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AISC Night School

Connection Design

Tips, Tricks, and Lessons Learned

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Course Description

19.4 Moment Connections
March 4, 2019

Bolted or welded; should the EoR and delegated connection designer collaborate on this decision? Gravity or Lateral; does it make a difference in the design of the connections? Beam axial load equals transfer force; should it be obvious to the connection designer where, and for what magnitude, transfer forces are required in design?

This presentation will present the information that a delegated connection designer will: (1) need to in order to make an informed decision on connection type and design, and; (2) need to provide to the EoR to facilitate the review process.

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Learning Objectives

- Identify unverified assumptions that connection design engineers might have when designing moment connections.
- Explain how safe moment connection design, at a beam-column connection, depends on understanding if the moment demands arise from gravity or lateral load effects.
- Describe how to detail moment connections to allow for construction tolerances.
- Explain the significance of whether the effect of inelastic panel-zone deformation on frame stability has been considered in the analysis, or not.

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Night School 19
Connection Design:
Tips, Tricks, and Lessons Learned

Session 4: Moment Connections
March 4, 2019

Dr. Patrick J. Fortney, P.E., S.E., P.Eng
Associate Professor
University of Cincinnati
Department of Civil and Architectural Engineering
and Construction Management
Cincinnati, Ohio

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Joint Types

Beam-to-Column Flange
Beam-to-Column Web
HSS Beam-to-Column Flange
HSS Beam-to-HSS Column
One-Sided
Two-Sided
Beam-to-Beam
Equal Depth
Unequal Depth
One-Sided (spandrel)
Two-Sided
Cantilevers

Moment Connections

Direct Welded
Bolted Flange Plate
End Plate
Saddle Plates
Wrap-Around Plates
Embedded

9

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One-Sided (spandrel)
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Cantilevers

Moment Connections

Direct Welded
Bolted Flange Plate
End Plate
Saddle Plates
Wrap-Around Plates
Embedded

etc., etc., etc. …

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Today's Discussion

- All in the context of delegated connection design

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Today's Discussion

- All in the context of delegated connection design
- To understand delegated design work, we need to understand
  - Responsibilities (Larry, Cliff, Pat – Sessions 1, 2, and 3)
  - Work Flow (Larry and Pat (Sessions 1 and 3)
  - Effective Communication (Larry, Cliff, Pat – Sessions 1, 2, and 3)

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Today's Discussion

- All in the context of delegated connection design
- To understand delegated design work, we need to understand
  - Responsibilities (Larry, Cliff, Pat – Sessions 1, 2, and 3)
  - Work Flow (Larry and Pat (Sessions 1 and 3)
  - Effective Communication (Larry, Cliff, Pat – Sessions 1, 2, and 3)
- We'll talk a little about
  - Effective Communication
  - General Detailing
  - Specific Issues Related to Moment Connections

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Today's Discussion

- AISC has offered many webinars and Night Schools on the design of connections
- Dowswell, Fortney, Muir, Murray, Sabelli, Thornton, etc.
  - Specific examples
  - Number crunching
- The SDM and Design Examples manual
- Today, I'd like to look at some detailing and any issues not typically addressed in regard to design in general, but specific to moment connections

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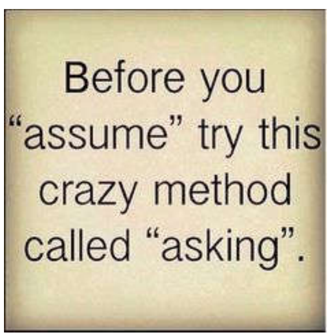
Communication – Unverified Assumptions



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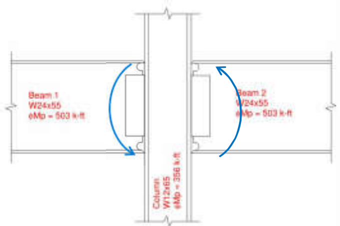

Structural Drawings specify CJP welds at top & bottom beam flanges for fully restrained moment connections

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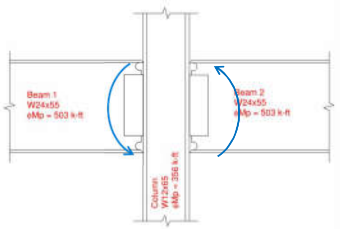
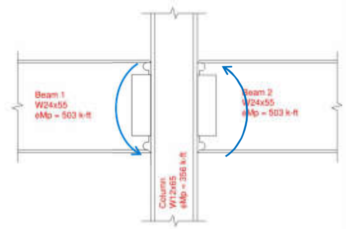


Structural Drawings specify CJP welds at top & bottom beam flanges for fully restrained moment connections

Connection designer assumed beam moments were solely due to lateral load cases such that:
$\Sigma\phi M_{p\_beams} \leq \Sigma\phi M_{p\_columns}$
and designed/detailed the beam-to-column connections for only 356 k-ft.

17



Structural Drawings specify CJP welds at top & bottom beam flanges for fully restrained moment connections

Connection designer assumed beam moments were solely due to lateral load cases such that:
$\Sigma\phi M_{p\_beams} \leq \Sigma\phi M_{p\_columns}$
and designed/detailed the beam-to-column connections for only 356 k-ft.

For this project, that was not a valid assumption!
(example courtesy of Susan Burmeister)

If it's not in the Design Drawings, it doesn't count. Ask for clarification (RFI process) in lieu of trying to second guess the intent…

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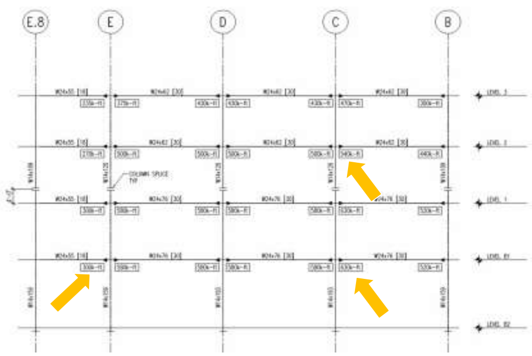

Figure courtesy of Susan Burmeister, P.E.

