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**Connection Design for Moment Frames and Braced Frames**

Session 1: Moment Connections, Part 1

February 19, 2020

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# AISC Live Webinars

## Course Description

Moment Connections, Part I
February 19, 2020

This session will address wind and low-seismic moment connection design. Common moment configurations, either with the beam flange welded directly to the column, or with flange plates connecting the beam to the column, will be presented. Local limit states for the column will be discussed and applied in a design example.

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## Learning Objectives

- Identify several types of moment connections.
- List the column-side strength limit states applicable to moment connections.
- Describe moment connections with flange plates and moment connections with beam flanges directly welded to the column.
- Describe the steps in designing a moment connection through the presentation of a design example.

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# Connection Design for Moment Frames and Braced Frames

## Session 1: Moment Connections – Part I

**February 19, 2020**

Brad Davis, PhD, SE
Associate Professor, University of Kentucky
Owner, Davis Structural Engineering

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# MOMENT CONNECTIONS PART I

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# TOPICS

Moment Connections
- Directly Welded Flange
- Flange-Plated (Welded)
- Flange-Plated (Bolted)
- Column Side Limit States
- Design Example

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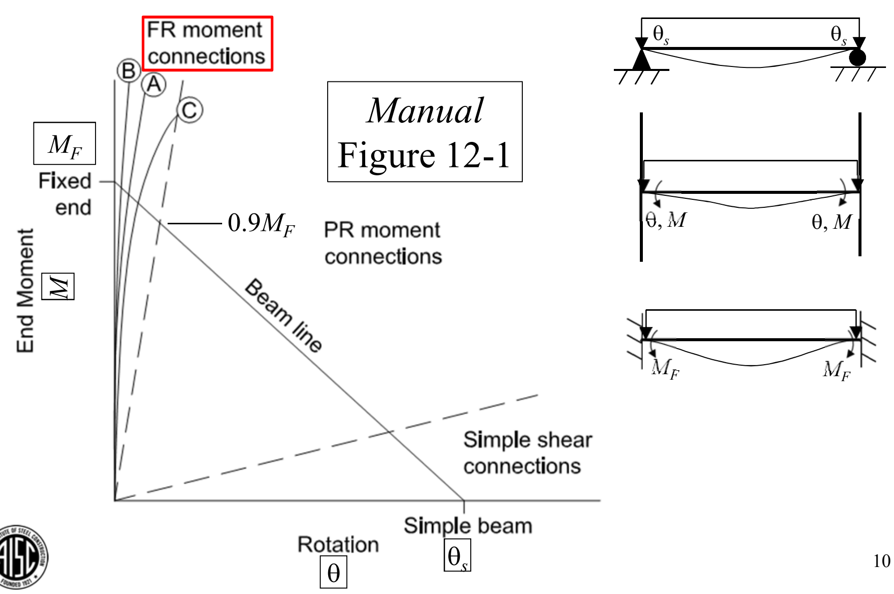
# Moment Connections

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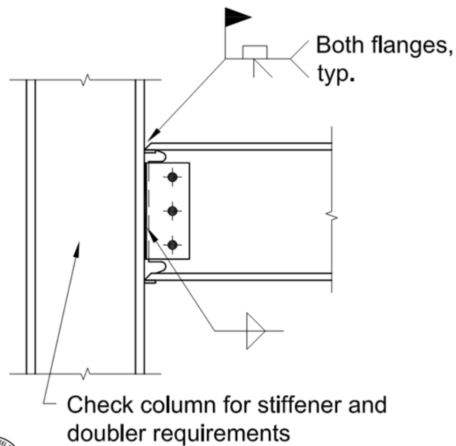
# DIRECTLY WELDED FLANGE MOMENT CONNECTIONS

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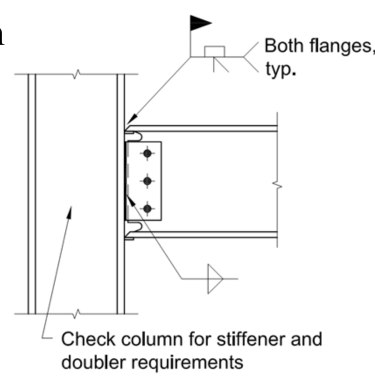
# Directly Welded Flange Connections

Limit States

- *Manual* Page 12-7
- Girder Flange-to-Column Flange Weld
  - Complete Joint Penetration
  - Fillet Welds (Shop)
- Note
  - Weld Access Holes Required

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## Directly Welded Flange Connections

- Girder Flange-to-Column Flange Weld

  PJP joint welds are not recommended.

  Fillet welds can be designed to develop the tensile yielding strength of beam flange or resist the required beam moment (*Manual* Page 8-8):

# of sixteenths

$$\phi R_{n,Flange} \le \phi R_{n,Weld}$$

$$(0.9)(F_{yf})(t_f)(1\text{ in.}) = (1.392)(1.5)(D_{req})(1\text{ in.})(2) \Rightarrow D_{req}$$

$$R_u \le \phi R_n$$

$$\frac{M_u}{d - t_f} = (1.392)(1.5)(D_{req})(b_f)(2) \Rightarrow D_{req}$$

(Recommended)

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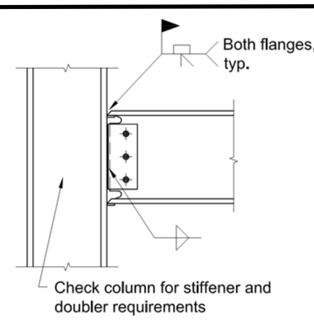
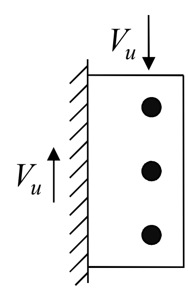
## Directly Welded Flange Connections

- Web Plate Limit States
  - Shear Transfer (minimum of bolt shear rupture, bearing, and tear-out)
  - Shear Rupture
  - Shear Yielding
  - Block Shear
  - Weld Shear Rupture

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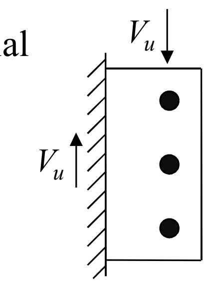
## Directly Welded Flange Connections

- Notes
  - Web plate connection is developed for direct shear (no eccentricity).
  - The moment is resisted by the flange force couple.
  - High seismic design requires special detailing.

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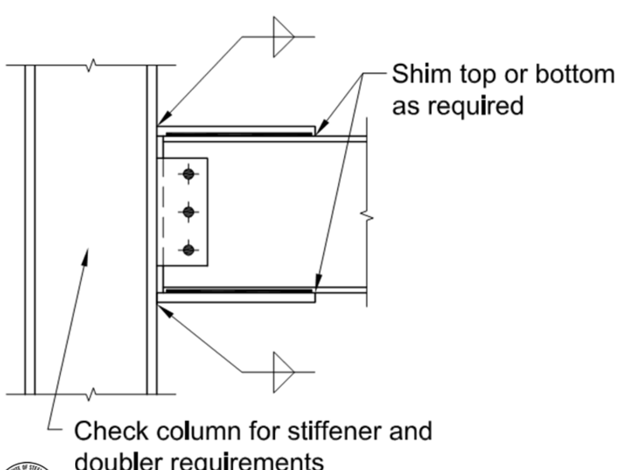
## FLANGE-PLATED (WELDED) MOMENT CONNECTIONS




*Manual* Page 12-4.

Top plate is narrower than beam flange.

Bottom plate is wider than beam flange.

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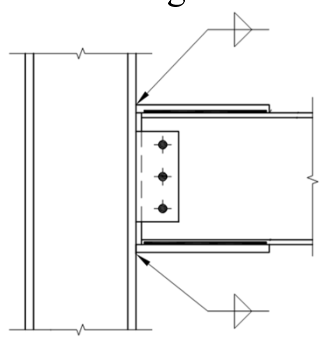
# Flange-Plated (Welded) Connections

Limit States

- Flange Plates-to-Column Flange Welds
  - Complete Joint Penetration (CJP)
  - Fillet Welds
  - Plate-to-column flange weld must be made first.





