13$ in.
(Use 2.75 in.)
p = flange length / bolt
$= s/2 + l_{eh} = 2.75 + (8.0 – 5.5)/2 = 4.0$ in.



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Bolted Hanger Connection Example

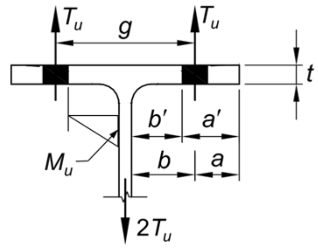
Required Tee Flange Thickness

$$t_{np} = \sqrt{\frac{4T_u b'}{\phi p F_u}}$$

No Prying

$$= \sqrt{\frac{(4)(20)(1.42)}{(0.9)(4)(65)}}$$

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



Check Bolt Strength

$$T_u = 80 \text{ kips} / 4$$
$$= 20 \text{ kips} \leq \phi r_n = 29.8 \text{ kips, OK}$$

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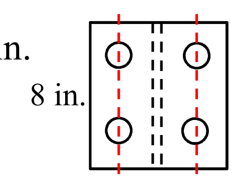
Bolted Hanger Connection Example

Tee Flange Strength Checks

- Shear Yielding of Flange with $t_f = 0.715$ in.
  $\phi R_n = \phi 0.6 F_y A_{gv}$ (*Spec.* Eq. J4-3)
  $= 1.0(0.6)(50)(8.0)(0.715)(2 \text{ planes})$
  $= 343 \text{ kips} > 80 \text{ kips}$ OK
- Shear Rupture of Flange with effective hole diameter, 7/8 in. (*Spec.* B4.3b)
  $\phi R_n = \phi 0.6 F_u A_{nv}$ (*Spec.* Eq. J4-4)
  $= 0.75(0.6)(65)[8.0–(2)7/8](0.715)(2)$
  $= 261 \text{ kips} > 80 \text{ kips}$ OK



Plan

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Bolted Hanger Connection Example

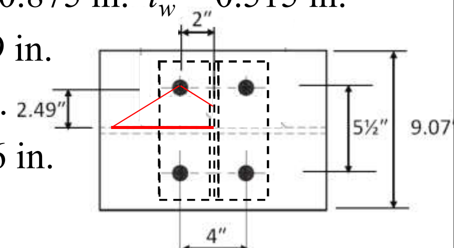
Required Beam Flange Thickness

W24x94 $b_f = 9.07$ in. $t_f = 0.875$ in. $t_w = 0.515$ in.

$b = (5.50 - 0.515)/2 = 2.49$ in.

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

$p = 1.75(2.49) + 2.0 = 6.36$ in.



Verify beam flange is thick enough to prevent prying:

$$t_{np} = \sqrt{\frac{4T_u b'}{\phi p F_u}} = \sqrt{\frac{(4)(20)(2.12)}{(0.9)(6.36)(65)}} = 0.675 \text{ in.} < 0.875 \text{ in. OK}$$

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Bolted Hanger Connection Example

Beam Web Local Yielding

W24x94: $t_w = 0.515$ in.

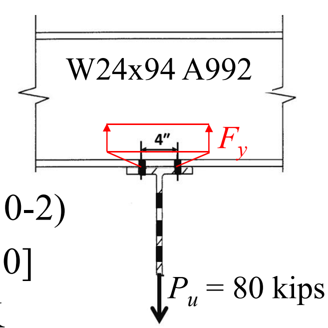
$k_{des} = 1.38$ in.

$l_b = 4$ in.

$\phi R_n = \phi F_{yw} t_w (5k + l_b)$ (*Spec.* Eq. J10-2)

$= (1.0)(50)(0.515)[5(1.38) + 4.0]$

$= 281 \text{ kips} > R_u = 80 \text{ kips}$ OK



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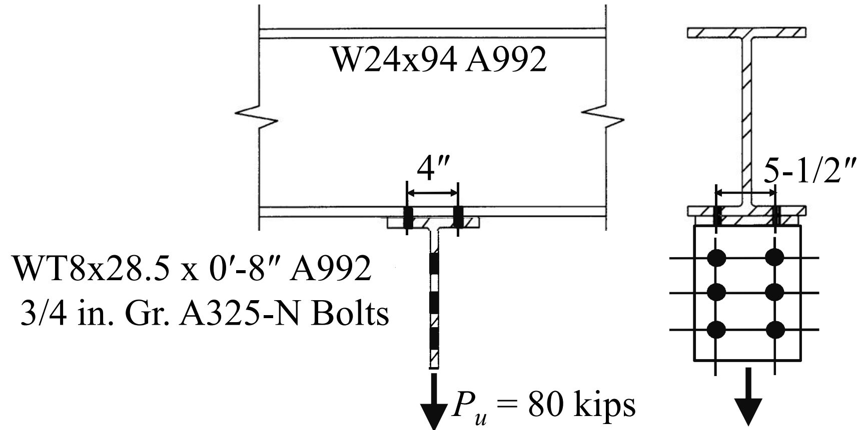
Bolted Hanger Connection Example



