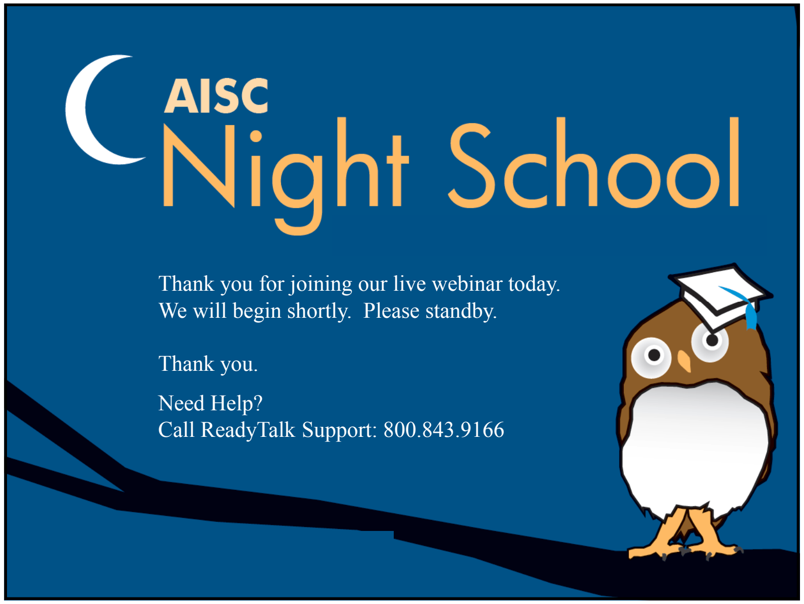

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Course Description

Session 5: November 7, 2017 – Moment Connections Part I

This live webinar covers wind and low seismic moment connection design. Various moment connections will be discussed including flange welded-web bolted connections, flange plate welded-web bolted connections, flange plate bolted-web bolted connections, and moment end-plate connections. Column side limit states will be discussed followed by the presentation of a design example.

Learning Objectives

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

- List various types of moment connections.
- List column side limit states in moment connections.
- Compare and contrast flange welded/web bolted vs. flange plate welded/web bolted vs flange plate bolted/web bolted moment connections.
- Describe the steps in designing a moment connection through the presentation of a design example.

There's always a solution in steel.

Fundamentals of Connection Design

Session 5: Moment Connections, Part I

November 7, 2017

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

structural STEEL

MOMENT CONNECTIONS PART I



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TOPICS

Moment Connections:

- Flange Welded / Web Bolted
- Flange Plate Welded / Web Bolted
- Flange Plate Bolted / Web Bolted
- Column Side Limit States
- Design Example

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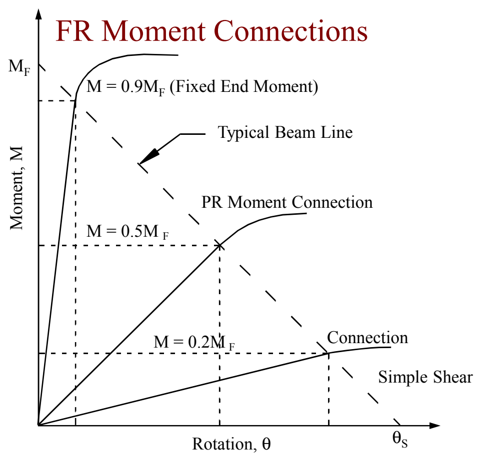
Moment Connections

FR Moment Connections



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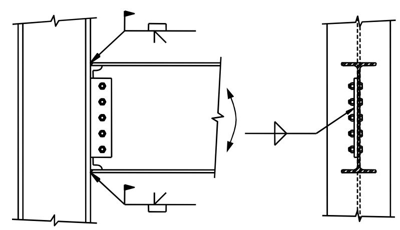
FLANGE WELDED / WEB BOLTED MOMENT CONNECTIONS

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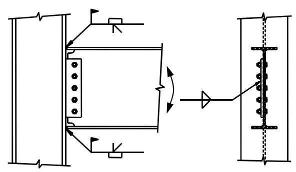
Flange Welded / Web Bolted

Limit States

- Girder Flange-to-Column Flange Weld
  - Complete Joint Penetration (CJP)
  - Partial Joint Penetration (PJP)
  - Fillet Welds (Shop)
- Note
  - Weld Access Holes Required for Field Welds



Flange Welded / Web Bolted

- Girder Flange-to-Column Flange Weld

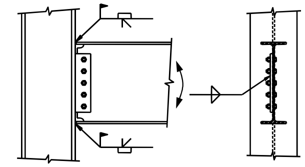
PJP joint welds are not recommended.

Fillet welds can be designed to develop the tensile strength of beam flange or resist the required beam moment:

$$D_{req'd} = \frac{0.9\ F_{yf}\ t_f\ (1\text{ in.})}{1.5 \times 1.392\ (1\text{ in.})}$$

or

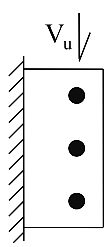
$$D_{req'd} = \frac{M_u/(d - t_f)}{1.5 \times 1.392 \times b_f}$$

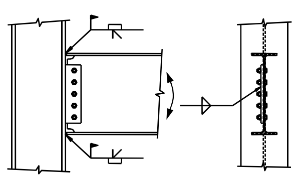


(Recommended)

Flange Welded / Web Bolted

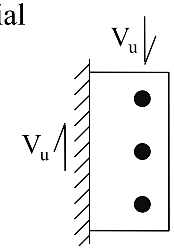
- Web Plate Limit States:
  - Shear Yielding
  - Shear Rupture
  - Shear Strength at the Elements
  - Block Shear
  - Weld Shear Rupture



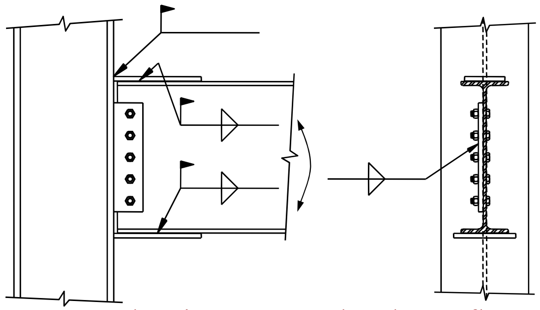


Flange Welded / Web Bolted

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



FLANGE PLATE WELDED / WEB BOLTED MOMENT CONNECTIONS



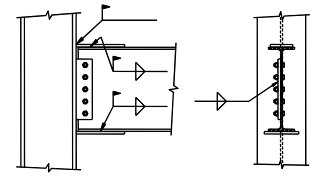
Top plate is narrower than beam flange.