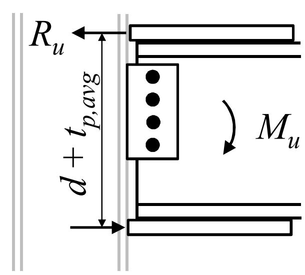
# Flange-Plated (Welded) Connections

- Tension Flange Plate Yielding

$$R_u \leq \phi R_n$$

where

$$R_u = \frac{M_u}{d + t_{p,avg}}$$




$$R_n = F_y A_g \qquad (\textit{Spec.}\ \text{Eq. J4-1})$$

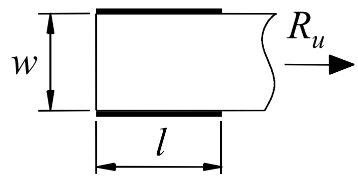
$$\phi = 0.9$$

# Flange-Plated (Welded) Connections

- Tension Flange Plate Rupture

  Longitudinal Welds Only

$R_u \le \phi R_n$




where

$\phi = 0.75$

$$R_n = F_u A_e = F_u U A_n = F_u U A_g \quad (\textit{Spec}.\ \text{Eq. J4-2})$$

$U$ = Shear Lag Factor

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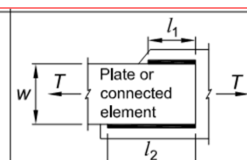
# Flange-Plated (Welded) Connections

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

$\boxed{\bar{x} = 0}$

| | | | |
|---|---|---|---|
| 4[a] | Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$. | $U = \dfrac{3l^2}{3l^2 + w^2}\left(1 - \dfrac{\bar{x}}{l}\right)$ |  |



$\boxed{l_1 = l_2 = l}$

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# Flange-Plated (Welded) Connections

- Top Flange Plate Weld

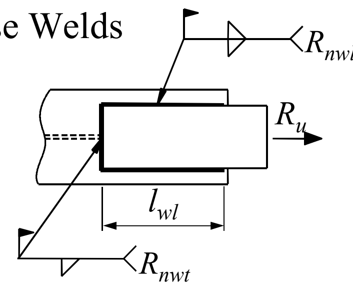
$$R_u = \frac{M_u}{d}$$

  - If Longitudinal Welds Only (*Manual* Page 8-8)

$$\phi R_n = (1.392)(D)(l_{wl})(2 \text{ welds})$$

  - If Longitudinal and Transverse Welds
    - *Specification* J2.4(b)(2)

$$R_n = \max \left| \begin{array}{l} R_{nwl} + R_{nwt} \\ 0.85 R_{nwl} + 1.5 R_{nwt} \end{array} \right.$$

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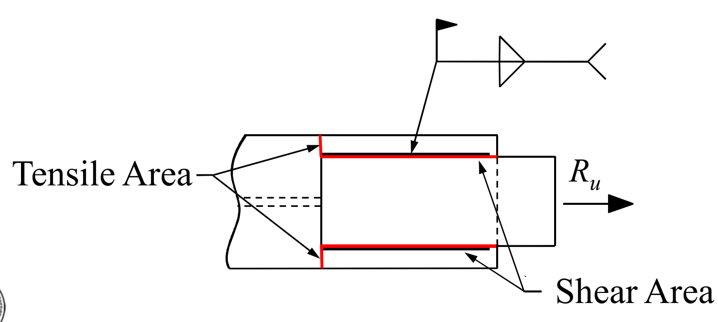
# Flange-Plated (Welded) Connections

- Beam Top Flange Block Shear
  - Applies for beam negative moment.

$$R_u = \frac{M_u}{d}$$

$\phi R_n$ calc (*Spec.* J4.3) for longitudinal welds only:

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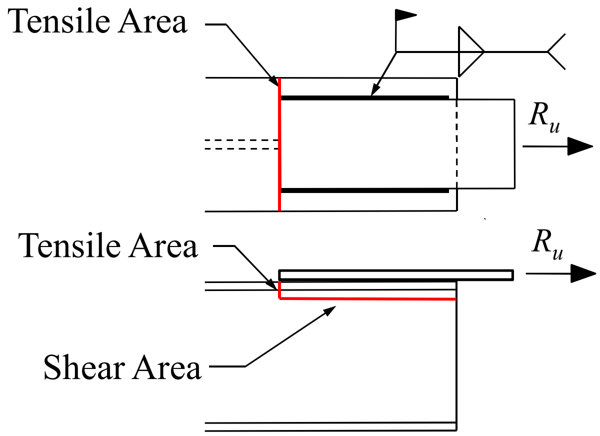
# Flange-Plated (Welded) Connections

- Beam Top Flange Block Shear

  $\phi R_n$ calc for longitudinal or longitudinal + transverse:

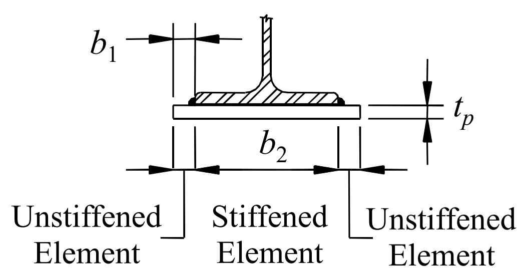
  Rarely Controls – short plates.






# Flange-Plated (Welded) Connections

- Compression Flange Plate Local Buckling
  - *Specification* Table B4.1a Axial Compression




$$\frac{b_1}{t_p} \leq 0.56\sqrt{\frac{E}{F_y}}$$

Case 1

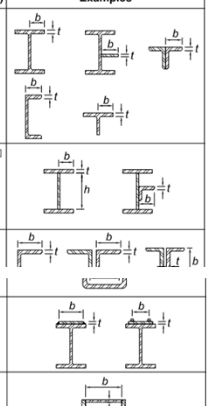
$$\frac{b_2}{t_p} \leq 1.40\sqrt{\frac{E}{F_y}}$$

Case 7

# Flange-Plated (Welded) Connections

**TABLE B4.1a**
**Width-to-Thickness Ratios: Compression Elements**
**Members Subject to Axial Compression**

| | Case | Description of Element | Width-to-Thickness Ratio | Limiting Width-to-Thickness Ratio $\lambda_r$ (nonslender/slender) | Examples |
|---|---|---|---|---|---|
| Unstiffened Elements | 1 | Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections, outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees | $b/t$ | $\boxed{0.56\sqrt{\frac{E}{F_y}}}$ | |
| | 2 | Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections | $b/t$ | $0.64\sqrt{\frac{k_c E}{F_y}}$ [a] | |
| | 3 | Legs of single angles, legs of double angles with | | | |
| Stiffened Elements | 7 | Flange cover plates and diaphragm plates between lines of fasteners or welds | $b/t$ | $\boxed{1.40\sqrt{\frac{E}{F_y}}}$ | |
| | 8 | All other stiffened elements | $b/t$ | $1.49\sqrt{\frac{E}{\dots}}$ | |



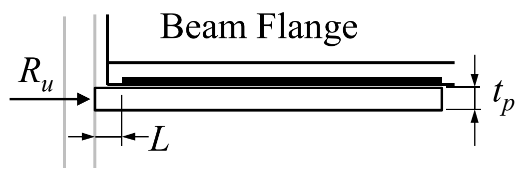
- Compression Flange Plate Local Buckling



# Flange-Plated (Welded) Connections

- Compression Plate Compression
  - Yielding or Buckling per *Specification* J4.4

$$R_u = \frac{M_u}{d + t_{p,avg}}$$




If $L_c/r \leq 25$, $\quad \phi P_n = \phi F_y A_g$

Otherwise, $\quad \phi P_n = \phi F_{cr} A_g$ (*Spec.* Ch. E)

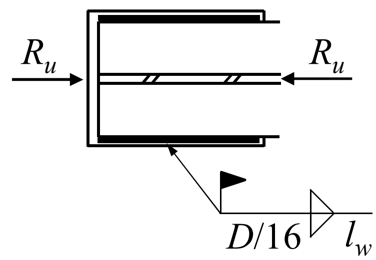
$$\phi = 0.9 \qquad L_c = KL = 0.65L \qquad r = t_p / \sqrt{12}$$

# Flange-Plated (Welded) Connections

- Bottom Flange Plate Weld
  - Longitudinal only – avoid overhead welds

$R_u = M_u / d$

$\phi R_n = (1.392 D l_w)(2)$

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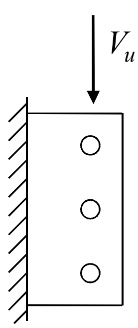
# Flange-Plated (Welded) Connections

- Web Plate / Web Bolts

No eccentricity – all moment is resisted by flange connections.

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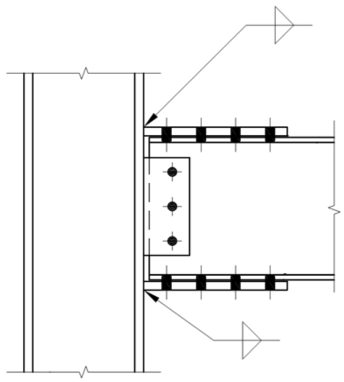
# FLANGE-PLATED (BOLTED) MOMENT CONNECTIONS



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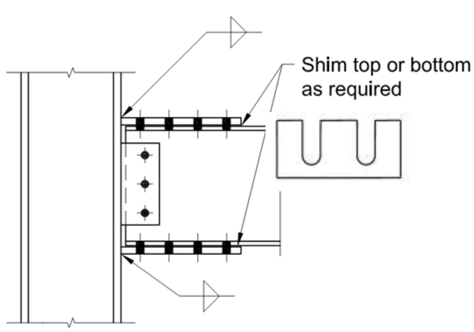
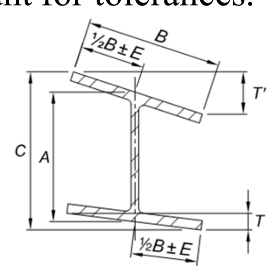
## Flange-Plated (Bolted) Connections

- Flange Plates
  - *Manual* Page 12-4.
  - Flange plates usually welded in shop.
  - Top flange plate is located 1/4 in. to 3/8 in. above tabulated beam depth to account for tolerances.