W24x55:
ϕM_p = 503 k-ft
vs. 300 k-ft

W24x62:
ϕM_p = 574 k-ft
vs. 540 k-ft

W24x76:
ϕM_p = 750 k-ft
vs. 630 k-ft

Member capacity design does not just impact the size of the connection – it has a direct impact on evaluations for continuity and web doubler plates

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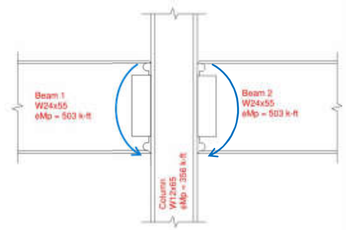

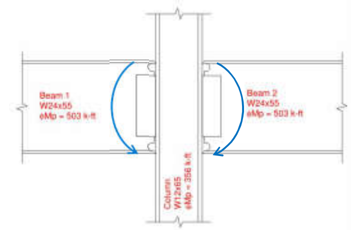

Moral…

…with balanced gravity beam moments, the available flexural strength of the column does not impact the required strength of the moment connection.

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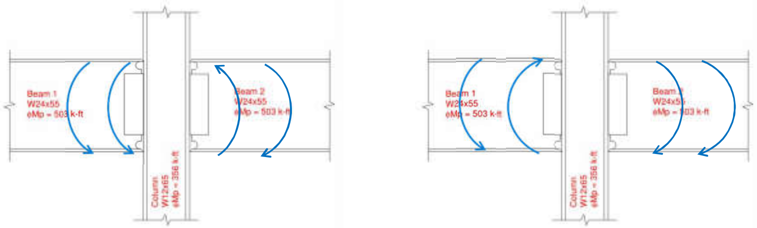
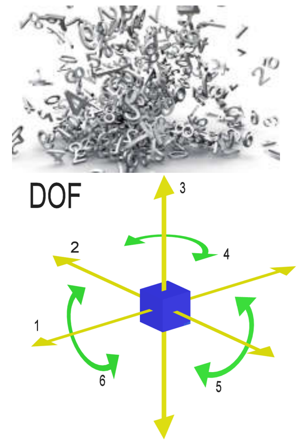


Note that the moment at the face of a beam-column moment joint may be a result of a combination of gravity and lateral…

…this effects the way panel zone shear is calculated; useful information to a connection designer

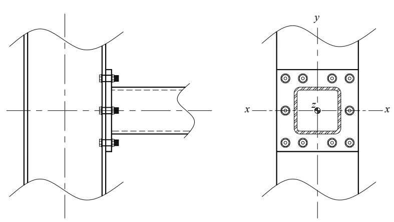
Moment Connections with Combined Loads – Boundary Conditions



Moment Connections with Combined Loads – Boundary Conditions

Joint	A_z	V_x	V_y	M_x	M_y	M_z
Coord.	(kips)	(kips)	(kips)	(k-in)	(k-in)	(k-in)
A-1	35	15	42	95	0.50	0.25
A-2	*	*	*	*	*	*

*Provide available axial, shear, or moment strength of HSS



Moment Connections with Combined Loads – Boundary Conditions

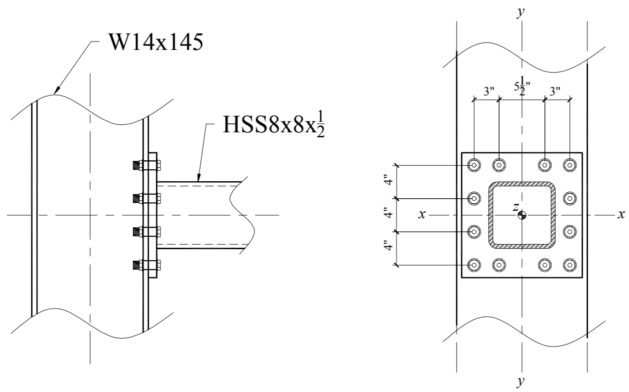
Joint	A_z	V_x	V_y	M_x	M_y	M_z
Coord.	(kips)	(kips)	(kips)	(k-in)	(k-in)	(k-in)
A-1	35	15	42	95	0.50	0.25
A-2	*	*	*	*	*	*

*Provide available axial, shear, or moment strength of HSS

SEoRs: Let's be reasonable…

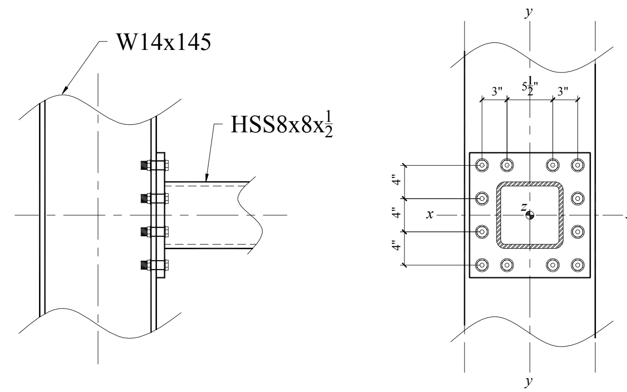
Connection Designers: It's okay to ask…

Moment Connections with Combined Loads – Boundary Conditions



Moment Connections with Combined Loads – Boundary Conditions

Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0



Moment Connections with Combined Loads – Boundary Conditions

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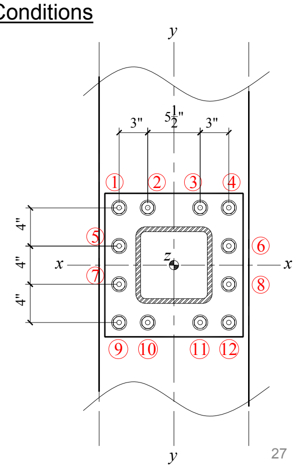


