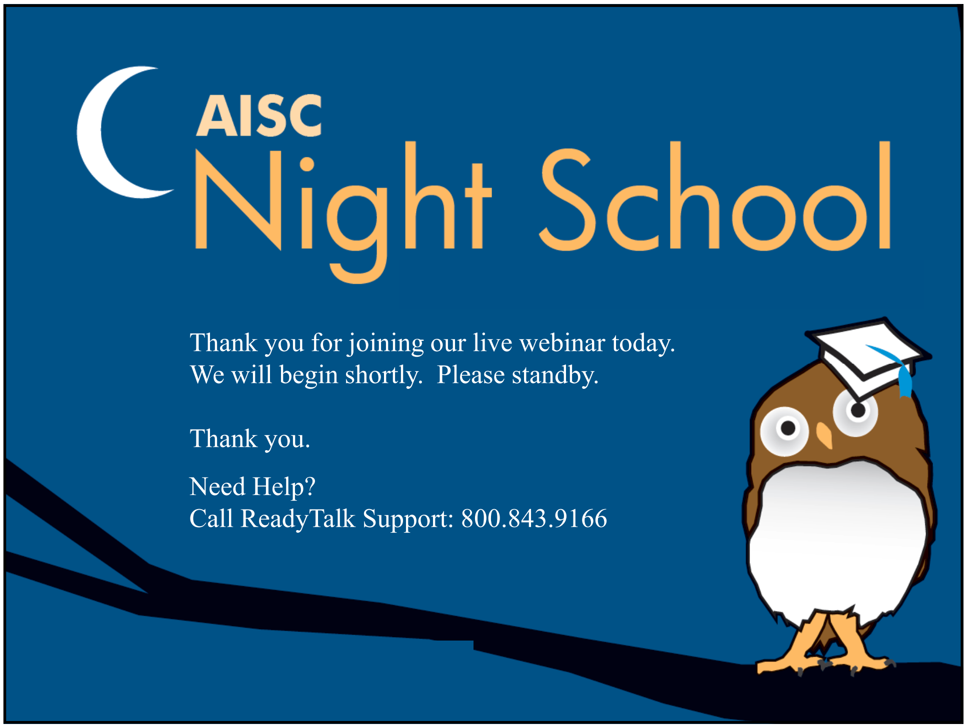

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Course Description

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

Lecturer: Tom Murray, P.E., PhD
Emeritus Professor, Virginia Tech

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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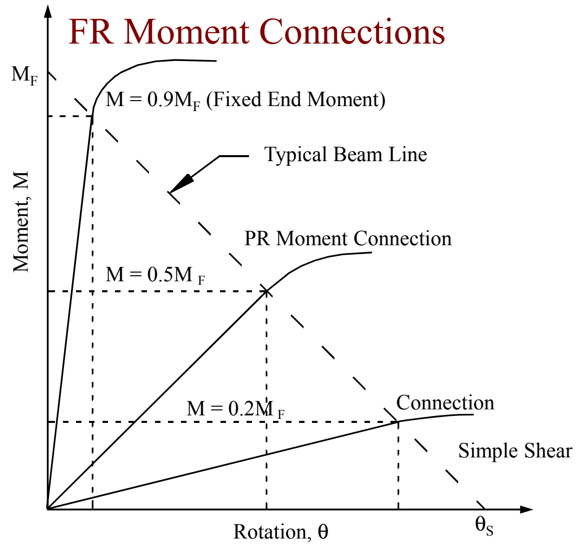
Moment Connections



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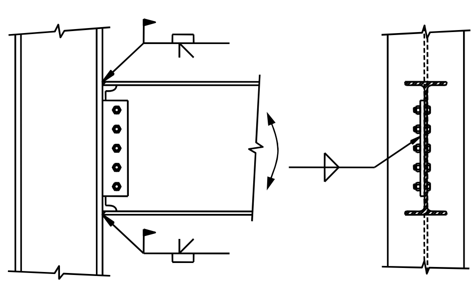
FLANGE WELDED / WEB BOLTED MOMENT CONNECTIONS



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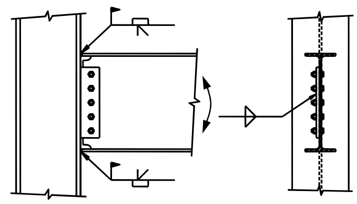
Flange Welded / Web Bolted

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



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Flange Welded / Web Bolted

- Girder Flange-to-Column Flange Weld

PJP joint welds are not recommended.

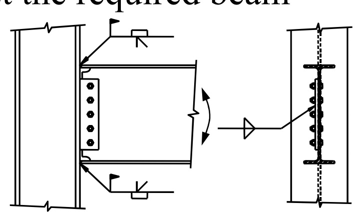
Fillet welds can be designed to develop the tensile strength of beam flange or resist the required beam moment:

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

or

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

(Recommended)



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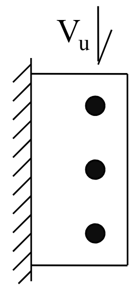
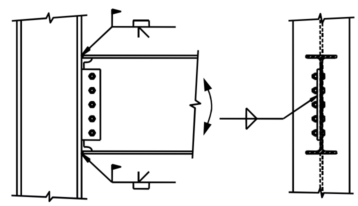
Flange Welded / Web Bolted

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





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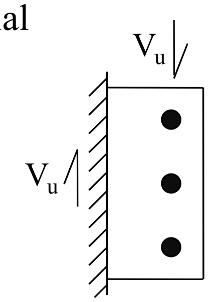
Flange Welded / Web Bolted

- 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.



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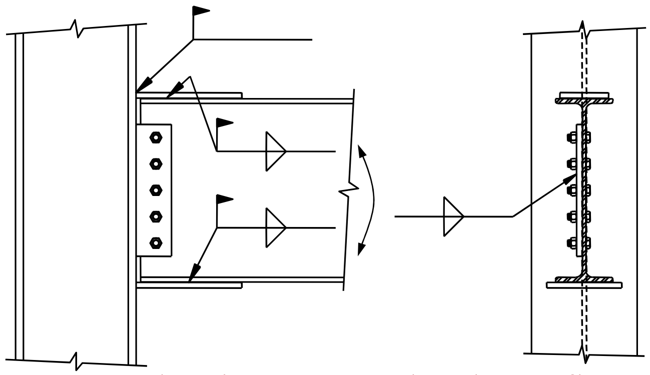
FLANGE PLATE WELDED / WEB BOLTED MOMENT CONNECTIONS



Top plate is narrower than beam flange.