*Manual* Table 1-22

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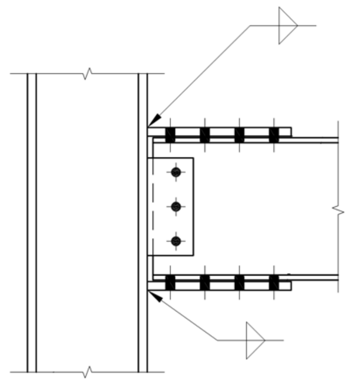
# Flange-Plated (Bolted) Connections

Limit States

- Flange Plates-to-Column Flange Welds
  - Complete Joint Penetration (CJP)
  - Fillet Welds
  - If welded in the field, welding must be done before flange bolting.



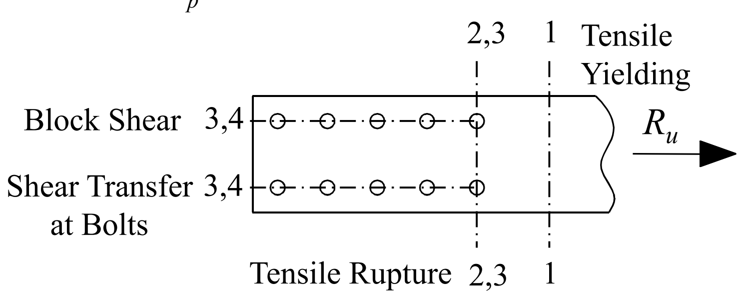


# Flange-Plated (Bolted) Connections

Limit States

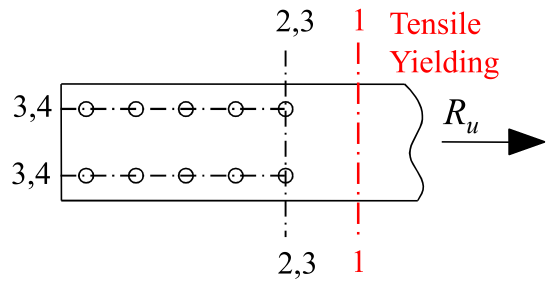
- Tension Flange Plate Limit States

$$R_u = \frac{M_u}{d + t_p}$$



## Flange-Plated (Bolted) Connections

1. Tensile Yielding



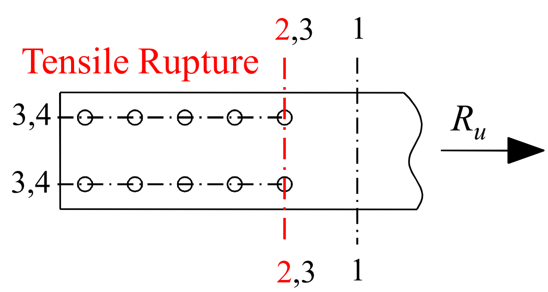

$\phi R_n = \phi F_y A_g$ (*Spec.* Eq. J4-1)

with $\phi = 0.9$



## Flange-Plated (Bolted) Connections

2. Tensile Rupture

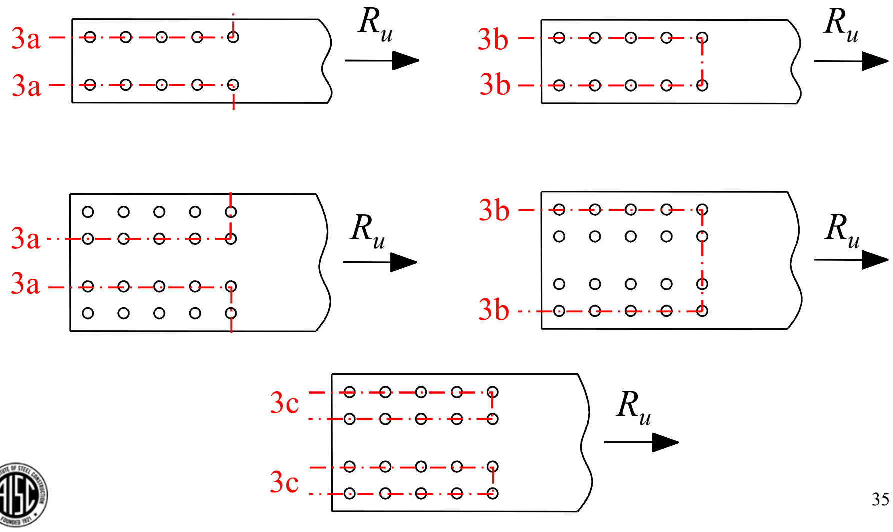



$\phi R_n = \phi F_u A_e$ (*Spec.* Eq. J4-2)

with $A_e = A_n$ and $\phi = 0.75$

# Flange-Plated (Bolted) Connections

3. Flange Plate Block Shear (*Spec.* Eq. J4-5)

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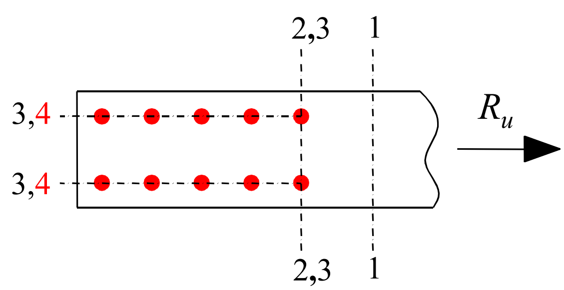
# Flange-Plated (Bolted) Connections

4. Shear Transfer Between Plate and Flange




$\phi = 0.75$

$R_n = \sum \min$(Bolt Shear Rupture, Bearing, Tearout)

*Spec.* Eq. J3-1 | Eq. J3-6a | Eq. J3-6c

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## Flange-Plated (Bolted) Connections

- Compression Plate Limit States
  5. Shear Transfer at Bolts
  6. Local and Flexural Buckling

5. Shear Transfer Between Plate and Flange
   Same method as for Tension Plate

$\phi = 0.75$

$R_n = \sum \min$(Bolt Shear Rupture, Bearing, Tearout)

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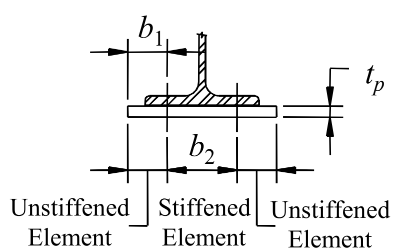
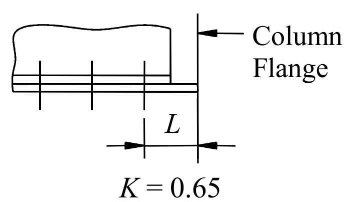
## Flange-Plated (Bolted) Connections

6. Local and Flexural Buckling




Local Buckling
(*Spec.* Section B4)




Flexural Buckling
(*Spec.* Section J4.4)

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# Flange-Plated (Bolted) Connections

- Girder/Beam Limit States
  7. Reduced Flexural Strength
  8. Block Shear

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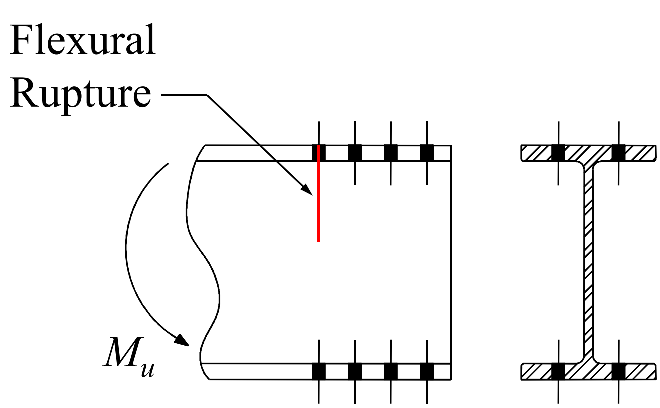
# Flange-Plated (Bolted) Connections

7. Reduced Flexural Strength (*Specification* F13.1)

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## Flange-Plated (Bolted) Connections

7. Reduced Flexural Strength (*Spec.* F13.1)

If $F_u A_{fn} < Y_t F_y A_{fg}$

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \qquad (Spec.\ \text{Eq. F13-1})$$

If $F_u A_{fn} \geq Y_t F_y A_{fg}$ flexural rupture does not apply.

where $Y_t = 1.0$ for $F_y/F_u \leq 0.8$ (A992: 50/65 = 0.769)
$= 1.1$ otherwise
$A_{fn}$ = net area of flange
$A_{fg}$ = gross area of flange