Moment Connections with Combined Loads – Boundary Conditions

Approach 1

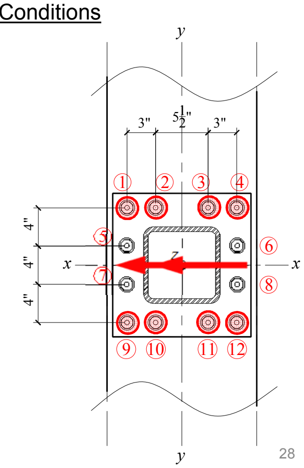
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 1-4 and 9-12 carry M_x *(T)*

e = 12”

$$T = \frac{250 \text{ k-in}}{(12 \text{ in})(4 \text{ bolts})}$$

$$T = 5.21 \text{ kips/bolt}$$



Moment Connections with Combined Loads – Boundary Conditions

Approach 1

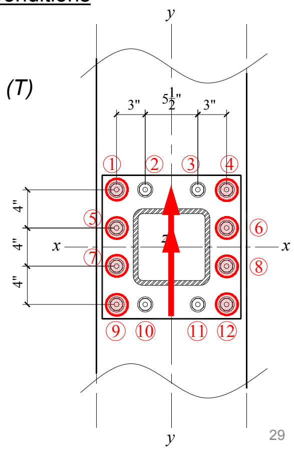
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 1, 4, 5, 6, 7, 8, 9, 12 carry M_y *(T)*

e = 11.5"

$$T = \frac{45.0 \text{ k-in}}{(11.5 \text{ in})(4 \text{ bolts})}$$

$$T = 0.978 \text{ kips/bolt}$$



Moment Connections with Combined Loads – Boundary Conditions

Approach 1

Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 1-4 and 9-12 carry M_z *(V)*

e = 12"

$$V_x = \frac{35.0 \text{ k-in}}{(12 \text{ in})(4 \text{ bolts})}$$

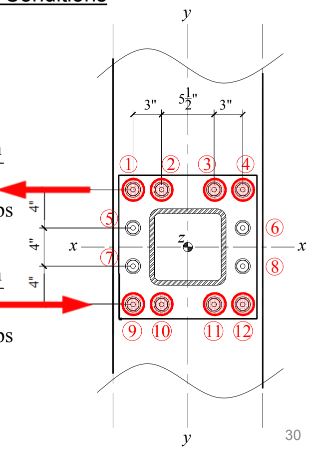
$$V_x = 0.729 \text{ kips/bolt}$$

$$C = \frac{35.0 \text{ k-in}}{(12 \text{ in})}$$

$$C = 2.917 \text{ kips}$$

$$C = \frac{35.0 \text{ k-in}}{(12 \text{ in})}$$

$$C = 2.917 \text{ kips}$$



Moment Connections with Combined Loads – Boundary Conditions

Approach 1

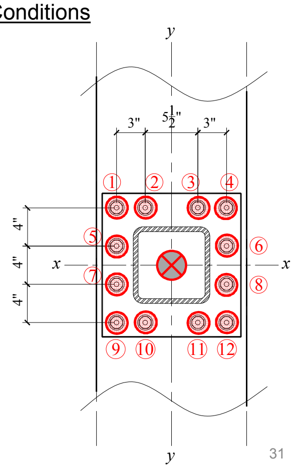
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

All bolts carry *A (T)*

e = n/a

$$T = \frac{15.0 \text{ k}}{12 \text{ bolts}}$$

$$T = 1.25 \text{ kips/bolt}$$



Moment Connections with Combined Loads – Boundary Conditions

Approach 1

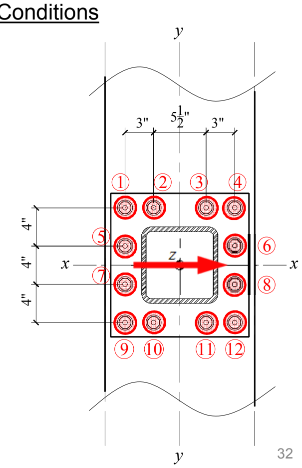
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

All bolts carry V_x *(V)*

e = n/a

$$V_x = \frac{10.0 \text{ k}}{12 \text{ bolts}}$$

$$V_x = 0.833 \text{ kips/bolt}$$



Moment Connections with Combined Loads – Boundary Conditions

Approach 1

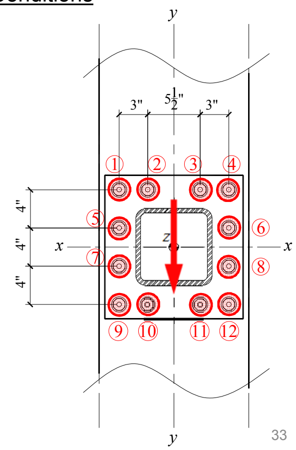
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

All bolts carry V_y (V)

e = n/a

$$V_y = \frac{65.0 \text{ k}}{12 \text{ bolts}}$$

$$V_y = 5.42 \text{ kips/bolt}$$



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Moment Connections with Combined Loads – Boundary Conditions