Bottom plate is wider than beam flange.

Flange Plate Welded / Web Bolted

Limit States

- Flange Plates-to-Column Flange Welds
  - Complete Joint Penetration (CJP)
  - Partial Joint Penetration (PJP)
  - Weld Access Holes (not shown) required for backup bar at top and access behind web at bottom.
  - Plate-to-Column Flange weld must be made first.



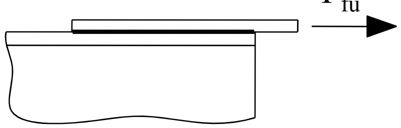
Flange Plate Welded / Web Bolted

- Tension Flange Plate Yielding

$F_{fu} = M_u / (d + t_{p,avg})$

$F_{fu} \le \phi F_y A_g$

$\phi = 0.9$

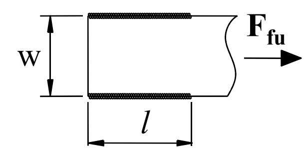


Flange Plate Welded / Web Bolted

- Tension Flange Plate Rupture

Longitudinal Welds Only

$F_{fu} \le \phi F_u A_e = \phi F_u U A_g$



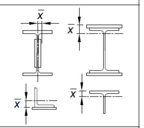
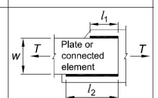
$\phi = 0.75$

Using *Specification* Table D3.1, Case 4 with $\bar{x} = 0$

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

Flange Plate Welded / Web Bolted

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

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

Flange Plate Welded / Web Bolted

- Tension Flange Plate Weld

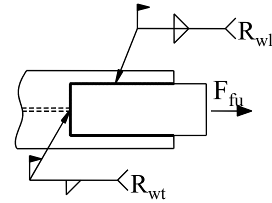
  $F_{fu} = M_u / d \leq \phi R_n \quad \phi = 0.75$

  - Only Longitudinal Welds

    $R_n = (1.392 \text{ x } D)(2 \text{ x } L_{wl})$

  - Long. + Transverse: *Specification* J2.4(b)(2)

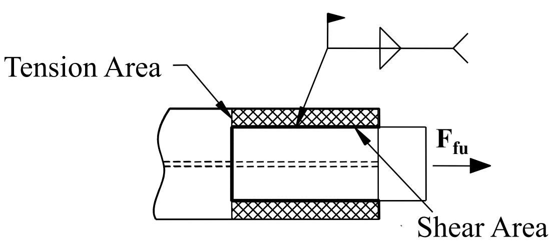
$$R_n = \max \left| \begin{array}{l} R_{wl} + R_{wt} \\ 0.85R_{wl} + 1.5R_{wt} \end{array} \right.$$



Flange Plate Welded / Web Bolted

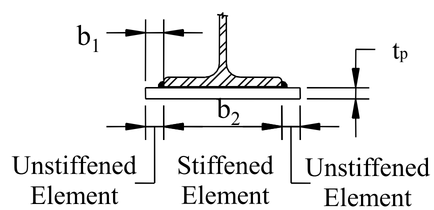
- Beam Tension Flange Block Shear

  Longitudinal welds only.



Flange Plate Welded / Web Bolted

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



Unstiffened : Case 1

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

Stiffened : Case 7

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

Flange Plate Welded / Web Bolted

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

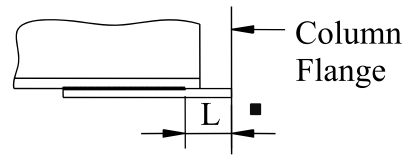
	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r (nonslender/slender)	Examples
Unstiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles connected with continuous contact,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

- Compression Flange Plate Local Buckling

Flange Plate Welded / Web Bolted

- Compression Plate Buckling
  - Flexural (Column) Buckling (Specification J4.4)

For $KL/r \le 25$
$\phi P_n = 0.9 F_y A_g$



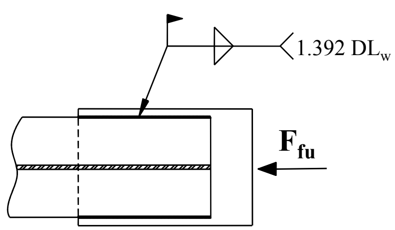
For $KL/r > 25$
$\phi P_n = 0.9 F_{cr} A_g$
and provisions of Chapter E apply with $K = 0.65$

Note: $r = t_p/\sqrt{12}$

Flange Plate Welded / Web Bolted

- Compression Flange Plate Weld

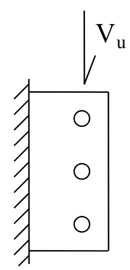
$F_{fu} = M_u / d \le \Sigma$ (weld strengths)



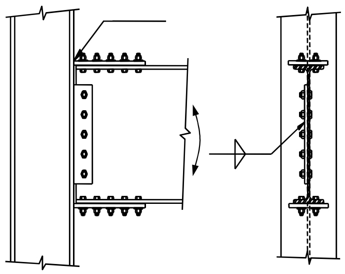
Flange Plate Bolted / Web Bolted

- Web Plate / Web Bolts

No eccentricity – all moment is resisted by flange connections.

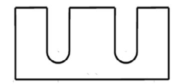


FLANGE PLATE BOLTED / WEB BOLTED MOMENT CONNECTIONS

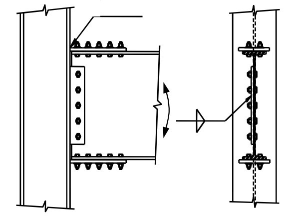


Flange Plate Bolted / Web Bolted

- Flange Plates
  - Flange plates usually welded in shop.
  - Top flange plate is located ¼″ to ⅜″ above tabulated beam depth to account for tolerances.
  - Finger shims are used if needed to fill gap when bolting.