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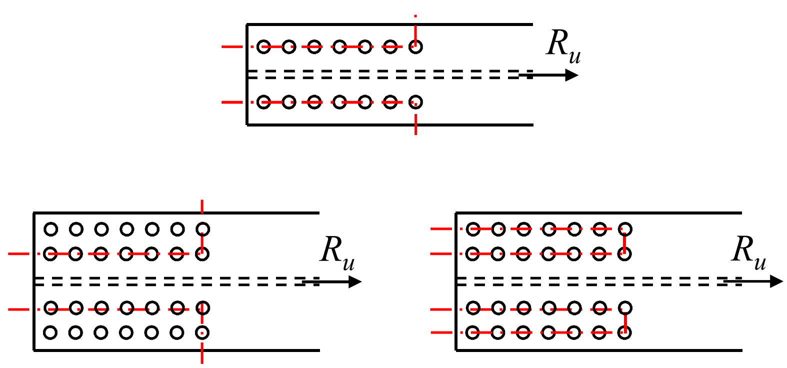
## Flange-Plated (Bolted) Connections

8. Beam Flange Block Shear

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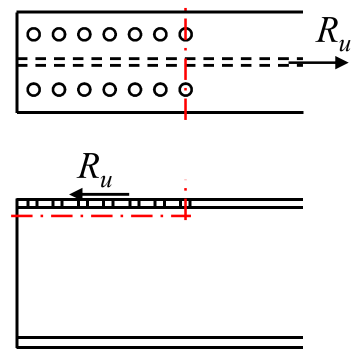
# Flange-Plated (Bolted) Connections

8. Beam Flange Block Shear

Flange / Web Pattern

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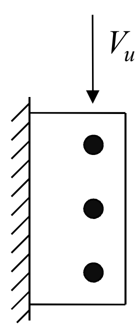
# Flange-Plated (Bolted) Connections

- Web Plate / Web Bolts

  Same as for Flange-Plated (Welded) / Web Bolted Connection – No Eccentricity

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# COLUMN SIDE LIMIT STATES AT MOMENT CONNECTIONS

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## Column Side Limit States

Flange and Flange-Plated Connections

*Specification* J10 and AISC Design Guide 13

1. Flange Local Bending (*Spec*. J10.1)
2. Web Local Yielding (*Spec*. J10.2)
3. Web Local Crippling (*Spec*. J10.3)
4. Web Compression Buckling (*Spec*. J10.5)
5. Transverse Stiffener Design (*Spec*. J10.8)
6. Web Panel Zone Shear (*Spec*. J10.6)

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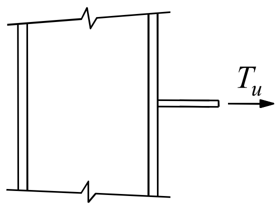
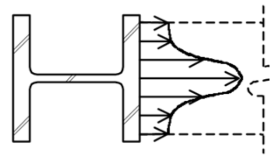
# Column Side Limit States

1. Flange Local Bending (*Spec.* J10.1)



DG13 Figure 2-5

$T_u \leq \phi R_n$
$\phi = 0.9$
$R_n = 6.25 F_{yf} t_f^2$

If $T_u > \phi R_n$ then half-depth web stiffeners required.

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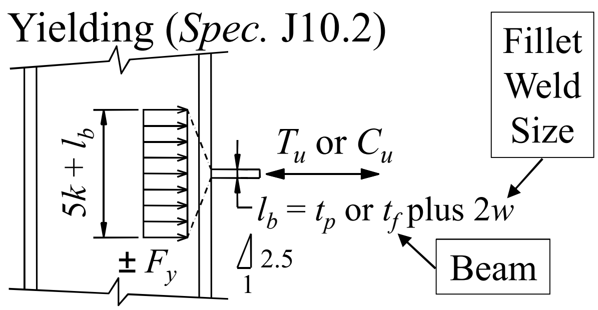
# Column Side Limit States

2. Web Local Yielding (*Spec.* J10.2)




When load is away from end of member,

$$T_u \text{ or } C_u \leq \phi R_n = \phi F_{yw} t_w (5k + l_b), \text{ where } \phi = 1.0$$

If $T_u$ or $C_u > \phi R_n$ then half-depth stiffeners or doubler plate(s) required.

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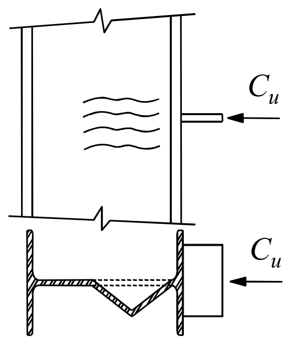
## Column Side Limit States

3. Web Local Crippling (*Spec.* J10.3)

Same design rules as for beam bearing with

$l_b = t_p$ or beam $t_f$ plus $2w$ if applicable




If $C_u > \phi R_n$ then 3/4-depth web stiffeners or doubler plate(s) required.

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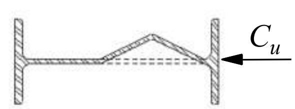
## Column Side Limit States

3. Web Local Crippling (*Spec.* J10.3)

When load is away from end of column:

$\phi = 0.75$




$$R_n = 0.8 t_w^2 \left[ 1 + 3 \left( \frac{l_b}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f$$

$Q_f = 1.0$ for wide flange sections

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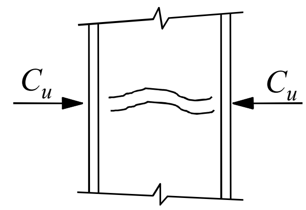
# Column Side Limit States

4. Web Compression Buckling (*Spec.* J10.5)

$C_u \leq \phi R_n \qquad \phi = 0.9$

When load is away from end of member,

$$R_n = \frac{24 t_w^3 \sqrt{E F_{yw}}}{h} Q_f$$




$h$ = clear distance between fillets

$Q_f$ = 1.0 for wide flange sections

If max $C_u > \phi R_n$ then full depth stiffeners or doubler plate(s) required.

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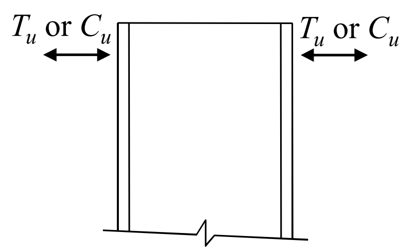
# Column Side Limit States

1-4. Connections at Top of Column

Column side strengths are approximately one-half of above. See *Specification* Sections J10.1, J10.2, J10.3, and J10.5.

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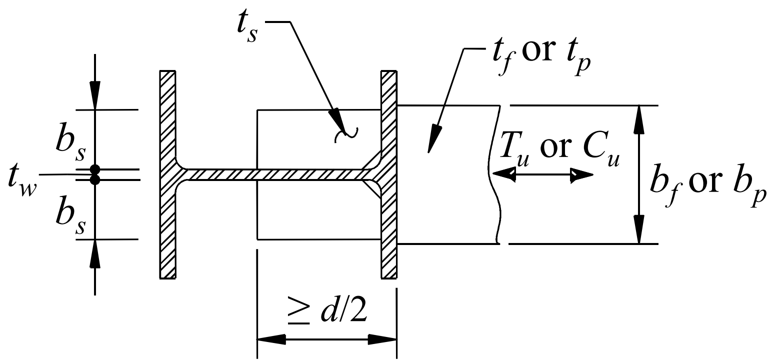
## Column Side Limit States

5. Transverse Stiffener Design

(*Spec.* J10.8 Requirements)






## Column Side Limit States

5. Transverse Stiffener Design

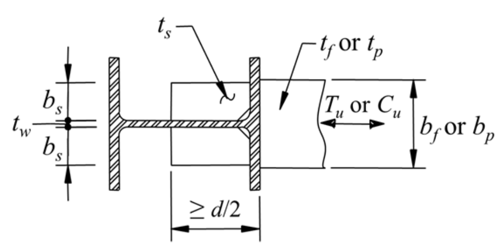
$$b_s + \frac{t_w}{2} \geq \frac{b_f \text{ or } b_p}{3}$$

$$t_s \geq \frac{t_f \text{ or } t_p}{2}$$

$$\frac{b_s}{t_s} \leq 0.56\sqrt{\frac{E}{F_y}}$$

Local Buckling Limit State
(Compression only)

$$t_s \geq \frac{b_s}{16}$$



# Column Side Limit States

5. Transverse Stiffener Design
Force Distribution Design Model (*Spec.* J10.8 and DG13):

Column resists $\phi R_n$ for each limit state.