Approach 1

Bolt Load	Load Type	Load (kips & inches)	e (in)	Qty	1 (kips)	2 (kips)	3 (kips)	4 (kips)	5 (kips)	6 (kips)	7 (kips)	8 (kips)	9 (kips)	10 (kips)	11 (kips)	12 (kips)
Tension	M_x	250.0	12.0	4	5.21	5.21	5.21	5.21	-	-	-	-	5.21	5.21	5.21	5.21
	M_y	45.0	11.5	4	0.978	-	-	0.978	0.978	0.978	0.978	0.978	0.978	-	-	0.978
	A	15.0	n/a	12	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
Sum - Tension					**7.44**	**6.46**	**6.46**	**7.44**	**2.23**	**2.23**	**2.23**	**2.23**	**7.44**	**6.46**	**6.46**	**7.44**
Shear (x)	M_z	35.0	12.0	4	0.729	0.729	0.729	0.729	-	-	-	-	0.729	0.729	0.729	0.729
Shear (x)	V_x	10.0	n/a	12	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833
Sum - Horizontal Shear					**1.56**	**1.56**	**1.56**	**1.56**	**0.83**	**0.83**	**0.83**	**0.83**	**1.56**	**1.56**	**1.56**	**1.56**
Shear (y)	V_y	65.0	n/a	12	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42
Sum - Vertical Shear					**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**
Shear Resultant					**5.64**	**5.64**	**5.64**	**5.64**	**5.48**	**5.48**	**5.48**	**5.48**	**5.64**	**5.64**	**5.64**	**5.64**

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Moment Connections with Combined Loads – Boundary Conditions

Approach 1

Bolt Load	Load Type	Load (kips & inches)	e (in)	Qty	1 (kips)	2 (kips)	3 (kips)	4 (kips)	5 (kips)	6 (kips)	7 (kips)	8 (kips)	9 (kips)	10 (kips)	11 (kips)	12 (kips)
Tension	M_x	250.0	12.0	4	5.21	5.21	5.21	5.21	-	-	-	-	5.21	5.21	5.21	5.21
	M_y	45.0	11.5	4	0.978	-	-	0.978	0.978	0.978	0.978	0.978	0.978	-	-	0.978
	A	15.0	n/a	12	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
Sum - Tension					**7.44**	**6.46**	**6.46**	**7.44**	**2.23**	**2.23**	**2.23**	**2.23**	**7.44**	**6.46**	**6.46**	**7.44**
Shear (x)	M_z	35.0	12.0	4	0.729	0.729	0.729	0.729	-	-	-	-	0.729	0.729	0.729	0.729
Shear (x)	V_x	10.0	n/a	12	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833	0.833
Sum - Horizontal Shear					**1.56**	**1.56**	**1.56**	**1.56**	**0.83**	**0.83**	**0.83**	**0.83**	**1.56**	**1.56**	**1.56**	**1.56**
Shear (y)	V_y	65.0	n/a	12	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42	5.42
Sum - Vertical Shear					**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**	**5.42**
Shear Resultant					**5.64**	**5.64**	**5.64**	**5.64**	**5.48**	**5.48**	**5.48**	**5.48**	**5.64**	**5.64**	**5.64**	**5.64**

This approach requires an evaluation of shear/tension interaction for every bolt

Section J3.7

$$R_n = F'_{nt} A_b$$

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt} \quad \text{(LRFD)}$$

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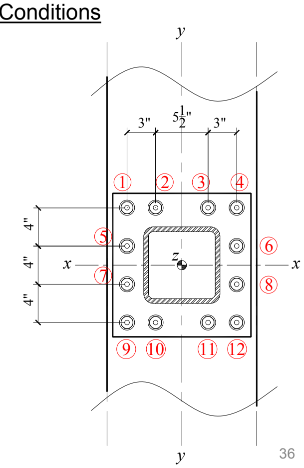
Moment Connections with Combined Loads – Boundary Conditions

Approach 2

Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

No Tension/Shear Interaction



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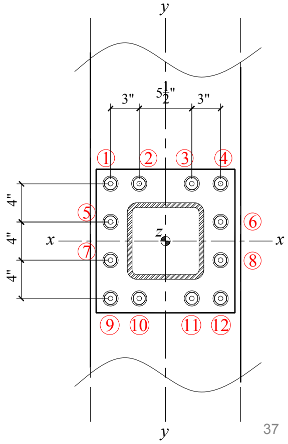
Moment Connections with Combined Loads – Boundary Conditions

Approach 2

Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Let corner bolts carry loads that produce tension,
e.g., M_x, M_y and A

Let other bolts carry loads that produce shear
e.g., V_x, V_y, and M_T



Moment Connections with Combined Loads – Boundary Conditions

Approach 2

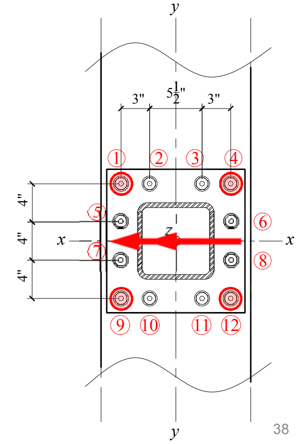
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 1, 4, 9,12 carry M_x *(T)*

e = 12"

$$T = \frac{250 \text{ k-in}}{(12 \text{ in})(2 \text{ bolts})}$$

$$T = 10.4 \text{ kips/bolt}$$



Moment Connections with Combined Loads – Boundary Conditions

Approach 2

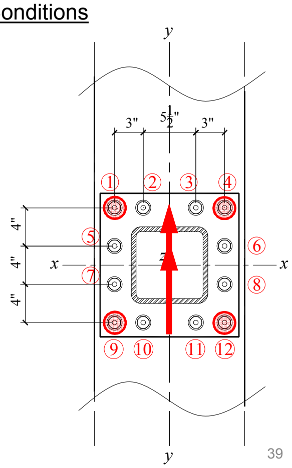
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 1, 4, 9, 12 carry M_y *(T)*