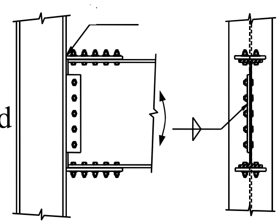
Finger Shim



Flange Plate Bolted / Web Bolted

Limit States

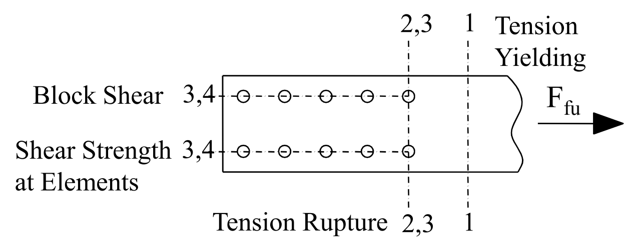
- Flange Plates-to-Column Flange Welds
  - Complete Joint Penetration (CJP)
  - Partial Joint Penetration (PJP)
  - Fillet Welds
  - If welded in the field, welding must be done before flange bolting and weld access holes may be required.



Flange Plate Bolted / Web Bolted

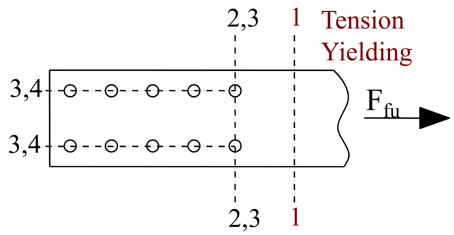
Limit States

- Tension Flange Plate Limit States (Same as for a Tension Member.)



Flange Plate Bolted / Web Bolted

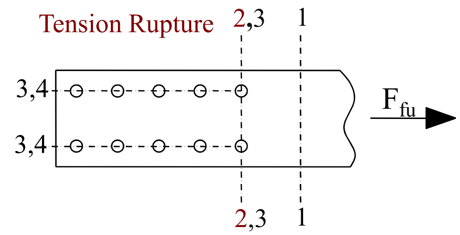
1. Tension Yielding



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

Flange Plate Bolted / Web Bolted

2. Tension Rupture



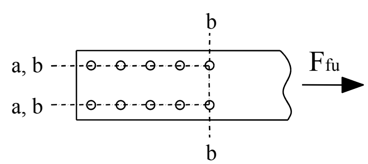
$$\phi\, T_n = 0.75\, F_u\, A_e$$

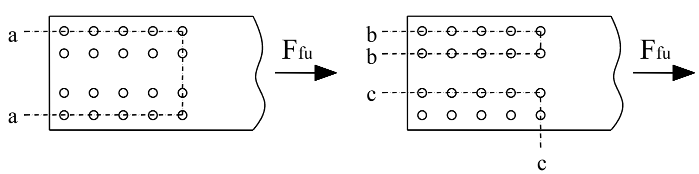
with $A_e = A_n$

Flange Plate Bolted / Web Bolted

3. Flange Plate Block Shear

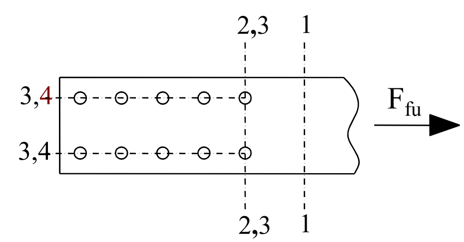
Block Shear Paths:





Flange Plate Bolted / Web Bolted

4. Shear Transfer at Elements



$\phi = 0.75$

$$R_n = \sum^{holes} \min.(\text{Bearing/Tear Out, Bolt Shear Rupture})$$

Flange Plate Bolted / Web Bolted

- COMPRESSION PLATE LIMIT STATES
  5. Shear Strength at Elements
  6. Local and Flexural Buckling

5. Shear Strength at Elements

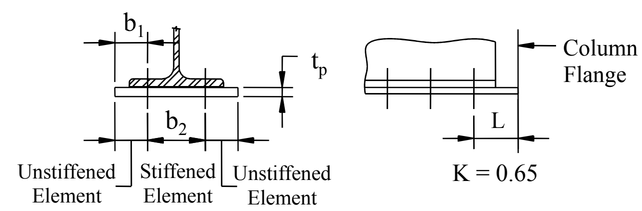
Same as for Tension Plate

$\phi = 0.75$

$R_n = \sum^{\text{holes}} \text{min.(Bearing/Tear Out, Bolt Shear Rupture)}$

Flange Plate Bolted / Web Bolted

6. Local and Flexural Buckling



Local Buckling (*Spec.* Section B4)

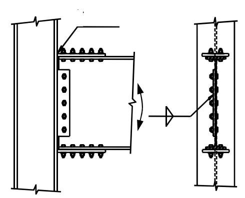
Flexural Buckling (*Spec.* Section J4.4)

Flange Plate Bolted / Web Bolted

7. Bolt Shear Rupture at Top and Bottom Flanges

Long connection rule (*Spec.* Table J3.2 Footnote [b])

For end loaded connections with a fastener pattern length greater than 38 in., F_{nv} shall be reduced to 83.3% of the tabulated values.

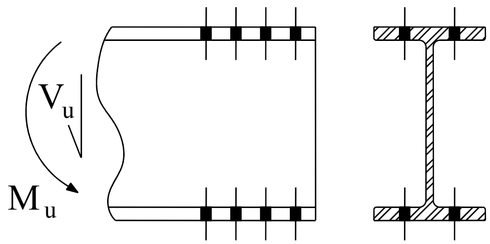


Flange Plate Bolted / Web Bolted

- Girder/Beam Limit States
  8. Reduced Flexural Strength
  9. Block Shear
  10. Bearing Tear-Out

Flange Plate Bolted / Web Bolted

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



Flange Plate Bolted / Web Bolted

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

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

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

If $F_u A_{fn} \geq Y_t F_y A_{fg}$ no reduction is needed.

where $Y_t = 1.0$ for $F_y/F_u \leq 0.8$ (A992: 50/65 = 0.77)

$= 1.1$ otherwise

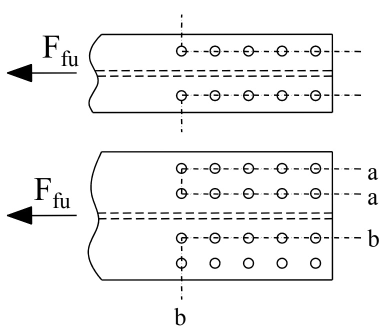
A_{fg} = flange gross area

A_{fn} = flange net area

Note: Reduction is based on tensile rupture strength.