Bottom plate is wider than beam flange.

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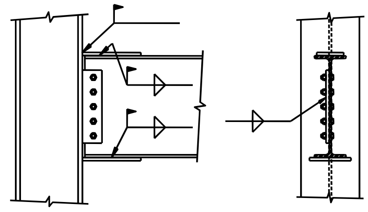
Flange Plate Welded / Web Bolted

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.



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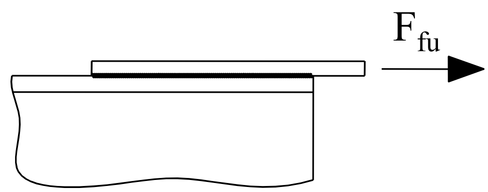
Flange Plate Welded / Web Bolted

- Tension Flange Plate Yielding

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

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

$\phi = 0.9$



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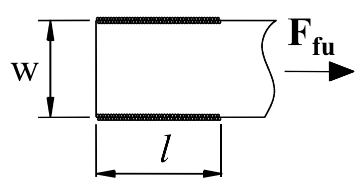
Flange Plate Welded / Web Bolted

- Tension Flange Plate Rupture

Longitudinal Welds Only

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



$\phi = 0.75$

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

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

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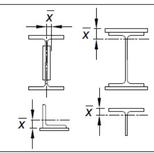
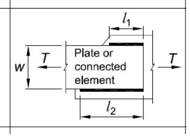
Flange Plate Welded / Web Bolted

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

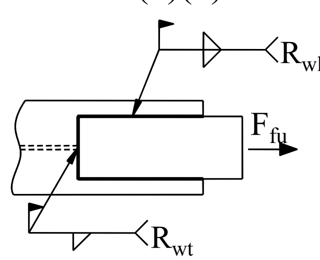
  $F_{fu} = M_u / d \leq \phi R_n \quad \phi = 0.75$

  - Only Longitudinal Welds

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

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

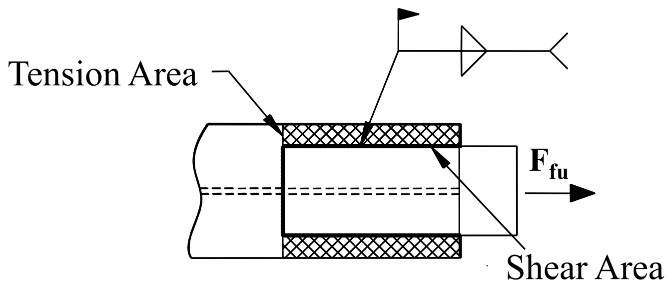
$$R_n = \max \left| \begin{array}{l} R_{wl} + R_{wt} \\ 0.85R_{wl} + 1.5R_{wt} \end{array} \right.$$



Flange Plate Welded / Web Bolted

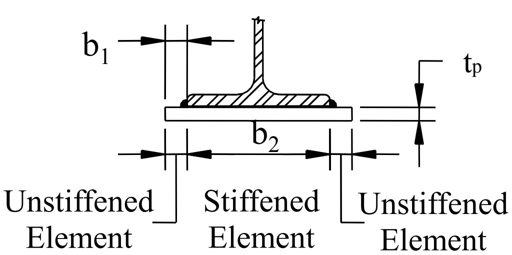
- Beam Tension Flange Block Shear

  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	$0.64\sqrt{\frac{k_c E}{F_y}}$ [a]	
	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			

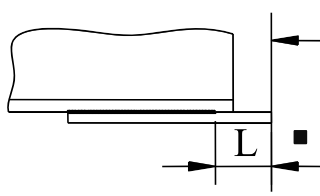
- Compression Flange Plate Local Buckling

Flange Plate Welded / Web Bolted

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

For $KL/r \leq 25$

$$\phi P_n = 0.9 F_y A_g$$



For $KL/r > 25$

$$\phi P_n = 0.9 F_{cr} A_g$$

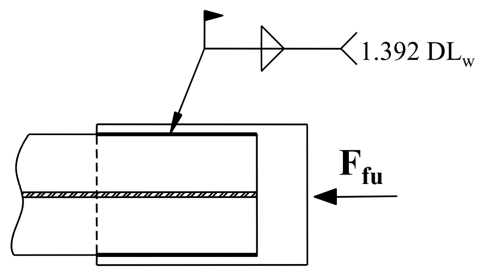
and provisions of Chapter E apply with $K = 0.65$

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

Flange Plate Welded / Web Bolted

- Compression Flange Plate Weld

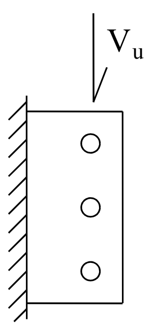
$F_{fu} = M_u / d \leq \Sigma$ (weld strengths)



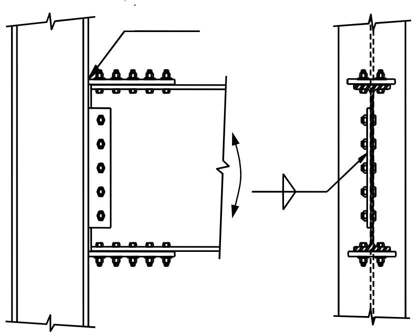
Flange Plate Bolted / Web Bolted

- Web Plate / Web Bolts

No eccentricity – all moment is resisted by flange connections.



FLANGE PLATE BOLTED / WEB BOLTED MOMENT CONNECTIONS



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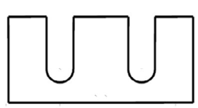
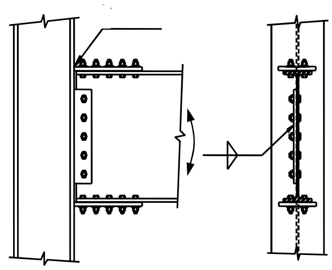
Flange Plate Bolted / Web Bolted

- 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



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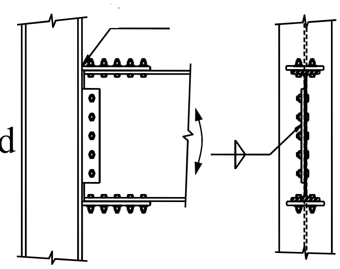
Lecturer: Tom Murray, P.E., PhD
Emeritus Professor, Virginia Tech

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.