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

→ The Connection is Adequate

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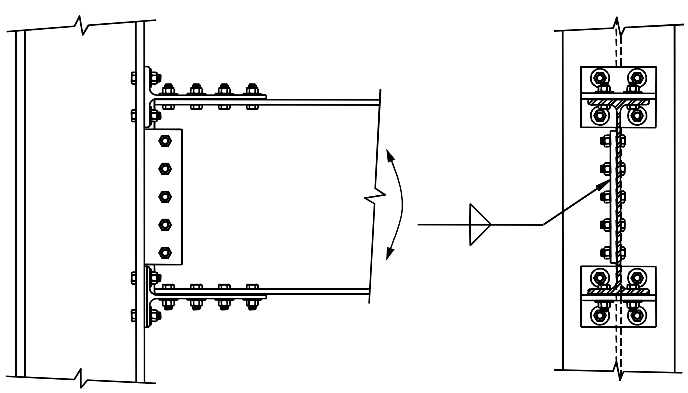
FLANGE TEE-STUB (BOLTED) MOMENT CONNECTIONS



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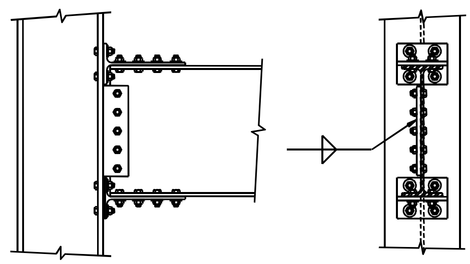
Flange Tee-Stub (Bolted) Connections

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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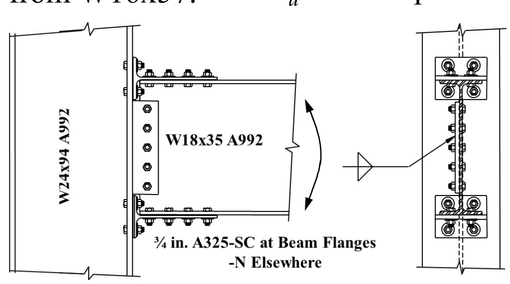
Flange Tee-Stub (Bolted) Example

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



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Flange Tee-Stub (Bolted) Example

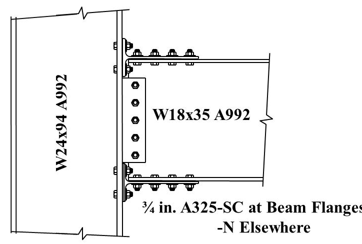
1. Limit States

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



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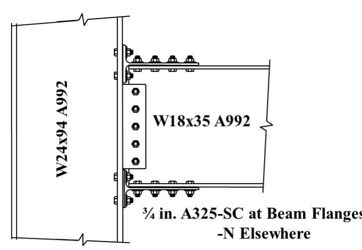
Flange Tee-Stub (Bolted) Example

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



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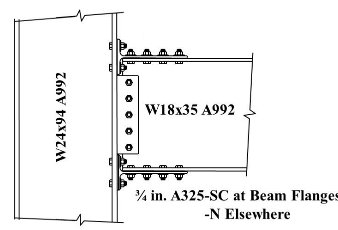
Flange Tee-Stub (Bolted) Example

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.

$R_u = M_u/(d + t_w)$
$= (100 \text{ x } 12)/(17.7 + 0.430)$
$= 66.2$ kips



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

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Flange Tee-Stub (Bolted) Example

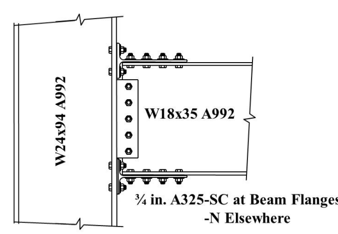
3. Check Beam Flange Bolt Slip Resistance

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)

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

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

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

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END-PLATE MOMENT CONNECTIONS

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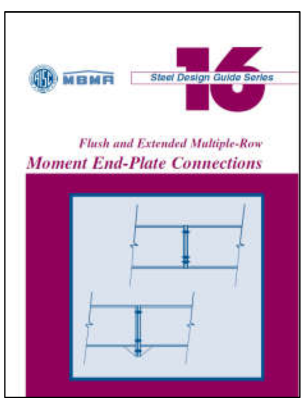
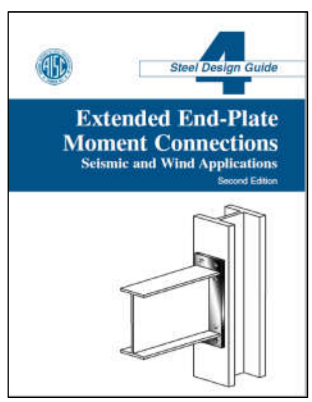
End-Plate Moment Connections