Flange Plate Bolted / Web Bolted

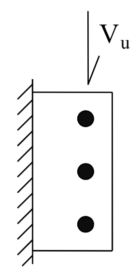
9. Beam Flange Block Shear



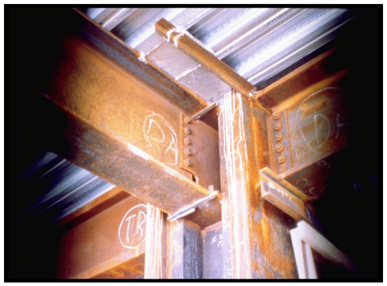
Flange Plate Bolted / Web Bolted

- Web Plate / Web Bolts

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



COLUMN SIDE LIMIT STATES AT MOMENT CONNECTIONS



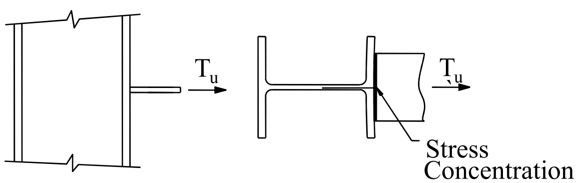
Column Side Limit States

Flange and Flange Plate M-Connections

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

Column Side Limit States

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



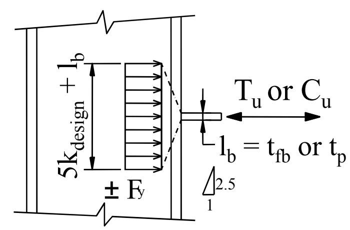
$\phi = 0.9$ $\quad T_u \leq \phi R_n$

$\phi R_n = 0.9\ (6.25\ t_{fc}^2\ F_{yc})$

If $T_u > \phi R_n$, half depth column web stiffeners required.

Column Side Limit States

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



$\phi = 1.0$

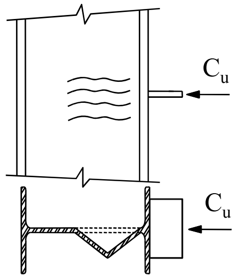
T_u or $C_u \leq \phi R_n = 1.0\ F_{yc}\ (5k_{design} + t_p)\ t_{wc}$

If T_u or $C_u > \phi R_n$, half depth stiffeners required

Column Side Limit States

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

Same design rules as for beam bearing with $l_b = t_p$ or t_{fb}.

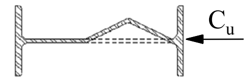


If $C_u > \phi R_n$, half depth column web stiffeners required.

Column Side Limit States

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

Web crippling @ $\geq d/2$



$\phi = 0.75$

$$R_n = 0.8\, t_{wc}^2 \left[1 + 3\left(\frac{l_b}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{EF_{yc}\, t_{fc}}{t_{wc}}}\, Q_f$$

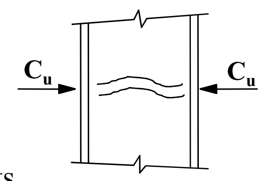
$Q_f = 1.0$ for wide flange sections

Column Side Limit States

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

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

$$\varphi R_n = 0.9 \frac{24\, t_{wc}^3 \sqrt{E\, F_{yc}}}{h} Q_f$$



h = clear distance between fillets
(get from tabulated h/t_w value)

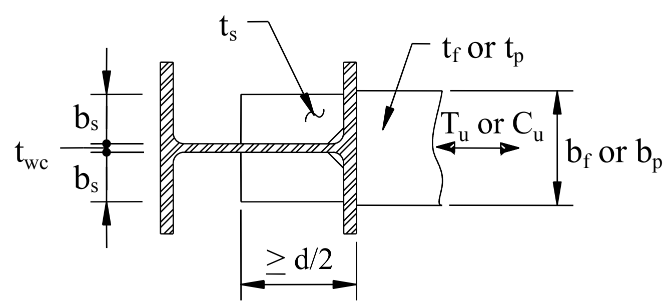
$Q_f = 1.0$ for wide flange sections

If max $C_u > \phi R_n$, full depth stiffeners required

Column Side Limit States

5. Transverse Stiffener Design

(*Spec.* J10.8 Requirements)



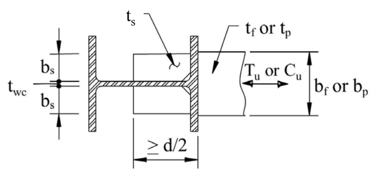
Column Side Limit States

5. Transverse Stiffener Design

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

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

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



Local Buckling Limit State
(Compression only)

Column Side Limit States

5. Transverse Stiffener Design

Load Path Assumptions When Transverse Stiffeners are Required:

Column resists ϕR_n for each limit state.
Stiffeners resist difference.

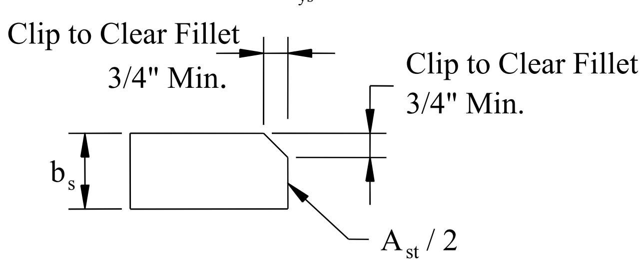
$$\text{Stiffener Force} = C_{u,net} \text{ or } T_{u,net}$$
$$= (C_u \text{ or } T_u) - \min \phi R_n$$
$$= R_u - \min \phi R_n$$

Column Side Limit States

5. Transverse Stiffener Design

Required Net Stiffener Area

$$A_{st} = \frac{R_u - \phi R_n}{0.9\, F_{ys}}$$



Column Side Limit States

5. Transverse Stiffener Design

Stiffener-to-Flange Weld

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

$$D = \frac{(0.9)\, F_y (b_s - \text{clip})\, t_s}{1.5\ (1.392)\ (b_s - \text{clip})\ (2)}$$

$$= \frac{(0.9)\, F_y\ t_s}{1.5\ (1.392)\ (2)} \quad (\text{no. of } 1/16\text{'s})$$

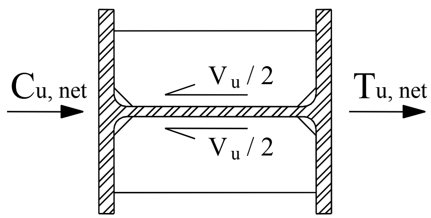
Or,

Complete Joint Penetration (CJP) Weld.

Column Side Limit States

5. Transverse Stiffener Design

Stiffener-to-Web Weld



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

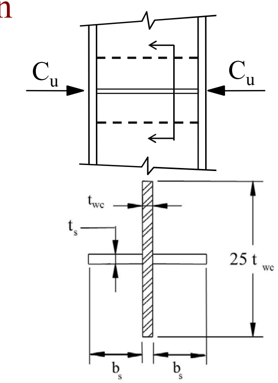
Column Side Limit States

5. Transverse Stiffener Design

When required for Web Compression Buckling

Design as a compression member with cruciform cross section.