e = 11.5"

$$T = \frac{45.0 \text{ k-in}}{(11.5 \text{ in})(2 \text{ bolts})}$$

$$T = 1.96 \text{ kips/bolt}$$



Moment Connections with Combined Loads – Boundary Conditions

Approach 2

Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 5, 6, 7, 8, M_z *(V)*

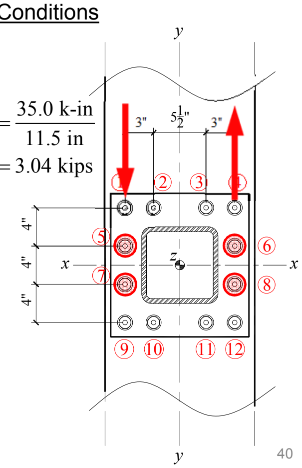
e = 11.5"

$$V_y = \frac{35.0 \text{ k-in}}{(11.5 \text{ in})(2 \text{ bolts})}$$

$$V_y = 1.52 \text{ kips/bolt}$$

$$C = \frac{35.0 \text{ k-in}}{11.5 \text{ in}}$$

$$C = 3.04 \text{ kips}$$



Moment Connections with Combined Loads – Boundary Conditions

Approach 2

Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 5, 6, 7, 8, M_z *(V)*

e = 11.5"

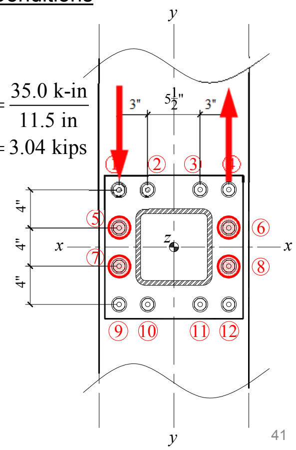
$$V_y = \frac{35.0\text{ k-in}}{(11.5\text{ in})(2\text{ bolts})}$$

$$V_y = 1.52\text{ kips/bolt}$$

Note the manner that M_z is evaluated here (vertical force couple in lieu of horizontal force couple)

$$C = \frac{35.0\text{ k-in}}{11.5\text{ in}}$$

$$C = 3.04\text{ kips}$$



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Moment Connections with Combined Loads – Boundary Conditions

Approach 2

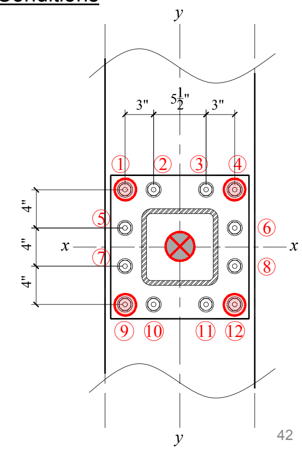
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 1, 4, 9, 12 carry *A (T)*

e = n/a

$$T = \frac{15.0\text{ k}}{4\text{ bolts}}$$

$$T = 3.75\text{ kips/bolt}$$



42

Moment Connections with Combined Loads – Boundary Conditions

Approach 2

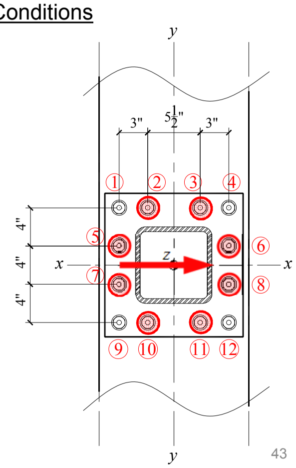
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 2-3, 5-8, 10-11 carry V_x *(V)*

e = n/a

$$V_x = \frac{10.0\text{ k}}{8\text{ bolts}}$$

$$V_x = 1.25\text{ kips/bolt}$$



43

Moment Connections with Combined Loads – Boundary Conditions

Approach 2

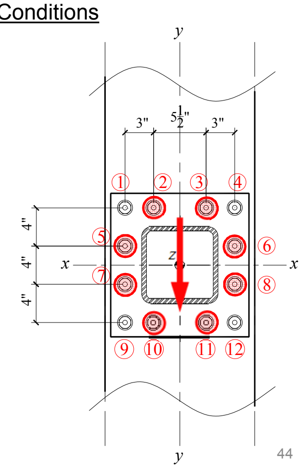
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

Bolts 2-3, 5-8, 10-11 carry V_y *(V)*

e = n/a

$$V_y = \frac{65.0\text{ k}}{8\text{ bolts}}$$

$$V_y = 8.13\text{ kips/bolt}$$



44

Moment Connections with Combined Loads – Boundary Conditions

Approach 2

Bolt Load	Load Type	Load (kips & inches)	e (in)	Qty	1 (kips)	2 (kips)	3 (kips)	4 (kips)	5 (kips)	6 (kips)	7 (kips)	8 (kips)	9 (kips)	10 (kips)	11 (kips)	12 (kips)
Tension	M_x	250.0	12.0	2	10.4	-	-	10.4	-	-	-	-	10.42	-	-	10.4
	M_y	45.0	11.5	2	1.96	-	-	1.96	-	-	-	-	1.96	-	-	1.96
	A	15.0	n/a	4	3.75	-	-	3.75	-	-	-	-	3.75	-	-	3.75
Sum - Tension					**16.1**	**-**	**-**	**16.1**	**-**	**-**	**-**	**-**	**16.1**	**-**	**-**	**16.1**
Shear (x)	V_x	10.0	n/a	8	-	1.25	1.25	-	1.25	1.25	1.25	1.25	-	1.25	1.25	-
Sum - Horizontal Shear					**-**	**1.25**	**1.25**	**-**	**1.25**	**1.25**	**1.25**	**1.25**	**-**	**1.25**	**1.25**	**-**
Shear (y)	V_y	65.0	n/a	8	-	8.13	8.13	-	8.13	8.13	8.13	8.13	-	8.13	8.13	-
Shear (y)	M_z	35.0	11.5	2	-	-	-	-	1.52	1.52	1.52	1.52	-	-	-	-
Sum - Vertical Shear					**-**	**8.13**	**8.13**	**-**	**9.65**	**9.65**	**9.65**	**9.65**	**-**	**8.13**	**8.13**	**-**
Shear Resultant					**-**	**8.22**	**8.22**	**-**	**9.73**	**9.73**	**9.73**	**9.73**	**-**	**8.22**	**8.22**	**-**