Stiffeners resist the load above $\phi R_n$.

$$\begin{aligned}\text{Stiffener Force} &= T_{u,net} \text{ or } C_{u,net}\\ &= (T_u \text{ or } C_u) - \min \phi R_n\\ &= R_u - \min \phi R_n\end{aligned}$$

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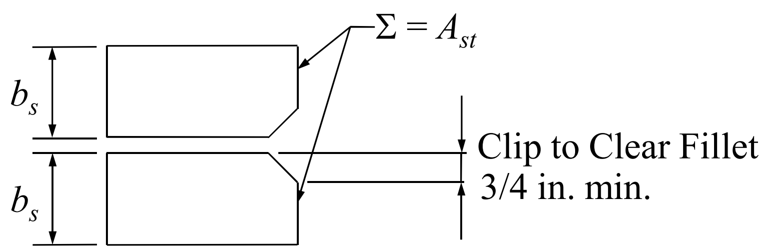
# Column Side Limit States

5. Transverse Stiffener Design
Required Net Stiffener Area (DG13)

$$T_{u,net} \text{ or } C_{u,net} \le \phi R_n$$

$$T_{u,net} \text{ or } C_{u,net} = \phi F_y A_{st,req} \quad \Longrightarrow \quad A_{st,req}$$

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## Column Side Limit States

5. Transverse Stiffener Design

Stiffener-to-Flange Weld

Recommend that weld develop the stiffener contact area (tensile yielding) at the column flange. For fillet welds:

$$\phi R_{n,st} \leq \phi R_{n,weld}$$

$$\phi F_y(b_s - clip)(t_s) = (1.392)(1.5)(D_{req})(b_s - clip)(2 \text{ welds})$$

$$D_{req} = \frac{(0.9)(F_y)(t_s)}{(1.392)(1.5)(2)} = 0.216 F_y t_s$$

If necessary: Complete Joint Penetration (CJP) Weld

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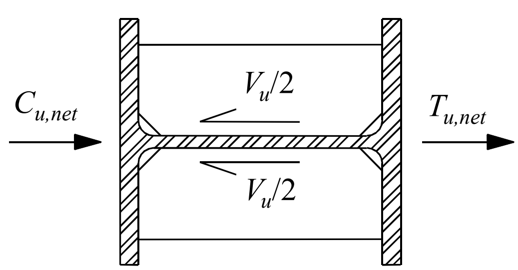
## Column Side Limit States

5. Transverse Stiffener Design

Stiffener-to-Web Weld (*Spec*. J10.8)




$$V_u = C_{u,net} + T_{u,net}$$

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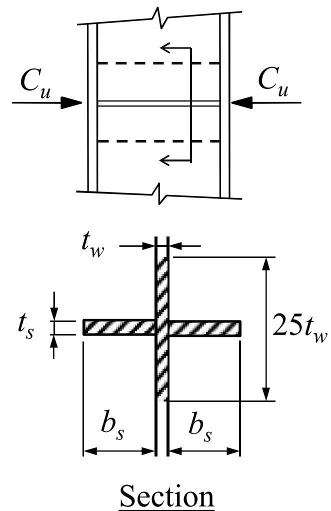
# Column Side Limit States

5. Transverse Stiffener Design

*Specification* J10.8

When full-depth stiffeners are required.

Design as a compression member with cruciform cross section (height of web is $25t_w$ when away from the top of the column).

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# Column Side Limit States

5. Transverse Stiffener Design

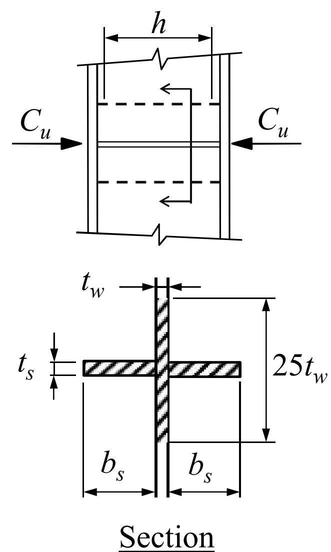
max $C_u \leq \phi R_n$

$R_n$ in compression per *Spec.* J4.4, with:

$A$ and $r$ for cruciform shape.

$L_c = KL = 0.75h$ (*Spec*. J10.8)

$L_c/r \rightarrow \phi R_n$

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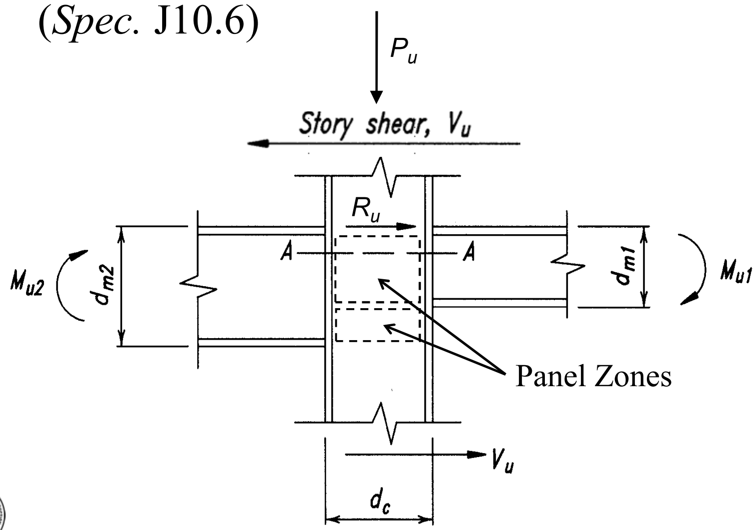
## Column Side Limit States

6. Web Panel-Zone Shear
   (*Spec.* J10.6)

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## Column Side Limit States

6. Web Panel-Zone Shear
   At Section A-A

$$R_u \le \phi R_n$$

where

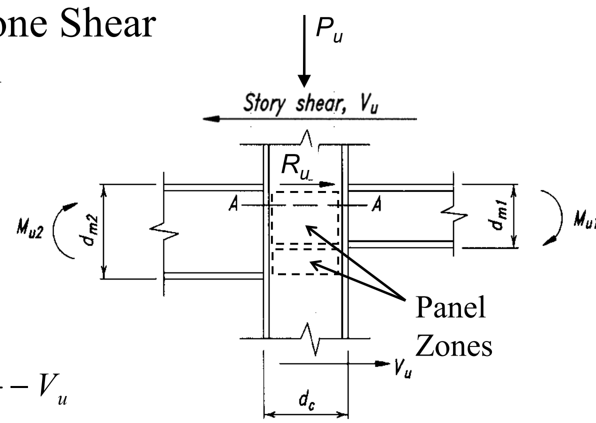
$\phi = 0.9$

$$R_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u$$

$$d_{m1} = d_1 - t_{f1}$$

$$d_{m2} = d_2 - t_{f2}$$

(Directly Welded Flanges)

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## Column Side Limit States

6. Web Panel-Zone Shear

When effect of panel zone deformation on frame stability is not considered (*Specification* Section J10.6(a)):

For $\alpha P_r \leq 0.4P_y = 0.4F_yA_g$

$$R_n = 0.60F_yd_ct_w$$

For $\alpha P_r > 0.4P_y = 0.4F_yA_g$

$$R_n = 0.60F_yd_ct_w(1.4 - \alpha P_r / P_y)$$

where $\alpha P_r = P_u$ for LRFD

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## Column Side Limit States

6. Web Panel-Zone Shear

See *Specification* Section J10.6(b) for strength when the effect of panel zone deformation on frame stability is considered in the structural analysis.

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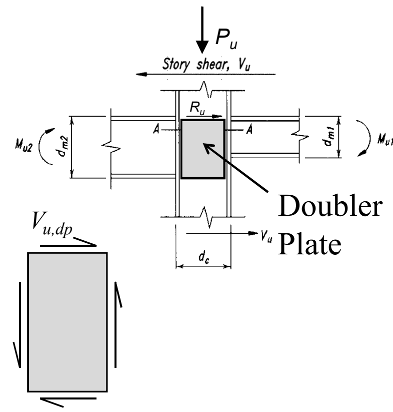
# Column Side Limit States

6. Web Panel-Zone Shear

Doubler Plate Design
*Spec.* Section J10.9

If $R_u > \phi R_n$ then
a doubler plate is required.

$$V_{u,dp} = R_u - \phi R_{n,col}$$




Note: Two plates are recommended if required doubler thickness exceeds 1 in.

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# Column Side Limit States

6. Web Panel-Zone Shear

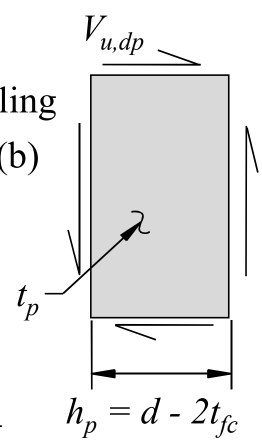
Doubler Plate Shear Yielding or Buckling
*Spec.* J10.9 → Ch. G; Use *Spec.* G2.1(b)

$$V_{u,dp} \le \phi V_n \qquad \phi = 0.9$$

$$V_n = 0.6 F_{yp} t_p d_c C_{v1}$$

$$\text{If } \frac{h_p}{t_p} \le 1.10\sqrt{\frac{k_v E}{F_{yp}}}, \ C_{v1} = 1.0$$

$$\text{If } \frac{h_p}{t_p} > 1.10\sqrt{\frac{k_v E}{F_{yp}}}, \ C_{v1} = \frac{1.10\sqrt{\frac{k_v E}{F_{yp}}}}{h_p / t_p}$$