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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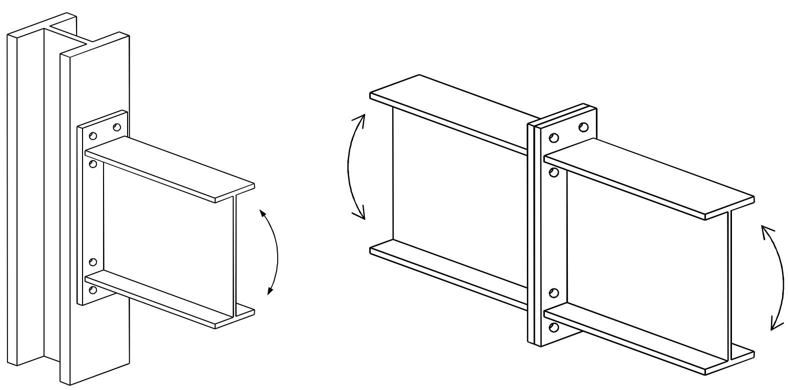
End-Plate Moment Connections



Beam-to-Column

Beam-to-Beam

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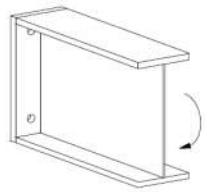
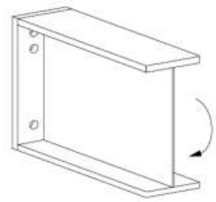
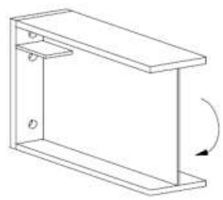
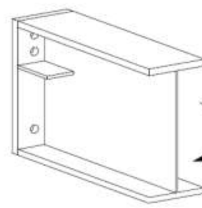
Flush End-Plate Moment Connections


Two Bolt Unstiffened


Four Bolt Unstiffened


Four Bolt Stiffened


Four Bolt Stiffened

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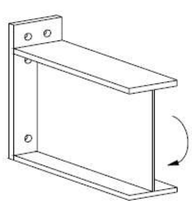
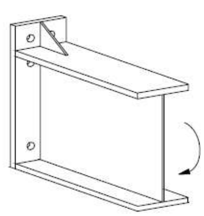
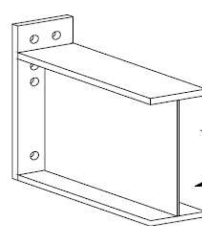
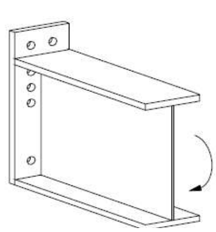
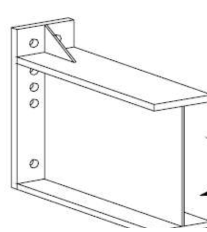
Extended End-Plate Moment Connections


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

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End-Plate Connection Classification

Stiffness Criterion

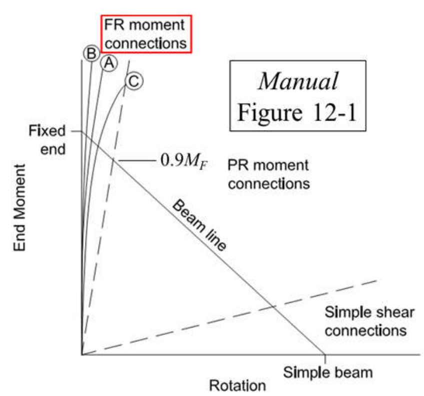
- Extended End-Plate Connections

  Fully Restrained (FR)

- Flush End-Plate Connections

  Fully Restrained (FR)

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



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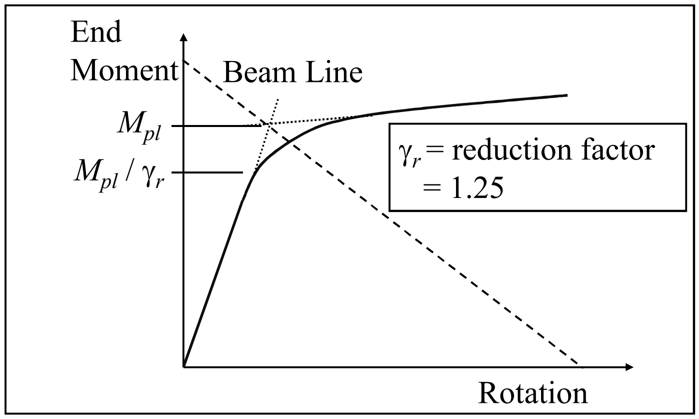
End-Plate Connection Classification