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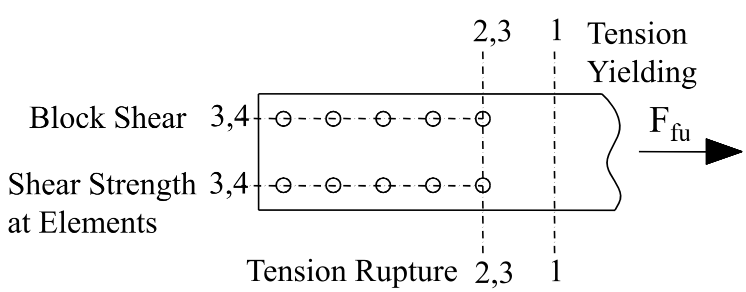
Flange Plate Bolted / Web Bolted

Limit States

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



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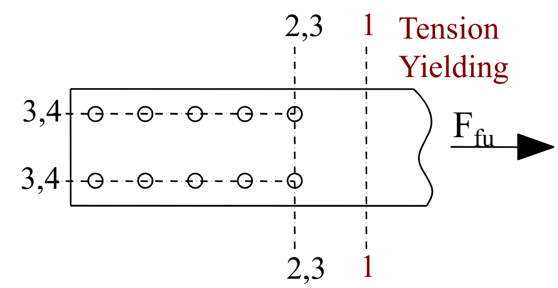
Flange Plate Bolted / Web Bolted

1. Tension Yielding



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

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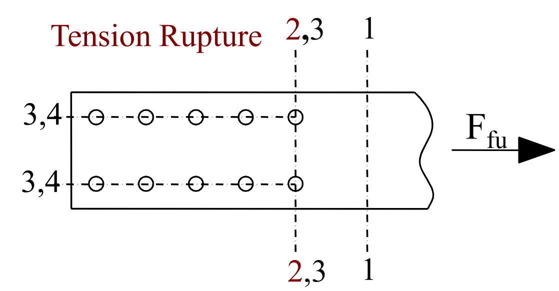
Flange Plate Bolted / Web Bolted

2. Tension Rupture



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

with $A_e = A_n$

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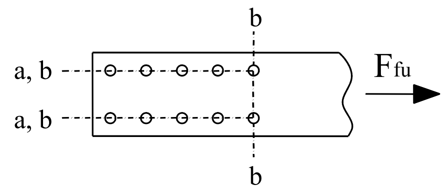
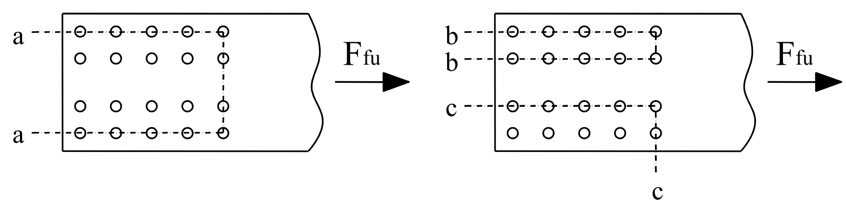
Flange Plate Bolted / Web Bolted

3. Flange Plate Block Shear

Block Shear Paths:





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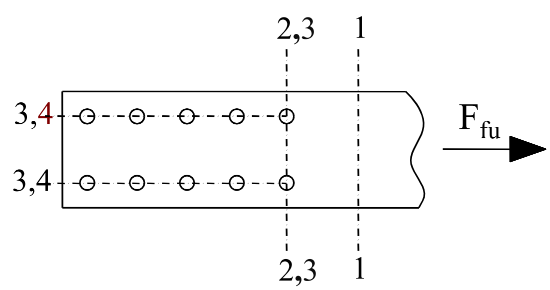
Flange Plate Bolted / Web Bolted

4. Shear Transfer at Elements



$\phi = 0.75$

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

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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

$\phi = 0.75$

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

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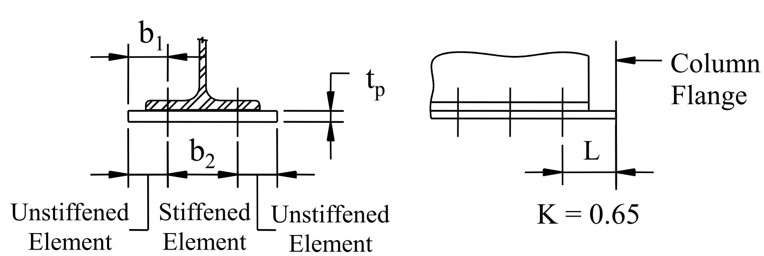
Flange Plate Bolted / Web Bolted

6. Local and Flexural Buckling



Local Buckling
(*Spec.* Section B4)

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

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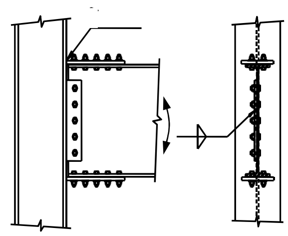
Flange Plate Bolted / Web Bolted

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.



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Flange Plate Bolted / Web Bolted

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

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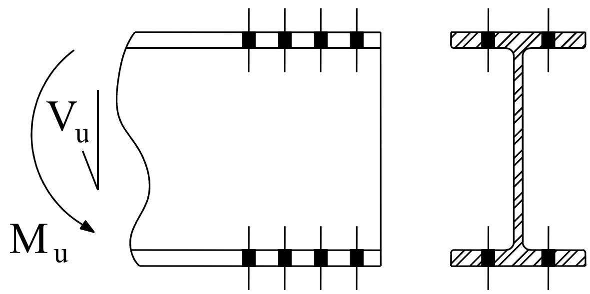
Flange Plate Bolted / Web Bolted

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



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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.