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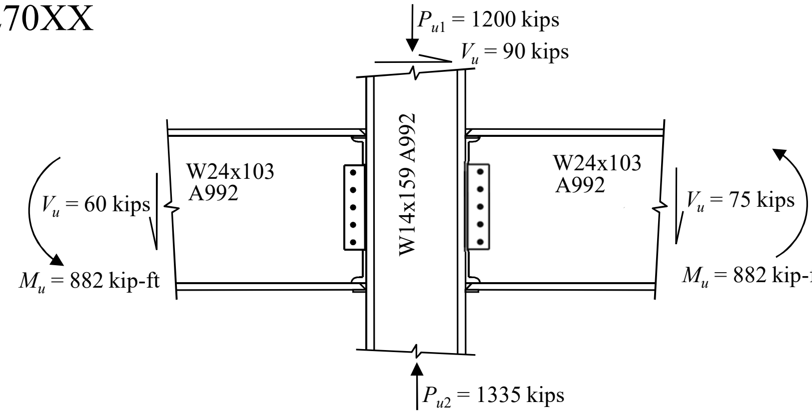
# Example: Design Directly Welded Flange Moment Connection

3/4 in. Gr. A325-N Bolts
E70XX

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# Example: Directly Welded Flange Connection

| | W24x103 | W14x159 |
|---|---|---|
| $d$ | 24.5 in. | 15.0 in. |
| $t_w$ | 0.550 in. | 0.745 in. |
| $b_f$ | 9.00 in. | 15.6 in. |
| $t_f$ | 0.980 in. | 1.19 in. |
| $k$ | | 1.79 / 2-1/2 in. |
| $h/t_w$ | | 15.3 |
| $A$ | | 46.7 in.$^2$ |
| $T$ | | 10 in. |

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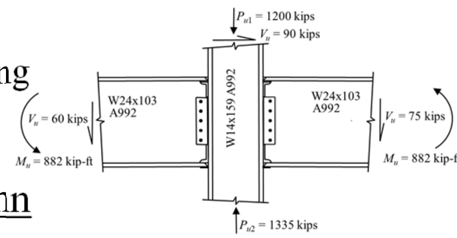
## Example: Directly Welded Flange Connection

W24x103 Flange-to-Column Flange Weld

CJP welds with backing bars.




W24x103 Web-to-Column Connection

Single Plate

No eccentricity considered.

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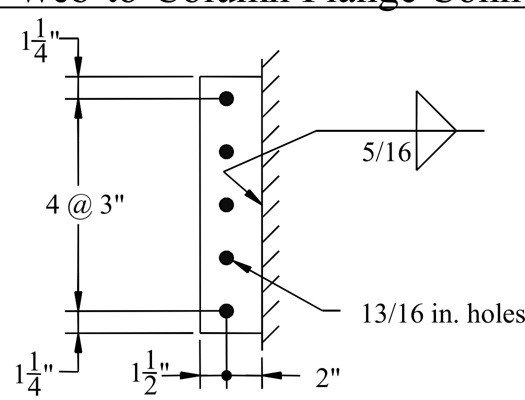
## Example: Directly Welded Flange Connection

W24x103 Web-to-Column Flange Connection




Try PL 5/16 x 3½ x 1′-2½″ A36 with
5 - 3/4 in. Gr. A325-N Bolts

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## Example: Directly Welded Flange Connection

W24x103 Web-to-Column Flange Connection

$V_u$ = 75 kips

Limit State Strengths

| | | |
|---|---|---|
| Shear Yielding: | $\phi V_n$ = 97.9 kips | OK |
| Shear Rupture: | $\phi V_n$ = 82.6 kips | OK |
| Block Shear: | $\phi V_n$ = 81.5 kips | OK |
| Shear Transfer: | $\phi V_n$ = 85.4 kips | OK |
| Weld Rupture: | $\phi V_n$ = 202 kips | OK |

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## Example: Directly Welded Flange Connection

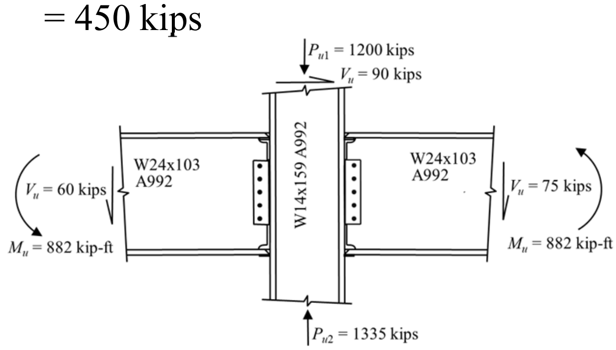
Beam Flange Forces

W24x103 $d$ = 24.5 in. $t_f$ = 0.980 in.

$M_u$ = 882 kip-ft

$T_u$ or $C_u$ = (882 x 12) / (24.5 – 0.980)

= 450 kips

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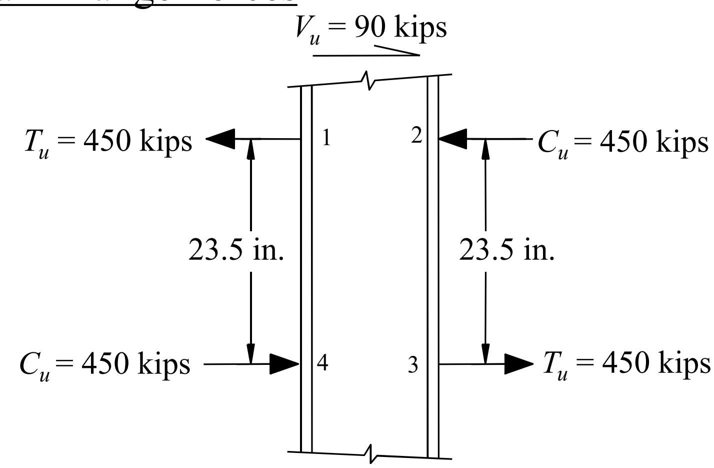
## Example: Directly Welded Flange Connection

Beam Flange Forces

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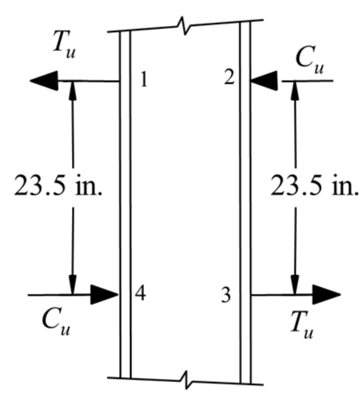
## Example: Directly Welded Flange Connection

Column Flange Local Bending (*Spec.* J10.1)

$$\phi R_n = \phi(6.25 F_{yw} t_f^2)$$
$$= 0.9(6.25)(50)(1.19^2)$$
$$= 398 \text{ kips } < T_u = 450 \text{ kips}$$

For this loading, half-depth stiffeners required at tension locations 1 and 3.

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## Example: Directly Welded Flange Connection

Column Web Local Yielding (*Spec.* J10.2)

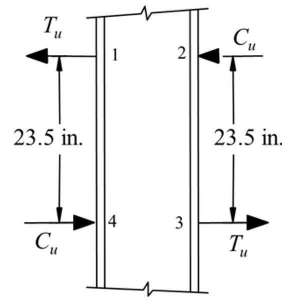
$\phi R_n = \phi\, F_{yw} t_w (5k + t_{fb})$

$= (1.0)(50)(0.745)[(5)(1.79) + 0.980]$

$= 370 \text{ kips} < T_u \text{ and } C_u = 450 \text{ kips}$

Half-Depth Stiffeners Required at all locations.

Use Full-Depth Stiffeners Top and Bottom.

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## Example: Directly Welded Flange Connection

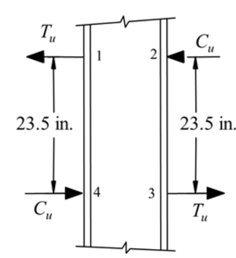
Column Web Local Crippling (*Spec.* J10.3)

With $l_b = t_f = 0.980$ in.,

$$\phi R_n = \phi 0.8 t_w^2 \left[ 1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f$$

$= 556$ kips

$C_u = 450 \text{ kips} < \phi R_n = 556 \text{ kips}$ OK

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## Example: Directly Welded Flange Connection

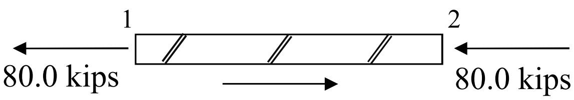
Transverse Stiffener 1-2 Design

$$T_{u1} = 450 \text{ kips} - \min \begin{vmatrix} 398 \text{ kips (FLB)} \\ 370 \text{ kips (WLY)} \end{vmatrix}$$

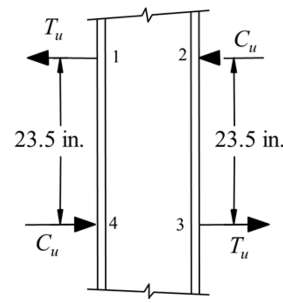
$$= 450 \text{ kips} - 370 \text{ kips} = 80.0 \text{ kips}$$

$$C_{u2} = 450 \text{ kips} - 370 \text{ kips}$$

$$= 80.0 \text{ kips}$$




Web Weld $V_u$ = 80.0 + 80.0
= 160 kips

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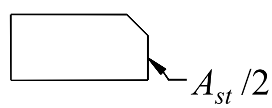
## Example: Directly Welded Flange Connection

Transverse Stiffener 1-2 Design




Same on Tension and Compression Sides

80.0 kips = (0.9)(36)($A_{st,req}$) → $A_{st,req}$ = 2.46 in.$^2$

Try PL 1/2 x 6

Clip: W14x159 $k_{det} - t_f$ = 2-1/2 - 1.19=1.31 in.