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

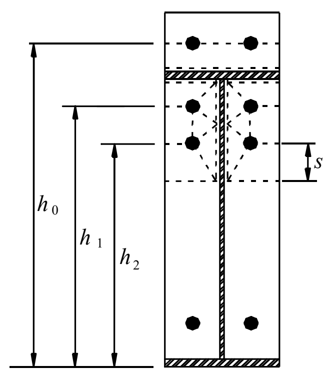
$$M_{pl} = F_y t_p^2 \, Y$$

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

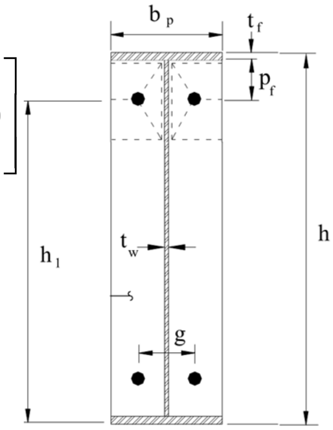
- Internal Work Example

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

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 $W_i = W_e$

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

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

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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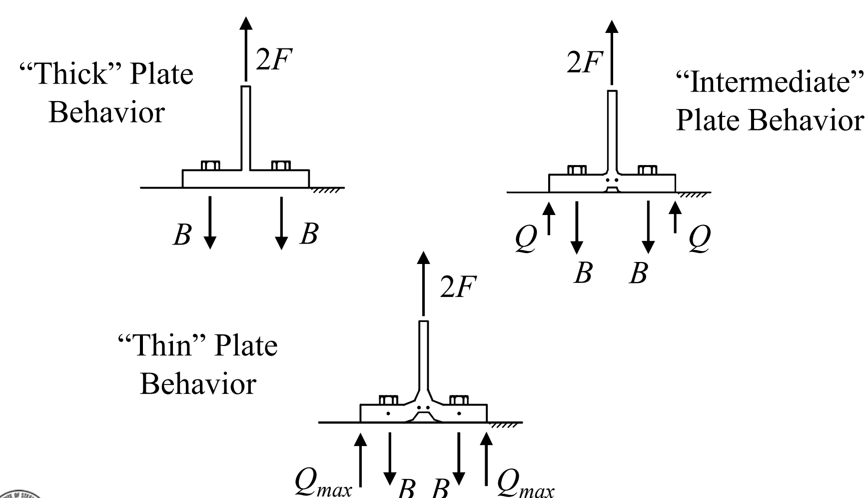
Bolt Forces/Prying Forces

Split-Tee Analogy



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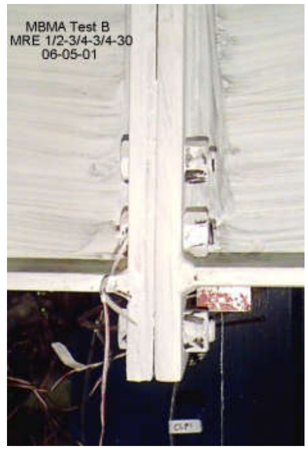
Bolt Forces/Prying Forces

Thick Plate Response



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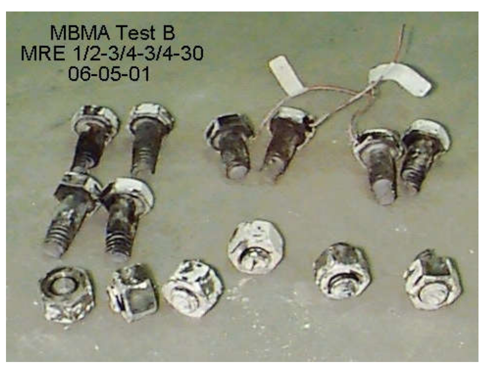
Bolt Forces/Prying Forces

Thick Plate Response



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Bolt Forces/Prying Forces

Thin Plate Response

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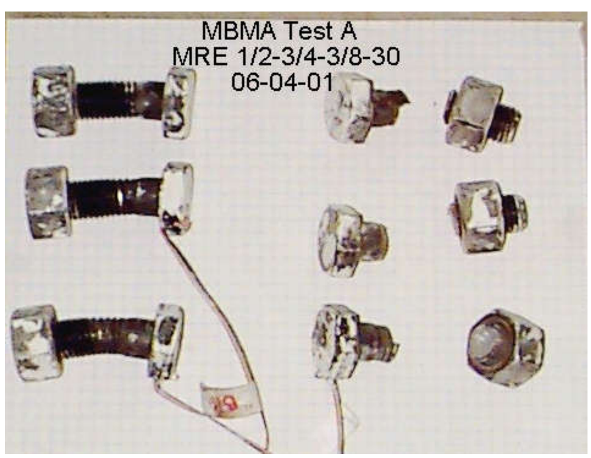
Bolt Forces/Prying Forces