45

Moment Connections with Combined Loads – Boundary Conditions

Approach 2

Bolt Load	Load Type	Load (kips & inches)	e (in)	Qty	1 (kips)	2 (kips)	3 (kips)	4 (kips)	5 (kips)	6 (kips)	7 (kips)	8 (kips)	9 (kips)	10 (kips)	11 (kips)	12 (kips)
Tension	M_x	250.0	12.0	2	10.4	-	-	10.4	-	-	-	-	10.42	-	-	10.4
	M_y	45.0	11.5	2	1.96	-	-	1.96	-	-	-	-	1.96	-	-	1.96
	A	15.0	n/a	4	3.75	-	-	3.75	-	-	-	-	3.75	-	-	3.75
Sum - Tension					**16.1**	**-**	**-**	**16.1**	**-**	**-**	**-**	**-**	**16.1**	**-**	**-**	**16.1**
Shear (x)	V_x	10.0	n/a	8	-	1.25	1.25	-	1.25	1.25	1.25	1.25	-	1.25	1.25	-
Sum - Horizontal Shear					**-**	**1.25**	**1.25**	**-**	**1.25**	**1.25**	**1.25**	**1.25**	**-**	**1.25**	**1.25**	**-**
Shear (y)	V_y	65.0	n/a	8	-	8.13	8.13	-	8.13	8.13	8.13	8.13	-	8.13	8.13	-
Shear (y)	M_z	35.0	11.5	2	-	-	-	-	1.52	1.52	1.52	1.52	-	-	-	-
Sum - Vertical Shear					**-**	**8.13**	**8.13**	**-**	**9.65**	**9.65**	**9.65**	**9.65**	**-**	**8.13**	**8.13**	**-**
Shear Resultant					**-**	**8.22**	**8.22**	**-**	**9.73**	**9.73**	**9.73**	**9.73**	**-**	**8.22**	**8.22**	**-**

Shear/tension interaction is not required for any bolt

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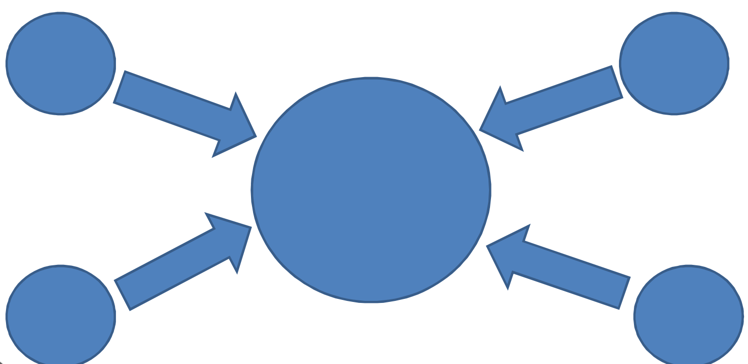
Transfer Forces and Collectors



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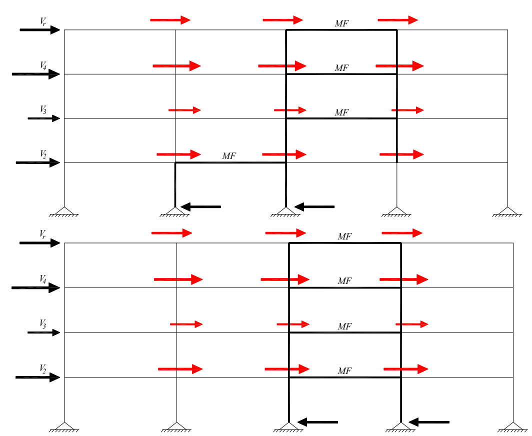
Transfer Forces



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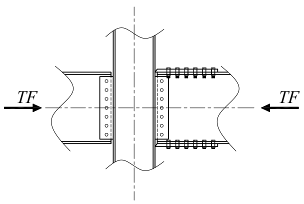
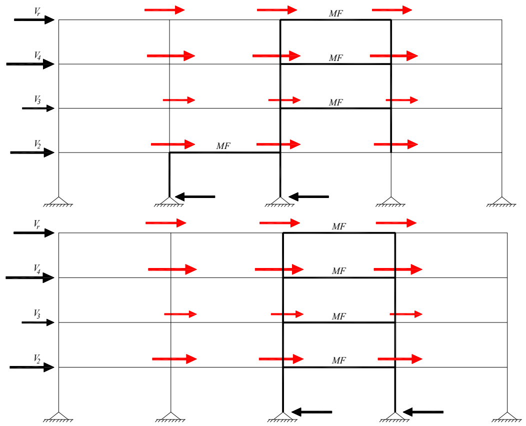
Transfer Forces