Use 1-1/2 in. clip

$A_{st}$ = 2 (6 – 1-1/2) (1/2) = 4.50 in.$^2$ ≥ $A_{st,req}$ OK

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## Example: Directly Welded Flange Connection

Transverse Stiffener 1-2 Design

Minimum Thickness

$t_s = 0.50$ in. $\geq t_{fb} / 2 = 0.980 / 2 = 0.490$ in. OK

Minimum Width

$$b_s + t_w/2 \geq b_f / 3$$

$$6 + 0.745/2 = 6.37 \text{ in.} > 9.00 / 3 = 3 \text{ in.} \quad \text{OK}$$

Local Buckling

$$\frac{b_s}{t_s} = \frac{6}{1/2} = 12 < 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29000}{36}} = 15.9, \text{ OK}$$

Use 2 PL 1/2 x 6, A36, Top and Bottom

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## Example: Directly Welded Flange Connection

Transverse Stiffener 1-2 Design

Stiffener-to-Column Flange Fillet Welds to Develop Plate Yield Strength at Weld

$$\phi R_{n,st} \leq \phi R_{n,weld}$$

$$\begin{aligned} D_{req} &= 0.216 F_y t_s \\ &= 0.216(36)(1/2) \\ &= 3.89 \text{ sixteenths} \end{aligned}$$

Minimum Weld Size = 1/4 in.

Use 1/4 in. double-sided fillet welds.

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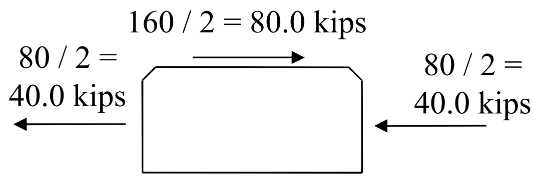
# Example: Directly Welded Flange Connection

## Transverse Stiffener 1-2 Design

Stiffener-to-Column Web Welds




$T = 10.0$ in. $> d_c\text{-}2t_{fc}–2clip = 15.0\text{-}2(1.19)\text{-}2(1.5)$
$= 9.62$ in. Use 9-1/2 in.

$$80 \text{ kips} = (1.392)(D_{req})(9.5 \text{ in.})(2 \text{ welds})$$

$$D_{req} = 3.02 \text{ sixteenths}$$

Minimum weld 1/4 in. Use 1/4 D.S. Welds

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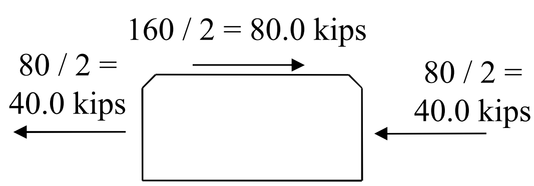
# Example: Directly Welded Flange Connection

## Transverse Stiffener 1-2 Design

Base Metal Strength of Column Web




$$\phi V_n = \phi(0.6F_u)(t_w L_w)$$
$$= (0.75)(0.6)(65)(0.745)(9.5)(2 \text{ shear planes})$$
$$= 414 \text{ kips} > (2)(80.0) = 160 \text{ kips} \quad \underline{\text{OK}}$$

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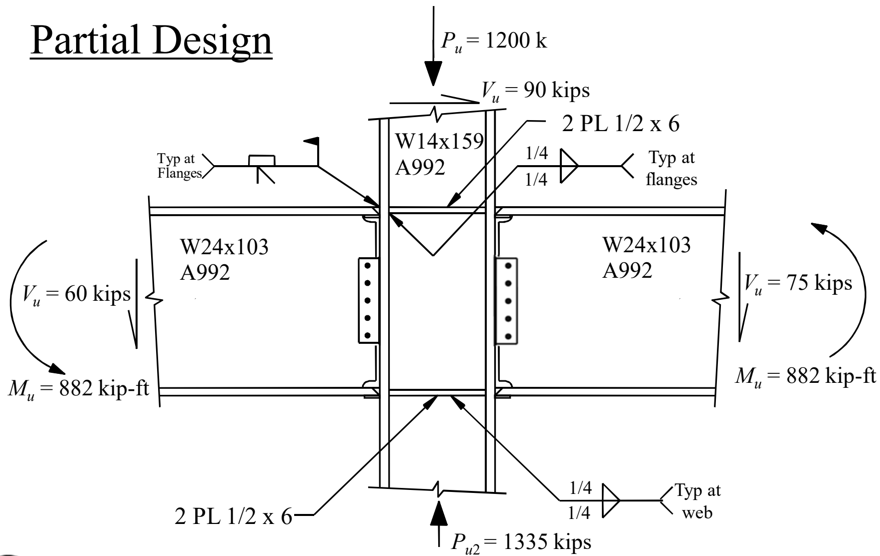
## Example: Directly Welded Flange Connection

### Partial Design

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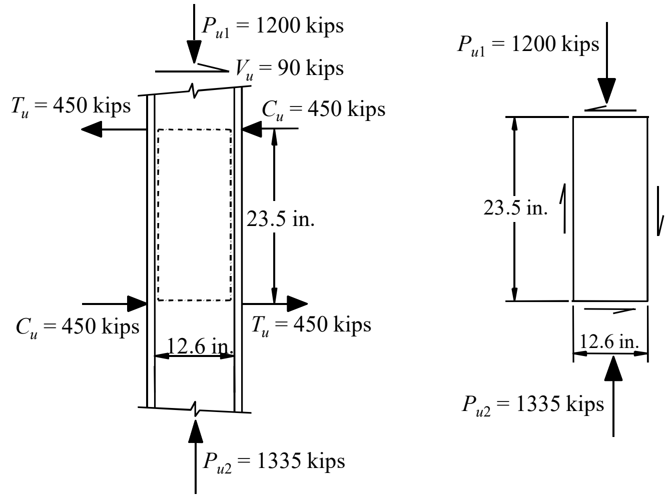
## Example: Directly Welded Flange Connection

### Panel-Zone Shear Strength

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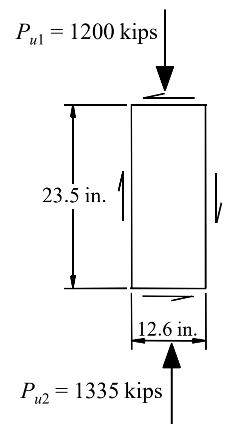
## Example: Directly Welded Flange Connection

Panel-Zone Shear Strength

$V_u = 450 + 450 - 90.0$
$= 810$ kips

$\alpha P_r = (1{,}200 + 1{,}335) / 2$
$= 1{,}270$ kips

$0.4P_y = 0.4\ F_y\ A_g$
$= (0.4)(50)(46.7)$
$= (0.4)(2335)$
$= 934 \text{ kips} < 1{,}270 \text{ kips}$

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## Example: Directly Welded Flange Connection

Panel-Zone Shear Strength

Shear Strength of Column Web

Since $\alpha P_r > 0.4P_y = 934$ kips

$\phi V_n = \phi 0.6 F_y d_c t_w (1.4 - \alpha P_r / P_y)$
$= 0.9(0.6)(50)(15.0)(0.745)(1.4 - 1270 / 2335)$
$= 259 \text{ kips} < V_u = 810 \text{ kips}$

Doubler Plate(s) Required

Doubler Plate (assume yielding controls)

Large Required Strength → Try Gr. 50 plate.

$\phi V_n = \phi 0.6 F_y d_c t_p$
$= (0.9)(0.6)(50)(15.0)(t_{p,req}) = 810 \text{ kips} - 259 \text{ kips}$
$t_{p,req} = 1.36$ in.

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# Example: Directly Welded Flange Connection

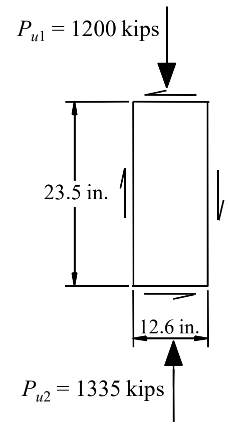
Panel-Zone Shear Strength

Try two 3/4 in. Web Doubler Plates

$h_p = d_c - 2t_f$

$= 15.0 - (2)(1.19)$

$= 12.6$ in.




Doubler Plate Slenderness:

$$\frac{h_p}{t_p} = \frac{12.6}{3/4} = 16.8 \leq 1.10\sqrt{\frac{k_v E}{F_y}} = 61.2$$

Shear yielding controls as assumed.