Thin Plate Response



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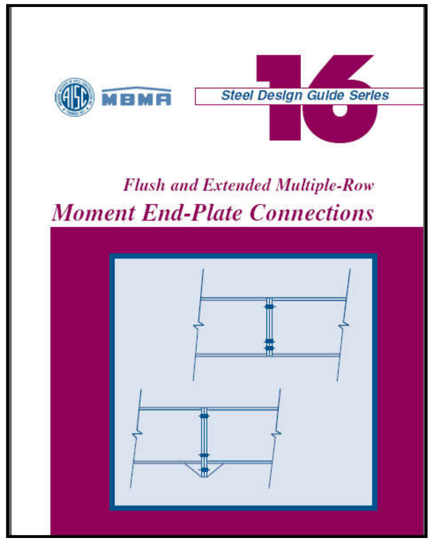
AISC Design Guide 16

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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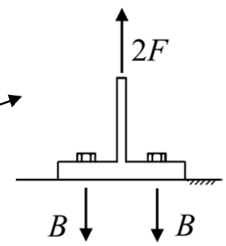
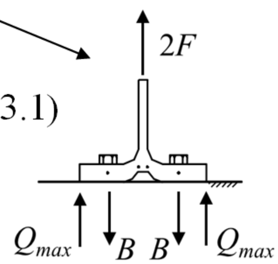
AISC Design Guide 16

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 (AISCS 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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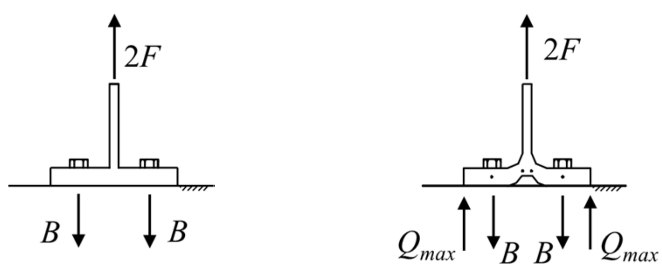
AISC Design Guide 16

“Thick” Plate and “Thin” Plate Design Options



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

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

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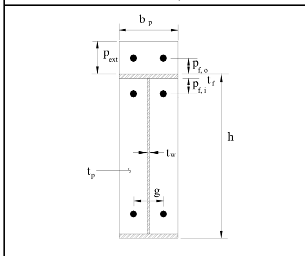
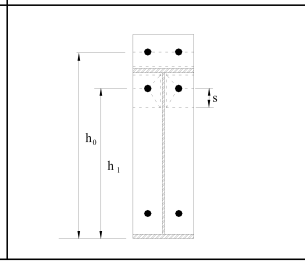
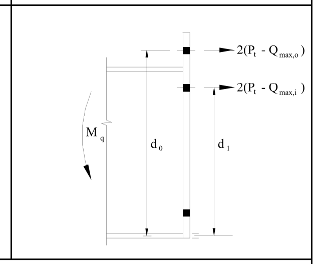
AISC Design Guide 16

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

Geometry	Yield-Line Mechanism	Bolt Force Model
		
End-Plate Yield	$\phi M_n = \phi_b M_{pl} = \phi_b F_{py} t_p^2 Y$ $Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}} + \frac{1}{s}\right) + h_0\left(\frac{1}{p_{f,o}}\right) - \frac{1}{2}\right] + \frac{2}{g}\left[h_1\left(p_{f,i} + s\right)\right]$ Note: Use $p_{f,i} = s$, if $p_{f,i} > s$ $s = \frac{1}{2}\sqrt{b_p g}$ $\phi_b = 0.90$	
Bolt Rupture w/Prying Action	$\phi M_n = \phi M_q = \max \begin{vmatrix} \phi\left[2(P_t - Q_{max,o})d_0 + 2(P_t - Q_{max,i})d_1\right] \\ \phi\left[2(P_t - Q_{max,o})d_0 + 2(T_b)(d_1)\right] \\ \phi\left[2(P_t - Q_{max,i})d_1 + 2(T_b)(d_0)\right] \\ \phi\left[2(T_b)(d_0 + d_1)\right] \end{vmatrix}$ $\phi = 0.75$	
Bolt Rupture No Prying Action	$\phi M_n = \phi M_{np} = \phi\left[2(P_t)(d_0 + d_1)\right]$ $\phi = 0.75$	

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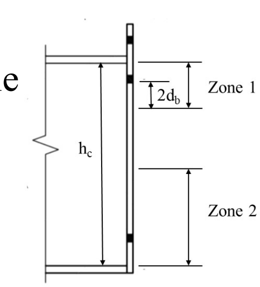
Weld Strength Requirements (*Manual* p. 12-10)