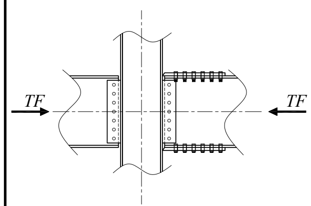


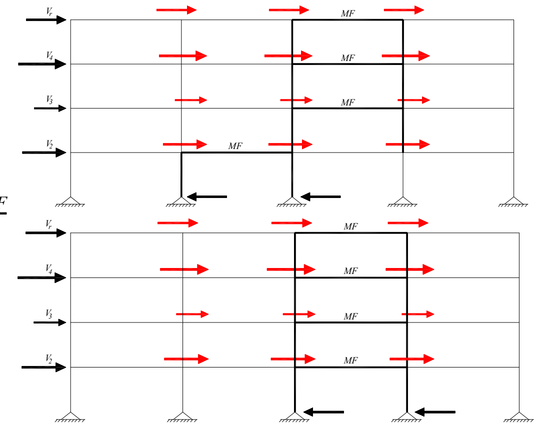
49

Transfer Forces

Rarely reported

- Impacts design of web connection and/or flange connections



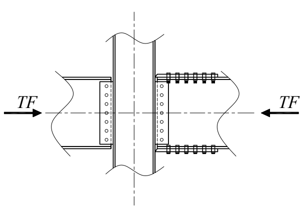


50

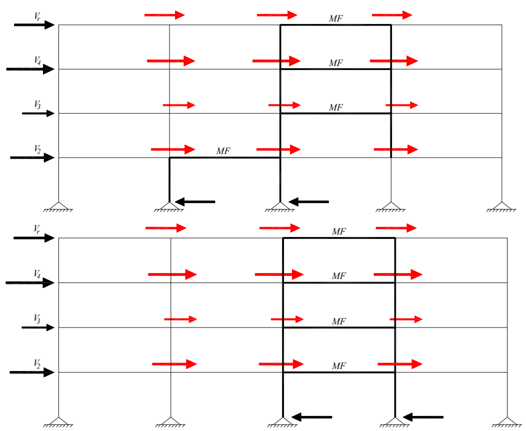
Transfer Forces

Rarely reported

- Impacts design of web connection and/or flange connections



Collectors/drags in general



51

Poll Question 1

Q: The design strength of a beam-to-column moment connection is determined based on the relative available flexural strengths of the beams and column.

a) True
b) False
c) It depends on source of the moment (gravity or lateral)
d) Only if the source of the moments is gravity load only

52

Poll Question 1

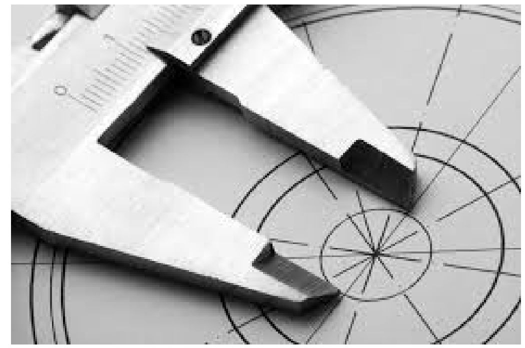
Q: The design strength of a beam-to-column moment connection is determined based on the relative available flexural strengths of the beams and column.

a) True
b) False
c) It depends on source of the moment (gravity or lateral)
d) Only if the source of the moments is gravity load only

Select your answer!

53

Erection and Mill Tolerances



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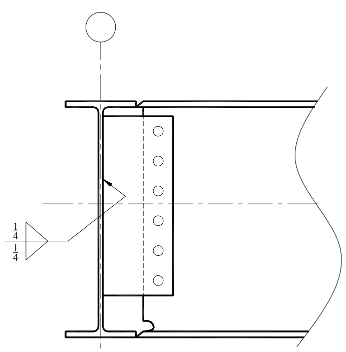
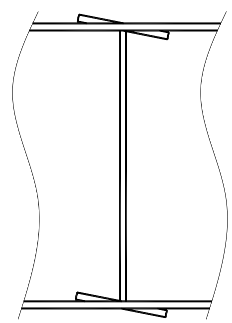
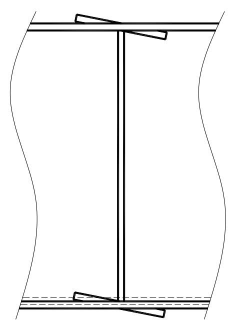


55

Potential Welding Issues



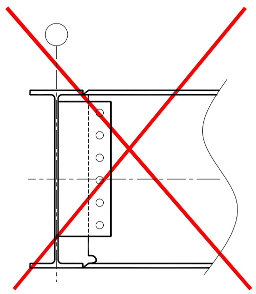
Flange Tilt



Flange Tilt w/ Under/Overroll

56

Possible Options



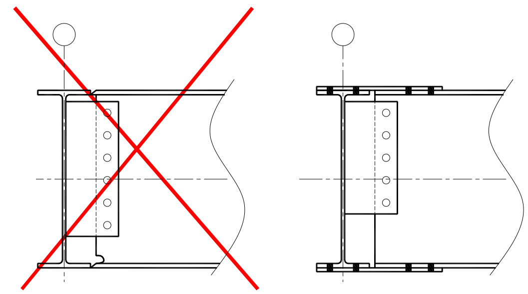
Moment Connection

Option 1

Option 2

57

Possible Options

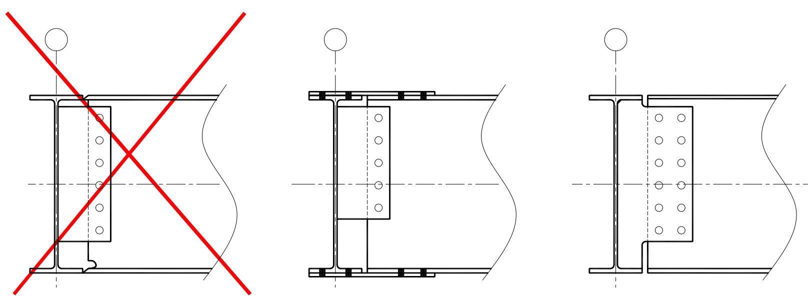


Moment Connection

Option 1

58

Possible Options



Moment Connection

Option 1

Option 2

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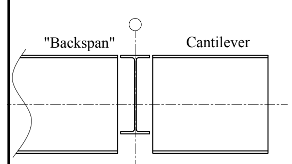
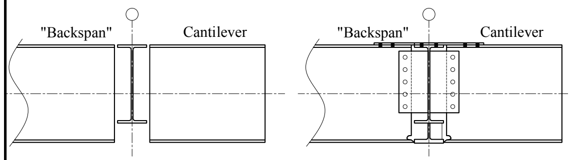
Shallow Supporting Members