Use 2 – 3/4 in. Gr. 50 Doubler Plates

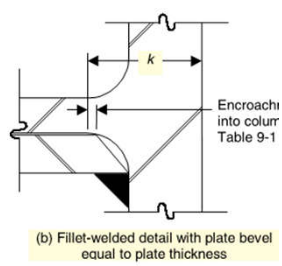


# Example: Directly Welded Flange Connection

Panel-Zone Welds

Long Side, Vertical Welds

Try fillet welds to develop the required doubler plate thickness (DG13 Eq. 4.4-7 approach).




DG13 Figure 4-13(b)

$$\phi 0.6 F_y t_{p,req}(1 \text{ in.}) = 1.392(D_{req})(1 \text{ in.})$$

$$(0.9)(0.6)(50)(1.36 \text{ in.}/2) = 1.392 D_{req}$$

$$D_{req} = 13.2 \text{ sixteenths}$$

Large, so use Doubler Plate Weld per AWS D1.8, Clause 4.3 (Develops the Doubler Plate)

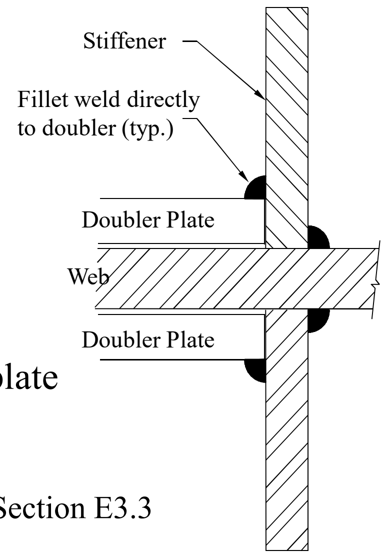
# Example: Directly Welded Flange Connection

## Panel Zone Welds

### Short Side, Horizontal Welds

Use DG13 Figure 4-12(d).

Weld transfers half of the stiffener force directly to doubler




More information on doubler plate welding:

DG13
AISC 341-16 Commentary Section E3.3
DG21 Section 4.3.3

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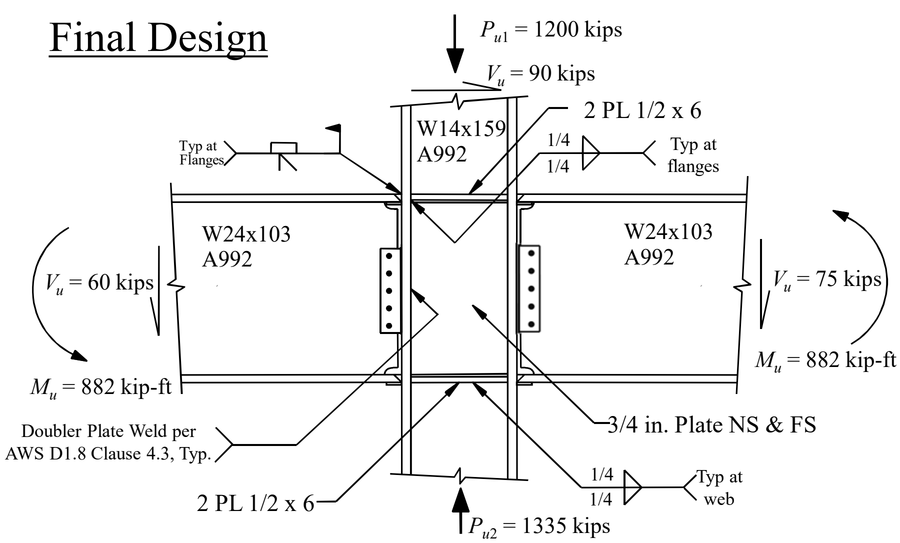
# Example: Directly Welded Flange Connection

## Final Design

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End of Session 1
Thank You for
Attending

Next Up

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## Next Session

- February 26, 2020 Moment Connections Part II

TOPICS

- Tee Stub Moment Connections
- End-Plate Moment Connections

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**AISC** | Questions?

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# Single-Session Registrants

## CEU / PDH Certificates

- 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!

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## 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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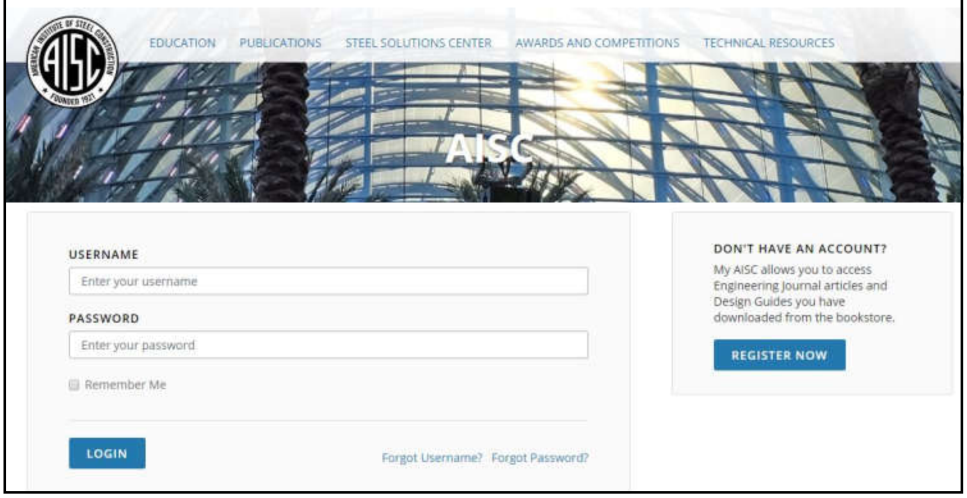
# 4-Session Registrants

## Course Resources

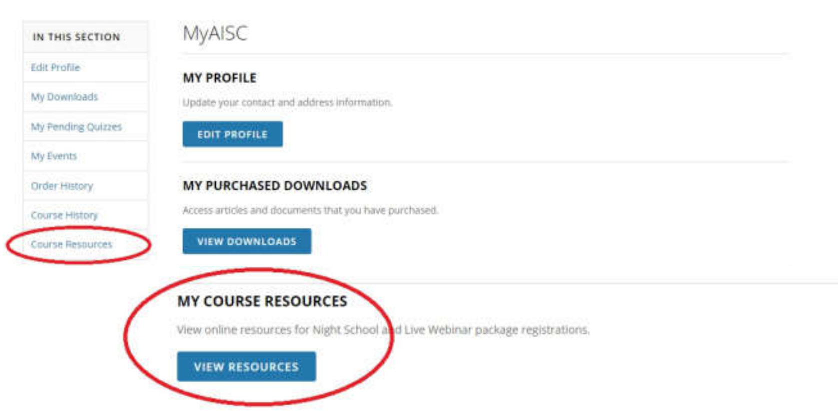
Go to www.aisc.org and sign in.




# 4-Session Registrants

## Course Resources

Go to www.aisc.org and sign in.



## 4-Session Registrants

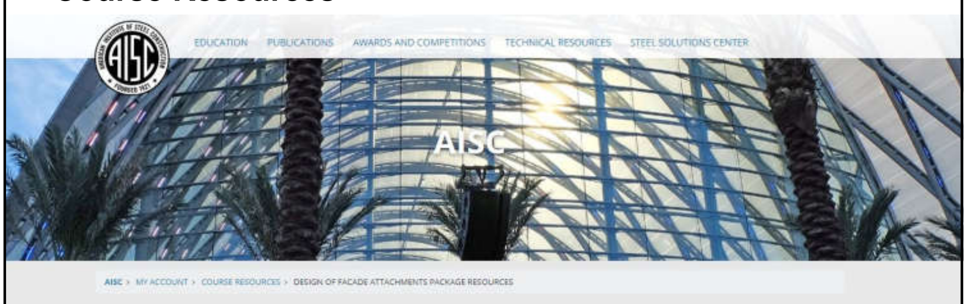
### Course Resources


Course Resources

| Event | Start Date |
|---|---|
| Seismic Design in Steel | 1/1/1900 12:00:00 AM |
| 4-Session Package-Design of Facade Attachments | 5/9/2019 1:30:00 PM |
| NS 15 8-Session Package-Night School 15 - Fundamentals of Connection Design | 10/3/2017 7:00:00 PM |
| NS 16 8-Session Package-Night School 16 - Seismic Design in Steel | 2/5/2018 7:00:00 PM |
| NS 17 4-Session Package-Night School 17: Design of Façade Attachments | 7/16/2018 7:00:00 PM |
| NS 18 8-Session Package-Night School 18: Steel Construction: Mill To Topping Out | 10/15/2018 7:00:00 PM |
| NS 19 8-Session Package-Night School 19: Connection Design | 2/4/2019 7:00:00 PM |
| NS 20 8-Session Package-Night School 20: Classical Methods of Structural Analysis | 6/3/2019 7:00:00 PM |
| 8-Session Package-Seismic Design in Steel - Concepts & Examples | 7/16/2018 1:30:00 PM |

## 4-Session Registrants

### Course Resources




Design of Facade Attachments

**4-SESSION PACKAGE RESOURCES**

| Event | Date | Handouts | Video | Quiz | Attendance |
|---|---|---|---|---|---|
| R1: Facade Fundamentals | N/A | Handouts | View<br>Passcode: AZNS175 | Pass<br>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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