Provisions are in *Spec.* J10.8.



Section

Column Side Limit States

5. Transverse Stiffener Design

Compression Member Design

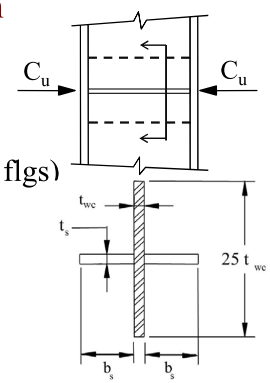
$A = 25\ t_w^{\,2} + (2b_s\ t_s)$

$K = 0.75$ (considering fixity at flgs)

$h = d_c - 2\ t_{fc}$

$Kh/r \rightarrow \phi F_{cr} \rightarrow \phi P_n$

$\phi P_n \geq \max\ C_u$



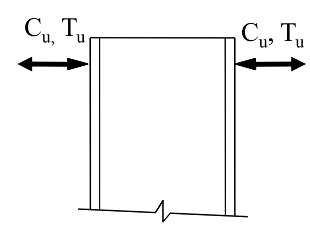
Section

Column Side Limit States

5. Transverse Stiffener Design

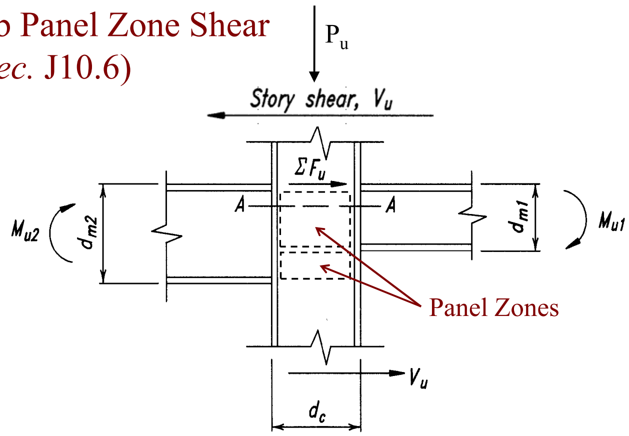
Connections at Top of Columns

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



Column Side Limit States

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



Column Side Limit States

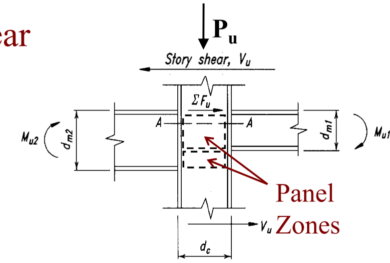
6. Web Panel Zone Shear
At Section A-A

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

$$d_{m1} = d_1 - t_{f1} \approx 0.95\ d_1$$

$$d_{m2} = d_2 - t_{f2} \approx 0.95\ d_2$$

$$\sum F_u \leq \phi R_v \qquad \phi = 0.9$$



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):

For $P_u \leq 0.4\ P_y = 0.4\ F_y\ A_g$

$R_v = (0.60\ F_y)(d_c\ t_{wc})$ (shear yielding)

For $P_u > 0.4\ P_y = 0.4\ F_y\ A_g$

$R_v = (0.60\ F_y)(d_c\ t_{wc})[1.4 - (P_u / P_y)]$

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.

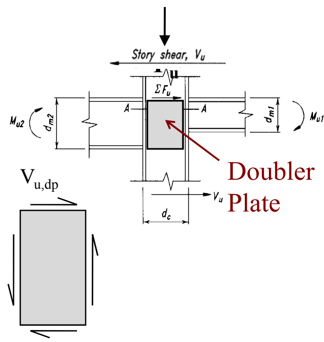
Column Side Limit States

6. Panel Zone Web Shear

Doubler Plate Design
Spec. Section J10.9

If $\Sigma F_u > \phi R_v$,
a doubler plate is required.

$$V_{u,dp} = \Sigma F_u - \phi R_{v,col}$$



Note: Two plates are recommended if required doubler thickness is greater than 1 in.

Column Side Limit States

6. Panel Zone Web Shear

Doubler Plate Shear Yielding or Buckling

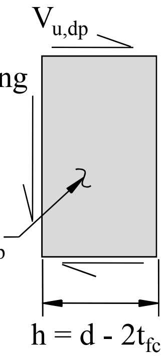
$V_u \le \phi V_n \qquad \phi = 1.0$

If $\dfrac{h}{t_p} \le 2.24\sqrt{\dfrac{E}{F_y}}$

$\phi V_n = 1.0(0.6F_y)(ht_p)$ (Shear Yielding)

Otherwise,

ϕV_n from *Specification* Section G2.



Column Side Limit States

6. Panel Zone Web Shear

Doubler Plate Shear Yielding or Buckling (*Spec.* G2-1)

$$\phi V_n = 0.9(0.6F_y)(ht_p)C_v$$

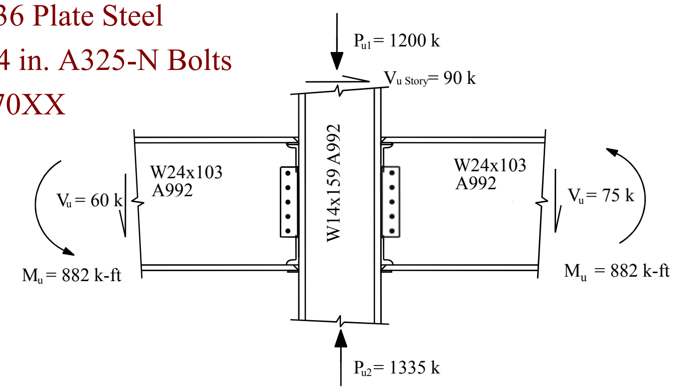
For $\dfrac{h}{t_p} \le 1.10\sqrt{\dfrac{k_v E}{F_y}}$ then $C_v = 1.0$ (Shear Yielding)

For $1.10\sqrt{\dfrac{k_v E}{F_y}} < \dfrac{h}{t_p}$

$$C_v = \frac{1.10\sqrt{k_v E / F_y}}{h / t_w} \quad \text{(Inelastic Buckling)}$$

Example: Design Flange Welded/Web Bolted Moment Connection

A36 Plate Steel
3/4 in. A325-N Bolts
E70XX

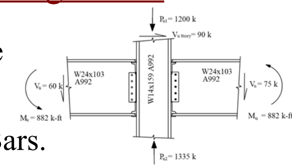


Ex. Flange Welded/Web Bolted M-Connection

	W24x103	W14x159
ϕM_n	1,050 k-ft	
d	24.5 in.	15.0 in.
t_w	0.550 in.	0.745 in.
b_f	9.000 in.	15.6 in.
t_f	0.980 in.	1.19 in.
k		1 .79 / 2-½ in.
h/t_w		15.3
A		46.7 in^2
T		10 in.