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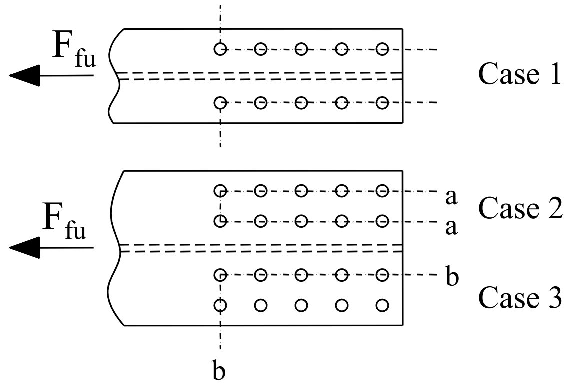
Flange Plate Bolted / Web Bolted

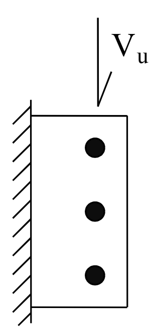
9. Beam Flange Block Shear



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

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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)

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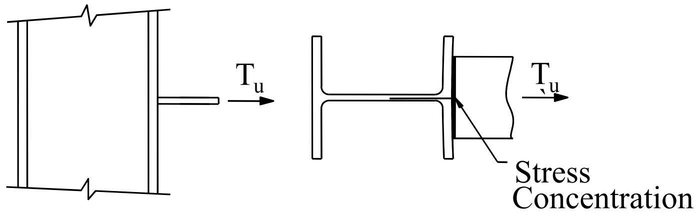
Column Side Limit States

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



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

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

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

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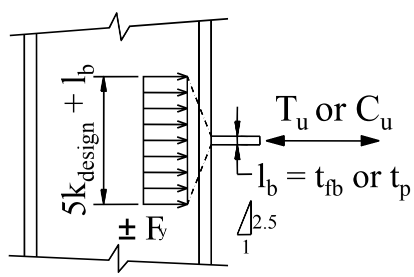
Column Side Limit States

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



$\phi = 1.0$

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

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

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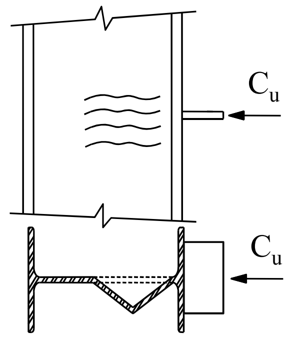
Column Side Limit States

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

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



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

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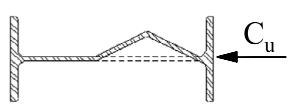
Column Side Limit States

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

Web crippling @ ≥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

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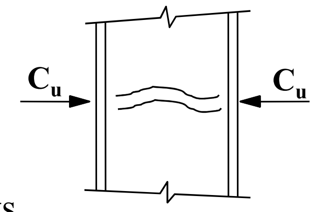
Column Side Limit States

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

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

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



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

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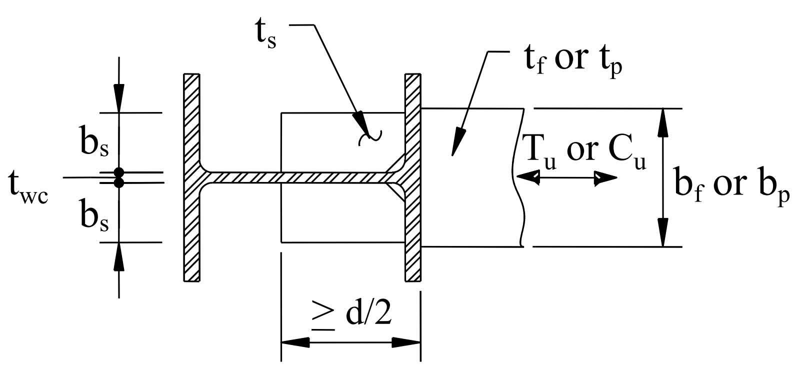
Column Side Limit States

5. Transverse Stiffener Design

(*Spec.* J10.8 Requirements)



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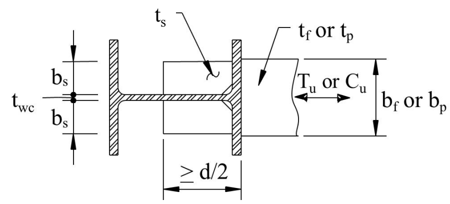
Column Side Limit States

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)

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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.

Stiffener Force = $C_{u,net}$ or $T_{u,net}$

= (C_u or T_u) – min ϕR_n

= R_u – min ϕR_n

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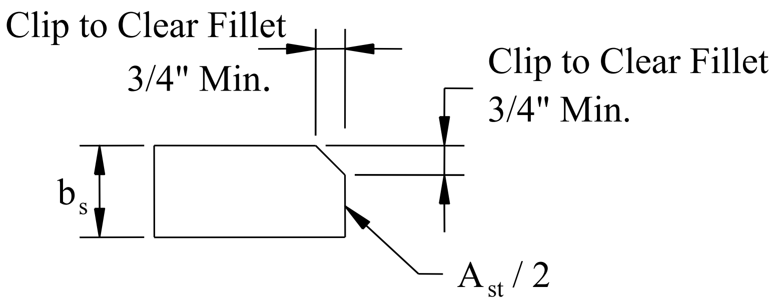
Column Side Limit States

5. Transverse Stiffener Design

Required Net Stiffener Area

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



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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.

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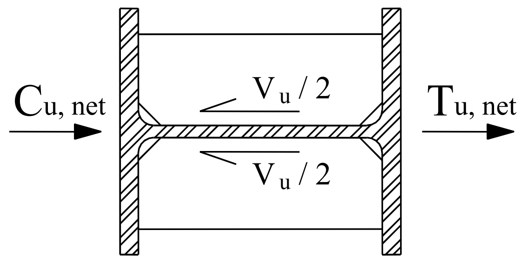
Column Side Limit States

5. Transverse Stiffener Design

Stiffener-to-Web Weld



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

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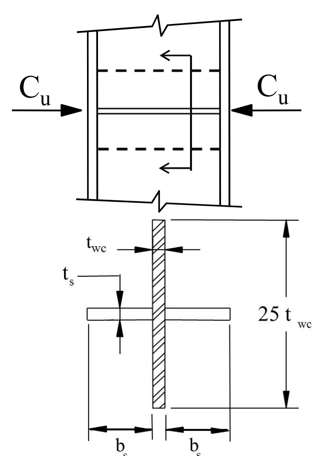
Column Side Limit States

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.