- Beam Flange to End-Plate Weld

$$T_u = \max \begin{vmatrix} M_u / (d - t_f) \\ 0.6 F_y b_f t_f \end{vmatrix}$$

- Beam Web to End-Plate Weld
  Zone 1: Max. of calculated web tensile stress and $0.6F_y t_w$, kips/in.
  Zone 2: $V_u \leq \phi V_n$



$$L = \min \begin{vmatrix} h_c - p_{f,i} - 2d_b \\ h_c / 2 \end{vmatrix}$$

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Example: 4-Bolt Extended Unstiffened "Thick" Plate

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

$\phi_b = 0.9$

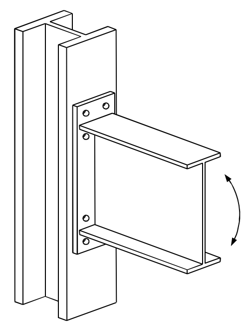
M_{pl} = plate nominal flexural strength

$= F_y t_p^2 Y$

F_y = end-plate yield stress

t_p = end-plate thickness

Y = yield line parameter



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Ex.4-Bolt Extended Unstiffened "Thick" Plate

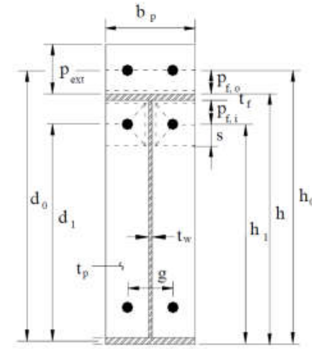
- Plate Design Strength $\phi_b M_{pl} = \phi_b F_{py} t_p^2 Y$

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

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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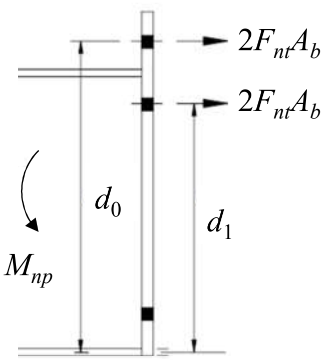
Ex.4-Bolt Extended Unstiffened "Thick" Plate

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

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

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

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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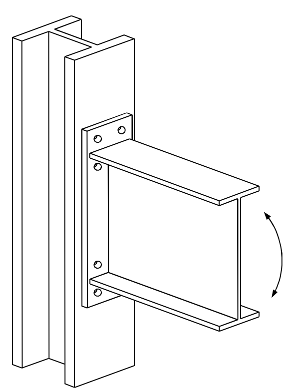
Ex.4-Bolt Extended Unstiffened "Thick" Plate

- For "Thick" Plate Design
  ϕ_b= 0.90 ϕ_t = 0.75

  To avoid prying:
  $\phi_b M_{pl}/\gamma_r \geq 1.11\phi_t M_{np}$

  Required strength:
  $M_u \leq \phi_b M_{pl}/\gamma_r$
  with γ_r = 1.0 (Extended)
  1.25 (Flush)



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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 ≤ 1-1/2 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 kip-in. V_u = 33.0 kips

End-Plate: A572 Gr 50

Bolts: Gr. A325-N

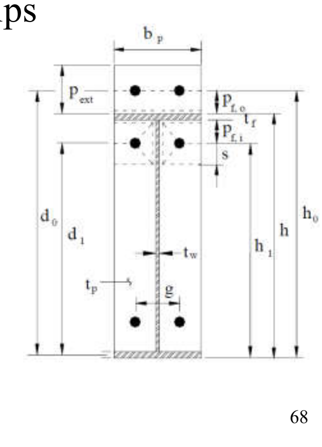
Geometry

$b_p = b_f$ = 8 in. t_f = 3/8 in.

t_w = 1/4 in. $p_{f,i}$ = 2 in.

h = 18 in. $p_{f,o}$ = 2-1/2 in.

g = 3-1/2 in. p_{ext} = 3-1/2 in.



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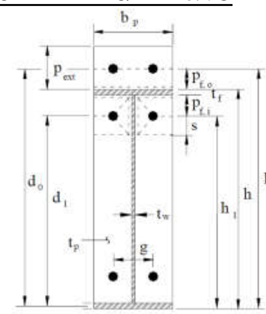
4E Moment End-Plate Design Example

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 in.
h_1 =15.4 + 0.375/2 = 15.6 in.
d_0 = 18 + 2.5 - 0.375/2 = 20.3 in.
d_1 = 18.0 - 0.375 - 2.0 - 0.375/2 = 15.4 in.
$\sum d_i$ = 20.3 + 15.4 = 35.8 in.