Ex. Flange Welded/Web Bolted M-Connection

W24x103 Flange-to-Column Flange Weld



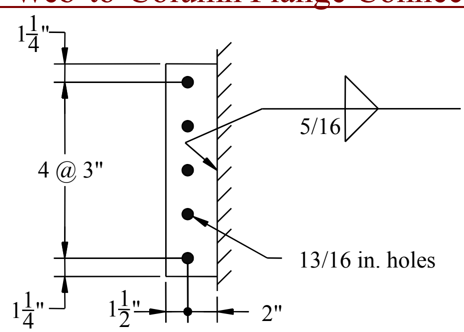
- Girder Flange-to-Column Flange
  Use Complete Penetration
  Welds (CJP) with Backing Bars.

W24x103 Web-to-Column Flange Weld

- Girder Web-to-Column Flange
  Single Plate
  No eccentricity considered.

Ex. Flange Welded/Web Bolted M-Connection

W24x103 Web-to-Column Flange Connection



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

Ex. Flange Welded/Web Bolted M-Connection

W24x103 Web-to-Column Flange Connection

$V_u = 75$ kips

Limit State Strengths

Shear Yielding:	$\phi V_n = 97.9$ k	OK
Shear Rupture:	$\phi V_n = 82.6$ k	OK
Block Shear:	$\phi V_n = 81.5$ k	OK
Shear Transfer @ Elements	$\phi V_n = 85.4$ k	OK
Weld Rupture:	$\phi V_n = 202$ k	OK

Ex. Flange Welded/Web Bolted M-Connection

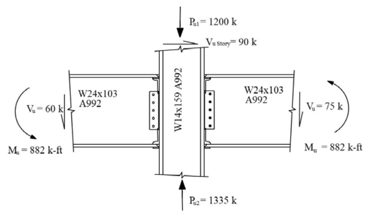
Beam Flange Forces

W24x103 d_c = 24.5 in. t_{fc} = 0.980 in.

M_u = 882 ft-kips

$T_u = C_u = (882 \times 12) / (24.5 - 0.980)$

$= 450$ k



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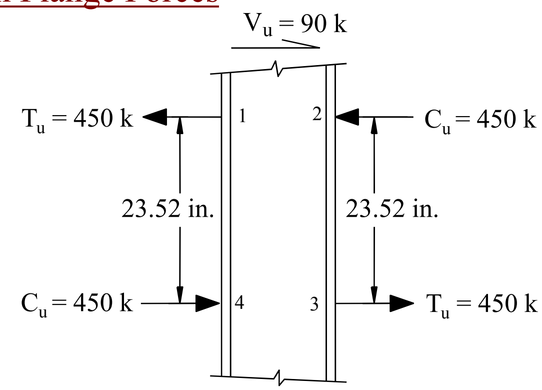
Ex. Flange Welded/Web Bolted M-Connection

Beam Flange Forces



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Ex. Flange Welded/Web Bolted M-Connection

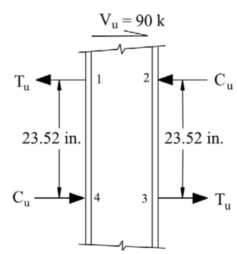
Column Flange Local Bending (*Spec.* J10.1)

$\phi R_n = 0.9(6.25\, t_{fc}^2\, F_y)$

$= 0.9\,(6.25 \times 1.19^2 \times 50)$

$= 398 \text{ k} < T_u = 450 \text{ k}$

For This Loading, Half-Depth Stiffeners Required at tension locations 1 and 3.



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Ex. Flange Welded/Web Bolted M-Connection

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

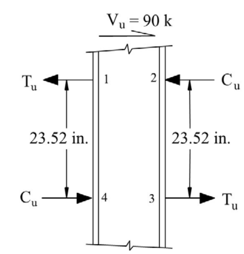
$\phi R_n = 1.0\,(5\, k_{design} + t_{fb})\, F_y\, t_{wc}$

$= 1.0\,(5 \times 1.79 + 0.980)\,(50)\,(0.745)$

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

Half-Depth Stiffeners Required at all locations.

Use Full Depth Stiffeners Top and Bottom.



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Ex. Flange Welded/Web Bolted M-Connection

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

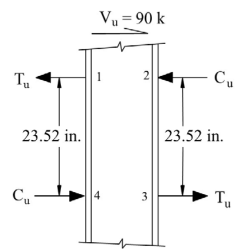
@ > d/2

$$\phi R_n = 0.75\,(0.80)\, t_{wc}^2 \left[1 + 3\left(\frac{l_b}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E\,F_{yw}\,t_{fc}}{t_{wc}}}\,Q_f$$

$$= 556\,\text{k}$$

W24x103 $\quad l_b = t_{fb} = 0.980$ in.

$C_u = 450$ k $< \phi R_n = 556$ k OK

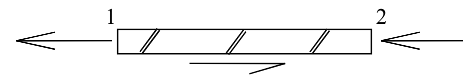


Ex. Flange Welded/Web Bolted M-Connection

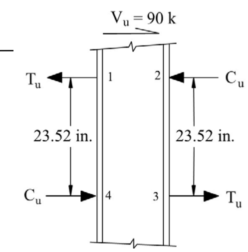
Transverse Stiffener 1-2 Design

$$T_{u1} = 450 - \min \left| \begin{array}{l} 398 \rightarrow \text{Flange Local Bending} \\ 370 \rightarrow \text{Web Local Yielding} \end{array} \right.$$

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

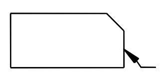


$V_u = 80.0 + 80.0$
$= 160$ k



Ex. Flange Welded/Web Bolted M-Connection

Transverse Stiffener 1-2 Design



Tension Side:

$A_{st,req'd} = 80.0 / (0.9 \times 36) = 2.46$ in^2

Compression Side:

$A_{st,req'd} = 80.0 / (0.9 \times 36) = 2.46$ in^2

Try PL 1/2 x 6

Clip: W14x159: $\quad k_{detailing} - t_f = 2.50 - 1.19 = 1.31''$

Use 1-½″ Clip

$A_{st} = 2\,(6 - 1.5)\,(0.50) = 4.50\ \text{in}^2 \geq A_{st,req'd}$ OK

Ex. Flange Welded/Web Bolted M-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:

$b_s = 6$ in. $\geq (9.0 / 3) - (0.745 / 2) = 2.62$ in. OK

Local Buckling:

$$\frac{b_s}{t_s} = \frac{6}{0.50} = 12.0 < 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29000}{36}} = 15.9 \quad \text{OK}$$

Use 2 PL 1/2 x 6 Top and Bottom

Ex. Flange Welded/Web Bolted M-Connection

Transverse Stiffener 1-2 Design

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

$$D = \frac{\text{Yield strength of stiffener plate / in.}}{\text{Rupture strength of fillet weld / in.}}$$

$$= \frac{0.9 \times 0.50 \times 36}{1.5 \times 1.392 \times 2} = 3.9 \quad 1/16\text{ths}$$

Minimum Weld Size = 1/4 in.

Use 1/4 in. fillet welds B.S.

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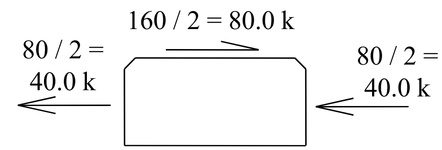
Ex. Flange Welded/Web Bolted M-Connection

Transverse Stiffener 1-2 Design

Stiffener-to-Column Web Welds:



$T = 10.0$ in. $> d_c - 2t_{fc} - 2\text{xclip} = 15.0 - 2(1.19) - 2(1.5)$
$= 9.62''$ Use 9-½″

D = (80) / (9.5 x 2 x 1.392) = 3.02 1/16s

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

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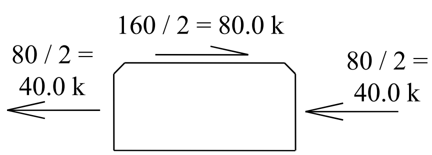
Ex. Flange Welded/Web Bolted M-Connection

Transverse Stiffener 1-2 Design

Strength of Column Web at Welds:



$\phi V_n = 0.75(0.6F_u)(t_{wc}L_w)$
$= 0.75(0.6x65)(0.745x9.5)$
$= 207$ k $> 2x80.0$ k $= 160$ k at T or B Weld Line

OK

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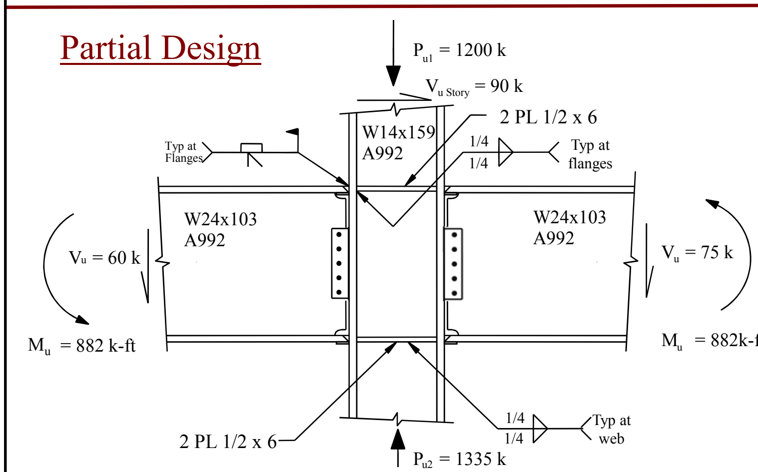
Ex. Flange Welded/Web Bolted M-Connection

Partial Design



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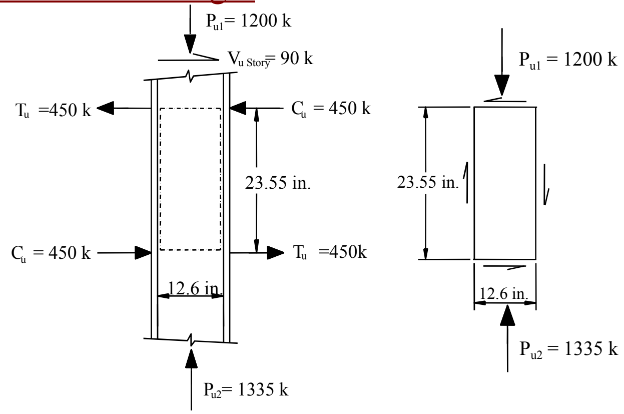
Ex. Flange Welded/Web Bolted M-Connection

Panel Zone Strength



Ex. Flange Welded/Web Bolted M-Connection

Panel Zone Strength

$V_u = 450 + 450 - 90.0$

$= 810$ k

$P_u = (1{,}200 + 1{,}335) / 2$

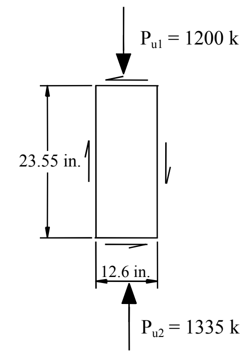
$= 1{,}268$ k

$0.4\ P_y = 0.4\ F_y\ A_g$

$= 0.4\ (50 \times 46.7)$

$= 0.4\ (2335)$

$= 934 \text{ k} < 1{,}268 \text{ k}$



Ex. Flange Welded/Web Bolted M-Connection

Panel Zone Strength

Shear Strength of Column Web

Since $P_u > 0.4\ P_y = 934$ k

$\phi V_n = 0.9\ (0.6\ F_y)\ (d_c \times t_{wc})\ (1.4 - P_u / P_y)$

$= 0.9(0.6 \times 50)(15.0 \times 0.745)\ (1.4 - 1268 / 2335)$

$= 259 \text{ k} < V_u = 810 \text{ k}$

Doubler Plate(s) Required

$$t_{dp,req'd} = \frac{810 - 259}{0.9(0.6 \times 50)(15.0)} = 1.36 \text{ in.}$$

Ex. Flange Welded/Web Bolted M-Connection

Panel Zone Strength

Try (2) 3/4 in. Web Doubler Plates

$h = d - 2\ t_{fc}$

$= 15.0 - 2\ (1.190)$

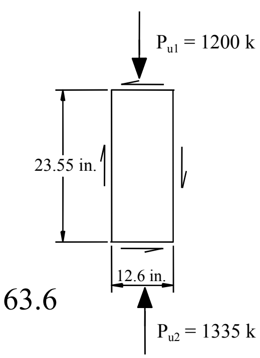
$= 12.6$ in.