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Column Side Limit States

5. Transverse Stiffener Design

Compression Member Design

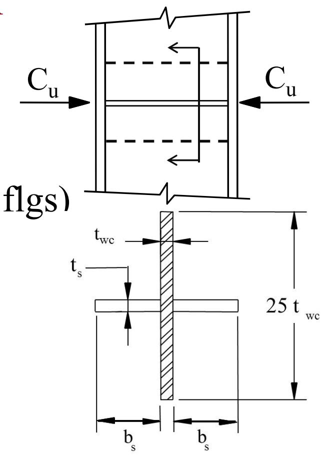
$A = 25\ t_w^{\ 2} + (2b_s\ t_s)$

$K = 0.75$ (considering fixity at flgs)

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

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

$\phi P_n \geq \max C_u$



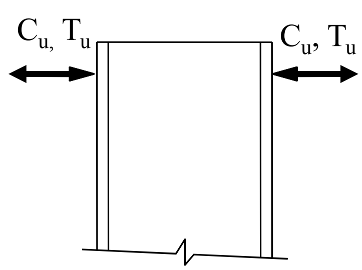
Section

Column Side Limit States

5. Transverse Stiffener Design

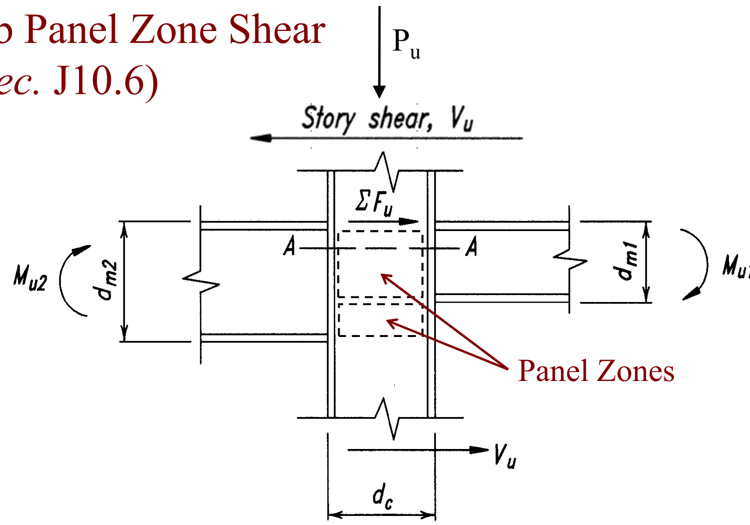
Connections at Top of Columns

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)



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Column Side Limit States

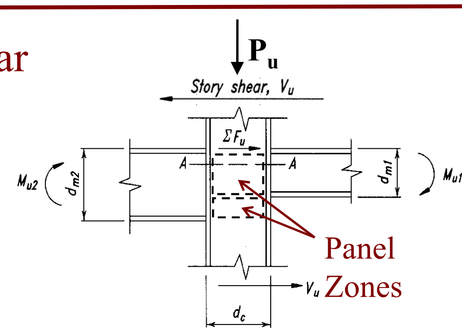
6. Web Panel Zone Shear
At Section A-A

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

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

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

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



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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):

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

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

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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.

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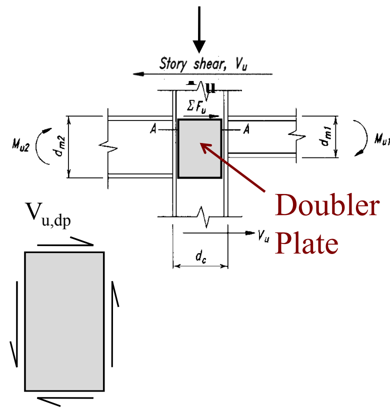
Column Side Limit States

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

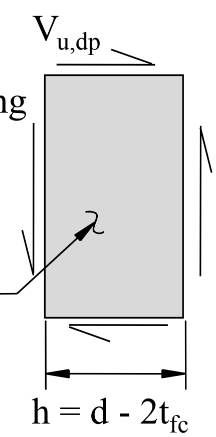
$V_u \leq \phi V_n \qquad \phi = 1.0$

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

$\phi V_n = 1.0(0.6F_y)(ht_p)$ (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 $\dfrac{h}{t_p} \leq 1.10\sqrt{\dfrac{k_v E}{F_y}}$ then $C_v = 1.0$ (Shear Yielding)

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

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

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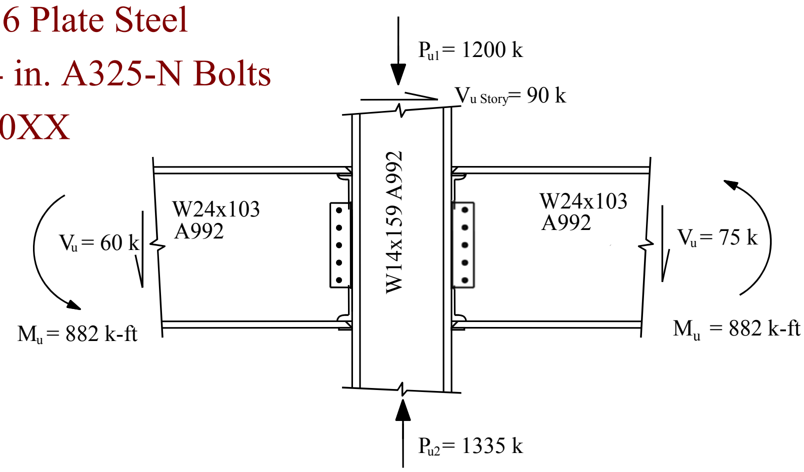
Example: Design Flange Welded/Web Bolted Moment Connection

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



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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.

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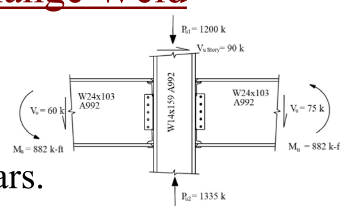
Ex. Flange Welded/Web Bolted M-Connection

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.