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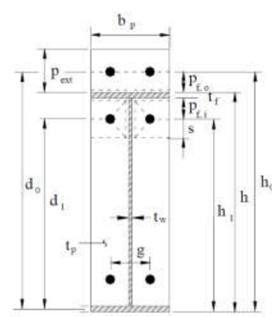
4E Moment End-Plate Design Example

Required Bolt Diameter

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



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

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

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

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

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

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4E Moment End-Plate Design Example

Required End-Plate Thickness to Prevent Prying

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

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

End-Plate Yield Line Parameter, Y:

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}} + \frac{1}{s}\right) + h_0 \frac{1}{p_{f,o}} - \frac{1}{2}\right] + \frac{2}{g} h_1 \left(p_{f,i} + s\right)$$
$$= 127 \text{ in.}$$

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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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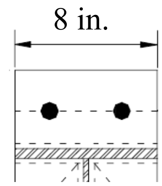
4E Moment End-Plate Design Example

Required End-Plate Thickness



- Check Shear Yielding of Extended Portion of End-Plate

  PL3/4 x 8 A572 Gr. 50

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

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

- Check Shear Rupture of Extended Portion of End-Plate

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

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

  $= 137$ kips $> V_u$, OK

Use 3/4 in. End-Plate

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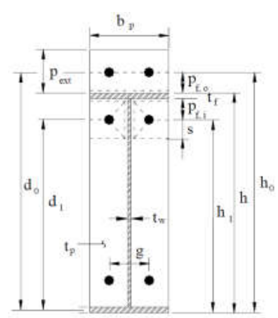
4E Moment End-Plate Design Example

Compression Side Bolts



- Check Shear Transfer

  $V_u = 33$ kips

  PL 3/4 x 8 A572 Gr50

  2 – 3/4 in. Gr. A325-N Bolts

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

  $= (2.4)(65)(0.75)(0.75) = 87.5$ kips/bolt

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

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

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

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

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

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

Use ¼ / ¼

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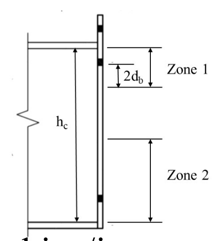
4E Moment End-Plate Design Example

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 kips / in.

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

  Min. is 3/16 in.

Use 3/16 / 3/16

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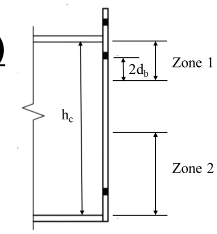
4E Moment End-Plate Design Example

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

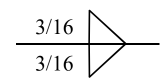


- Beam Web to End-Plate Weld
  Zone 2: $V_u = 33.0$ kips
  $h_c = h - 2t_f = 18.0 - 2(0.375) = 17.3$ in.
  Length of weld:

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

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

Min. is 3/16 in.

Use 

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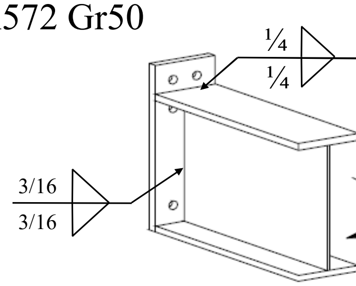
4E Moment End-Plate Design Example

4-Bolt Extended Unstiffened "Thick" Plate Design

- Final Design
  $M_u = 2{,}000$ kip-in.
  $V_u = 33.0$ kips
  End-Plate 3/4x8x1′-9½″ 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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AISC | Questions?

Smarter.
Stronger.
Steel.

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Smarter.
Stronger.
Steel.

Single-Session Registrants

CEU / PDH Certificates

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

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4-Session Registrants

CEU / PDH Certificates

One certificate will be issued at the conclusion of the course.

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4-Session Registrants

Attendance and PDH Certificates

- You have two options to receive credit for a given session.
  - Option 1: Watch the live session. Credit for live attendance will be displayed on the Course Resources table within two days of the session.
  - Option 2: Watch the recording and pass the associated quiz.

Videos and Quizzes

- For each session, find access by the end of the day, Friday, after the live air date. (An email will be sent from webinars@aisc.org.)
- Quiz scores are displayed in the Course Resources table.

Distribution of Certificates

All certificates will be issued after the course is completed. Only the registrant will receive a certificate for the course.

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4-Session Registrants

Course Resources

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

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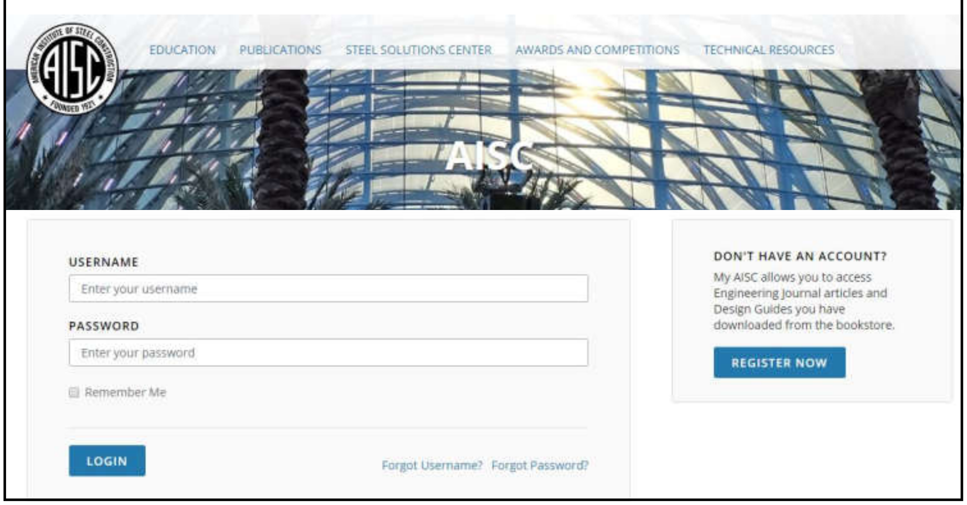
4-Session Registrants