Doubler Plate Slenderness:

$$\frac{h}{t_p} = \frac{12.6}{0.75} = 16.8 < 2.24\sqrt{\frac{29000}{36}} = 63.6$$

Shear yielding controls as assumed.

Use 2 – 3/4″ A36 Doubler Plates



Ex. Flange Welded/Web Bolted M-Connection

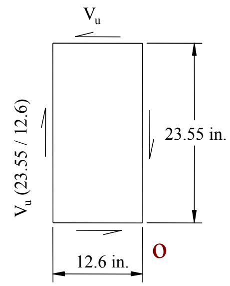
Panel Zone Welds

Long Side:

$\sum M_o = 0$

$$V_{uw} = \left(\frac{810-259}{2}\right)\left(\frac{23.55}{12.6}\right)$$

$$= 515 \text{ k}$$



D = 515 / (1.392 x 23.55) = 15.7 – 1/16s

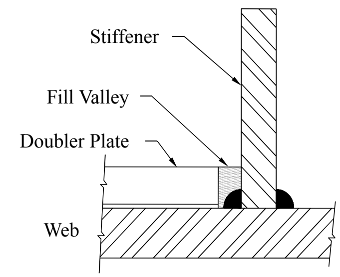
Use Complete Joint Penetration Weld (CJP)

Ex. Flange Welded/Web Bolted M-Connection

Panel Zone Welds

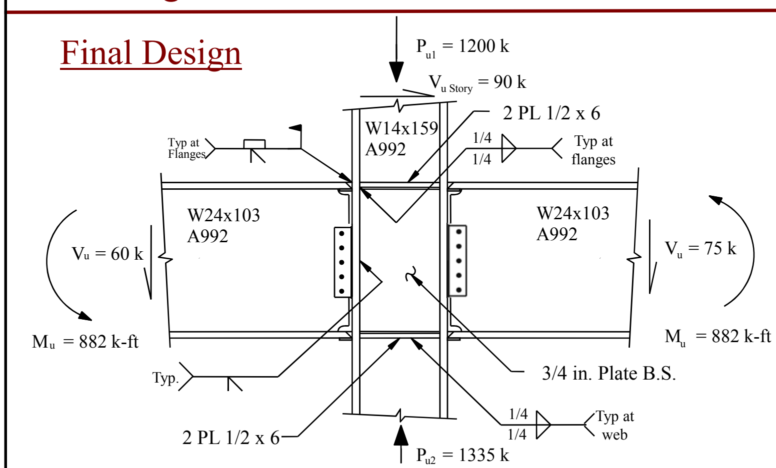
Short Side:

Use Minimum Fillet Weld, 1/4 in., or Fill Valley



Ex. Flange Welded/Web Bolted M-Connection

Final Design



End of Session 5
Thank You for Attending

Next Up

Next Session

- November 14, 2017 Moment Connections Part II

TOPICS

- Tee Stub Moment Connections
- End-Plate Moment Connections

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Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

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

8-Session Registrants

CEU/PDH Certificates

One certificate will be issued at the conclusion of all 8 sessions.

8-Session Registrants

Access to the quiz: Information for accessing the quiz will be emailed to you by Thursday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG

Quiz and Attendance records: Posted Wednesday mornings. www.aisc.org/nightschool - click on Current Course Details.

Reasons for quiz:

- EEU – must take all quizzes and final to receive EEU
- CEUs/PDHS – If you watch a recorded session you must take quiz for CEUs/PDHs.
- REINFORCEMENT – Reinforce what you learned tonight. Get more out of the course.

NOTE: If you attend the live presentation, you do not have to take the quizzes to receive CEUs/PDHs.

8-Session Registrants

Access to the recording: Information for accessing the recording will be emailed to you by this Thursday. The recording will be available for three weeks. For 8-session registrants only. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

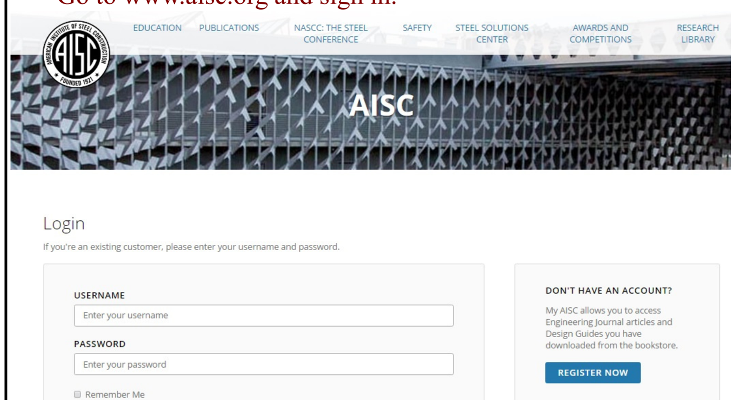
CEUs/PDHS – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHs.

Night School Resources for 8-session package Registrants

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

Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.



Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.

IN THIS SECTION
- Edit Profile
- My Downloads
- My Pending Quizzes
- My Events
- Order History
- Course History
- Course Resources

MyAISC

MY PROFILE

Update your contact and address information.

EDIT PROFILE

MY PURCHASED DOWNLOADS

Access articles and documents that you have purchased.

VIEW DOWNLOADS

MY COURSE RESOURCES

View online resources for Night School and Live Webinar package registrations.

VIEW RESOURCES

Night School Resources for 8-session package Registrants

AISC › MYAISC › COURSE RESOURCES

Course Resources

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

Night School Resources for 8-session package Registrants

AISC › MYAISC › NIGHT SCHOOL RESOURCES › NS13 8-SESSION PACKAGE RESOURCES

Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	View Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Dsn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	

Night School Resources for 8-session package Registrants

- Weekly "quiz and recording" email.
- Weekly updates of the master Quiz and Attendance record found at www.aisc.org/nightschool. Scroll down to Quiz and Attendance records.
  - Updated on Wednesday mornings.

Night School Resources for 8-session package Registrants

- Webinar connection information:
  - Found in your registration confirmation/receipt.
  - Reminder email sent out Tuesday mornings.
- Link to handouts also found here.

There's always a solution in steel.

Thank You

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

structural STEEL