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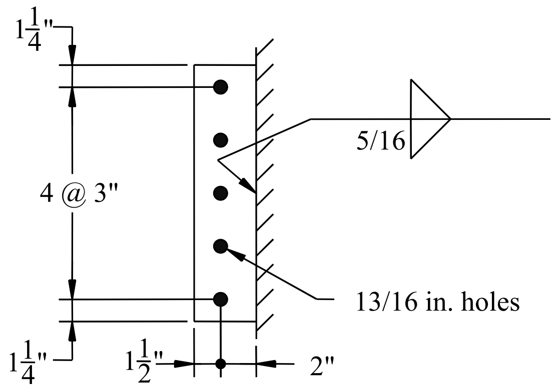
Ex. Flange Welded/Web Bolted M-Connection

W24x103 Web-to-Column Flange Connection



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

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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

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Ex. Flange Welded/Web Bolted M-Connection

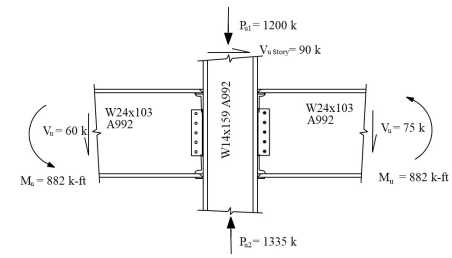
Beam Flange Forces

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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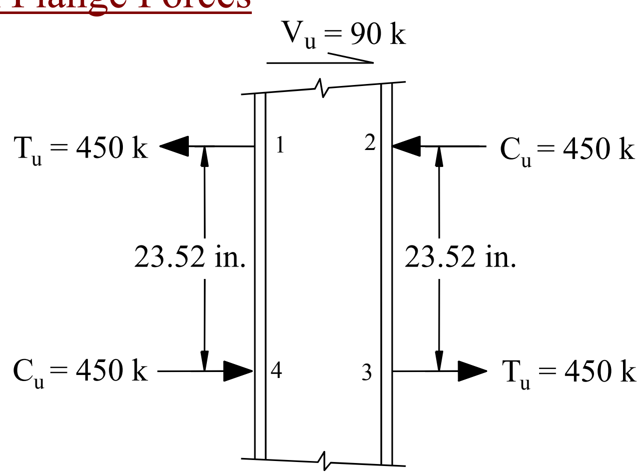
Ex. Flange Welded/Web Bolted M-Connection

Beam Flange Forces



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Ex. Flange Welded/Web Bolted M-Connection

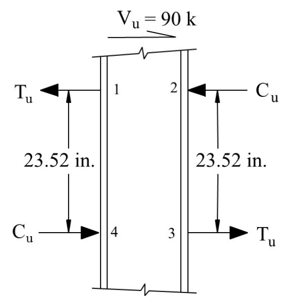
Column Flange Local Bending (*Spec.* J10.1)

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

$= 0.9\,(6.25 \text{ x } 1.19^2 \text{ x } 50)$

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

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)

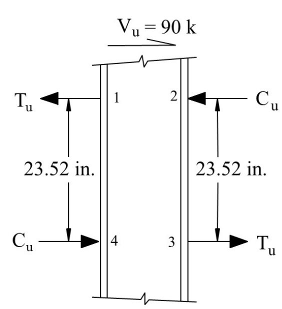
$\phi R_n = 1.0\,(5\, k_{design} + t_{fb})\, F_y\, t_{wc}$

$= 1.0\,(5 \text{ x } 1.79 + 0.980)\,(50)\,(0.745)$

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

Half-Depth Stiffeners Required at all locations.

Use Full Depth Stiffeners Top and Bottom.



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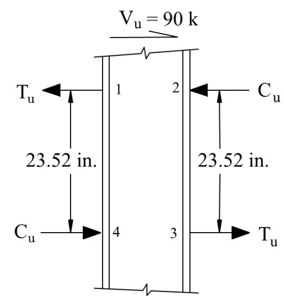
Ex. Flange Welded/Web Bolted M-Connection

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 $l_b = t_{fb} = 0.980$ in.

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

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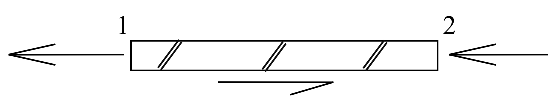
Ex. Flange Welded/Web Bolted M-Connection

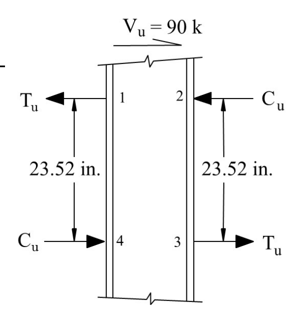
Transverse Stiffener 1-2 Design

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

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



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



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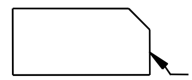
Ex. Flange Welded/Web Bolted M-Connection

Transverse Stiffener 1-2 Design



A_{st} /2

Tension Side:

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

Compression Side:

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

Try PL 1/2 x 6

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

Use 1-½″ Clip

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

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Ex. Flange Welded/Web Bolted M-Connection

Transverse Stiffener 1-2 Design

Minimum Thickness:

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

Minimum Width:

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

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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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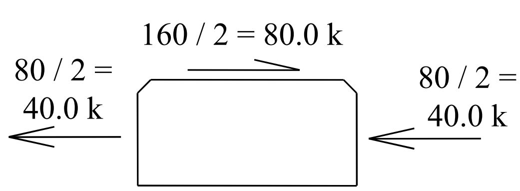
Ex. Flange Welded/Web Bolted M-Connection

Transverse Stiffener 1-2 Design

Stiffener-to-Column Web Welds:



T = 10.0 in. > d_c-2t_{fc}–2xclip = 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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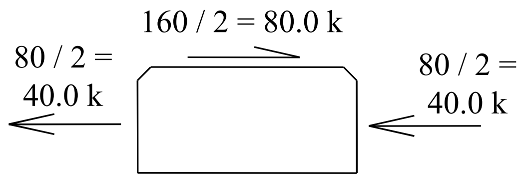
Ex. Flange Welded/Web Bolted M-Connection