Course Resources

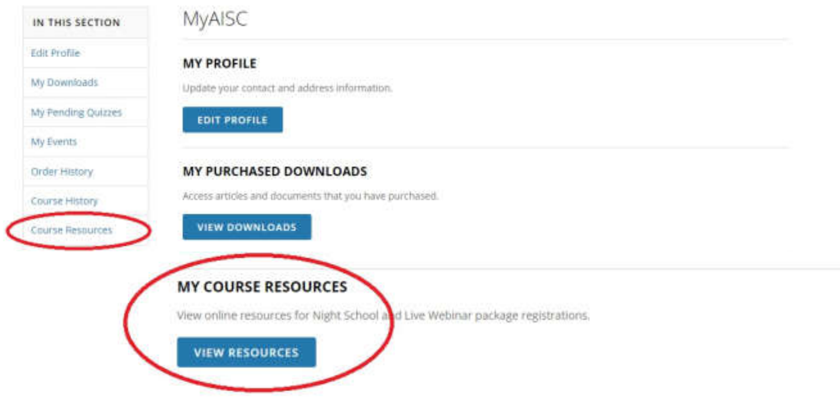
Go to www.aisc.org and sign in.



4-Session Registrants

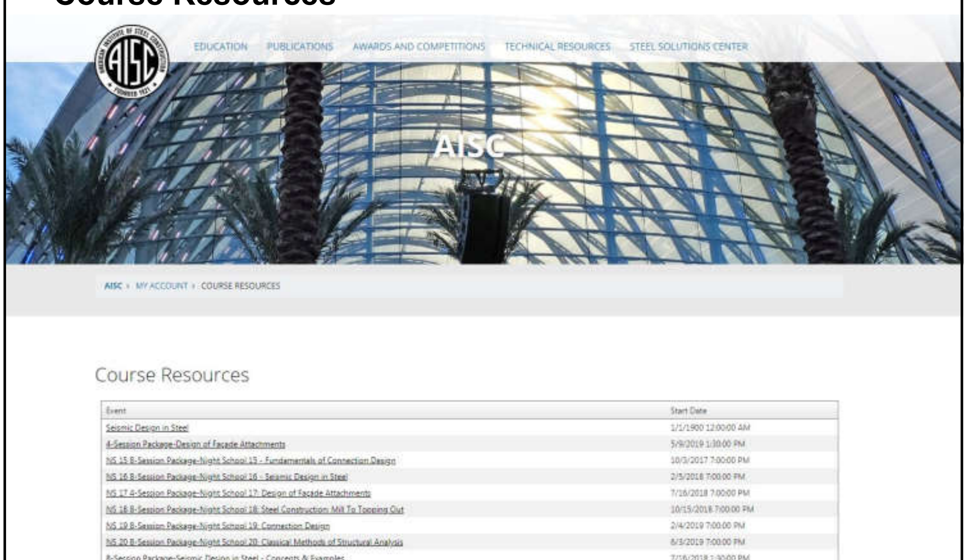
Course Resources

Go to www.aisc.org and sign in.



4-Session Registrants

Course Resources



4-Session Registrants

Course Resources

AISC > MY ACCOUNT > COURSE RESOURCES > DESIGN OF FACADE ATTACHMENTS PACKAGE RESOURCES

Design of Facade Attachments

4-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
R1: Facade Fundamentals	N/A	Handouts	View Passcode: AZNS175	Pass Score: 100	N/A
L1: Façade Attachments Part 1	May 9 2019 1:30PM EDT	Handouts	Available 05/11/2019 5:00PM EDT	Available 05/11/2019 5:00 PM EDT	Pending
L2: Façade Attachments Part 2	May 16 2019 1:30PM EDT	Handouts	Available 05/18/2019 5:00PM EDT	Available 05/18/2019 5:00 PM EDT	Pending
L3: Façade Attachments - Building Lateral Drifts	May 23 2019 1:30PM EDT	Handouts	Available 05/25/2019 5:00PM EDT	Available 05/25/2019 5:00 PM EDT	Pending
Final Exam	N/A			Available 5/27/2019 5:00 PM EDT	

AISC | Thank you.

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