Transverse Stiffener 1-2 Design

Strength of Column Web at Welds:



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

OK

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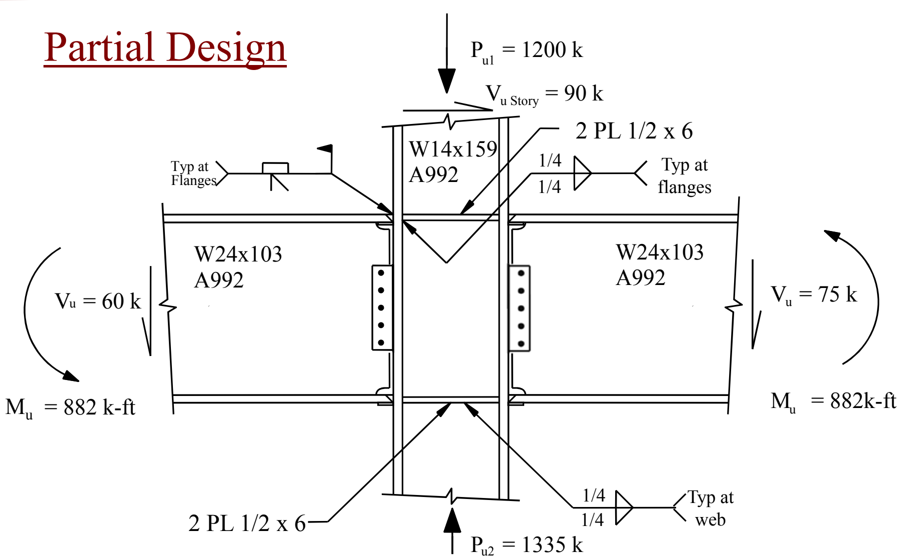
Ex. Flange Welded/Web Bolted M-Connection

Partial Design



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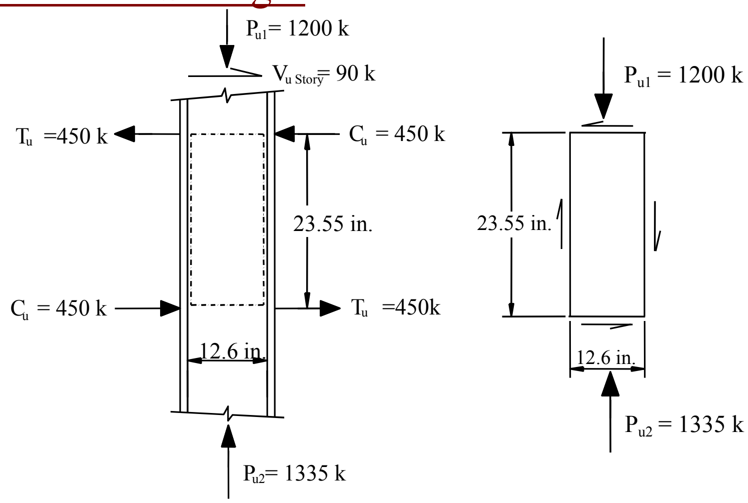
Ex. Flange Welded/Web Bolted M-Connection

Panel Zone Strength



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Ex. Flange Welded/Web Bolted M-Connection

Panel Zone Strength

$V_u = 450 + 450 - 90.0$

$= 810$ k

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

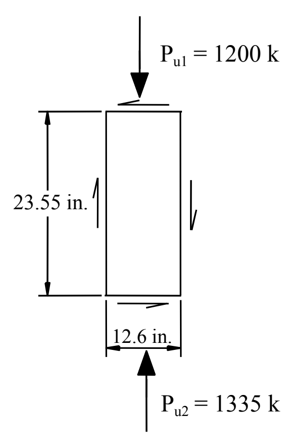
$= 1{,}268$ k

$0.4\ P_y = 0.4\ F_y\ A_g$

$= 0.4\ (50 \times 46.7)$

$= 0.4\ (2335)$

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



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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

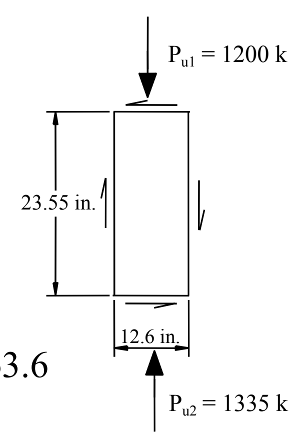
Panel Zone Strength

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

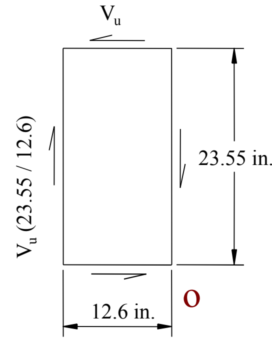
Panel Zone Welds

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

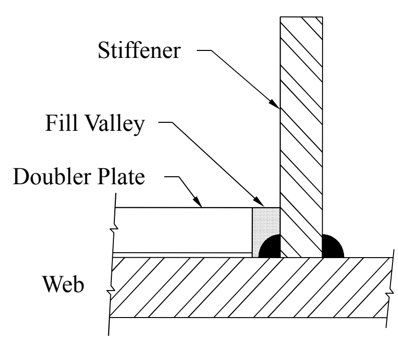
Use Complete Joint Penetration Weld (CJP)

Ex. Flange Welded/Web Bolted M-Connection

Panel Zone Welds

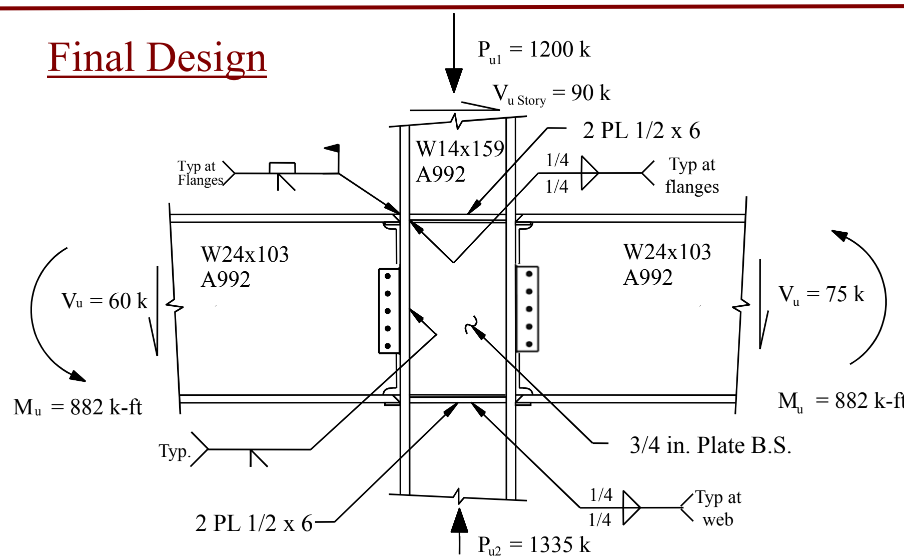
Short Side:

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



Ex. Flange Welded/Web Bolted M-Connection

Final Design



VirginiaTech Invent the Future

91

End of Session 5
Thank You for
Attending

Next Up

VirginiaTech Invent the Future

92

Next Session

- November 14, 2017 Moment Connections Part II

TOPICS

- Tee Stub Moment Connections
- End-Plate Moment Connections

VirginiaTech
Invent the Future

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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.

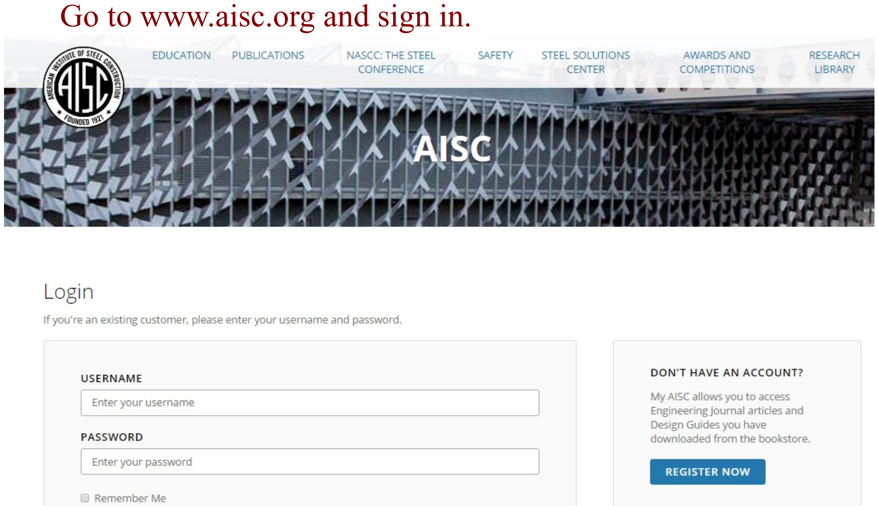
CEUs/PDHS – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHs.

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!

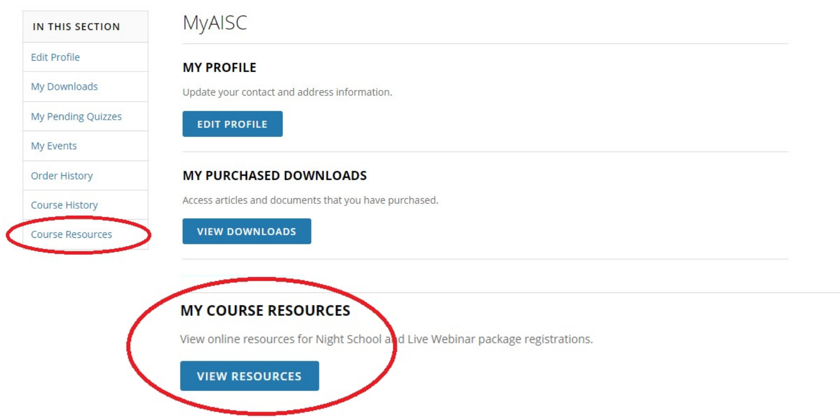
Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.

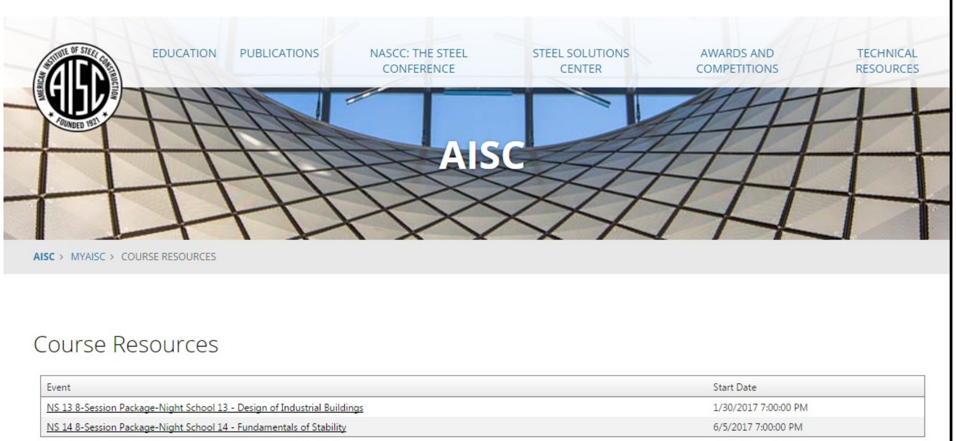


Night School Resources for 8-session package Registrants

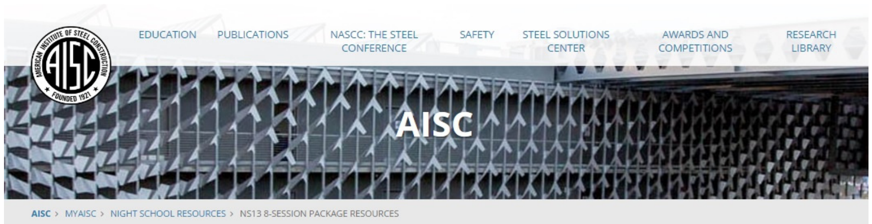
Go to www.aisc.org and sign in.



Night School Resources for 8-session package Registrants



Night School Resources for 8-session package Registrants



Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	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.

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