Beam-to-Beam

60

Shallow Supporting Members

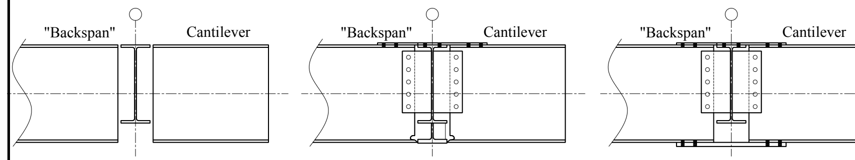


Beam-to-Beam

Flange Plate and Tee

61

Shallow Supporting Members



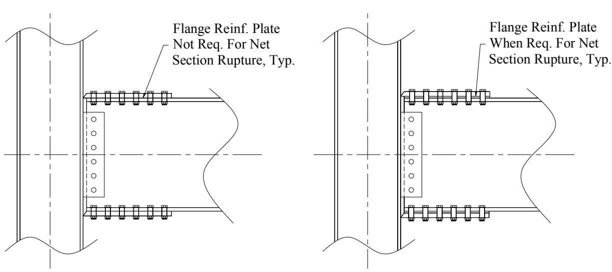
Beam-to-Beam

Flange Plate and Tee

Flange Plates

62

Account for Bolting – Limits on Stress Ratios



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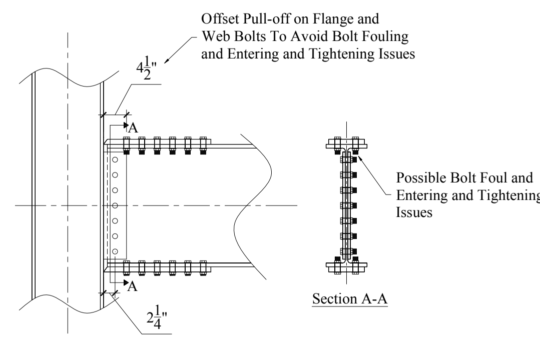
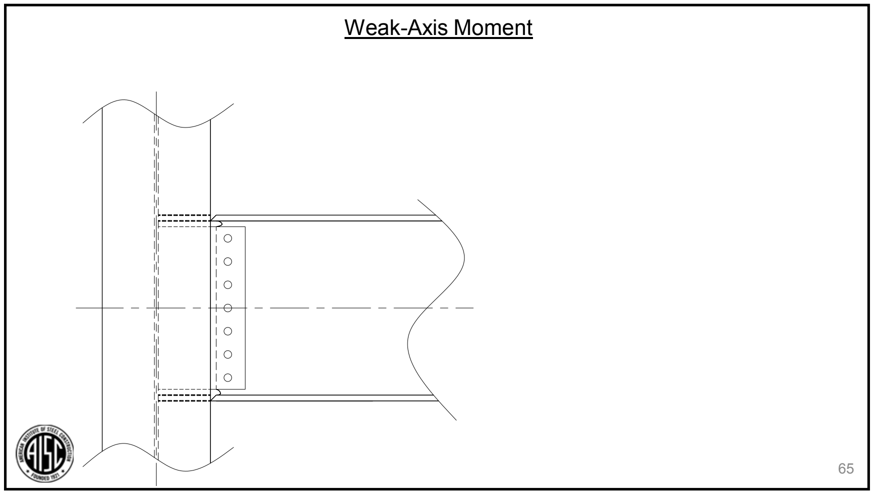
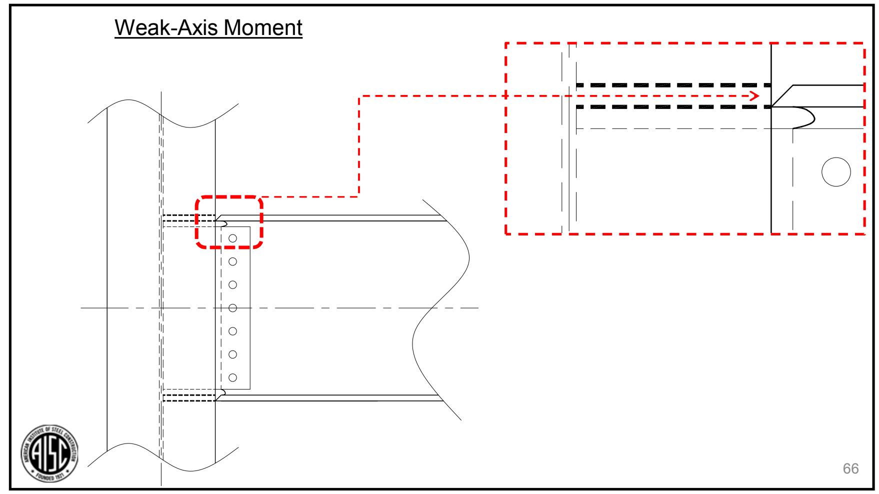
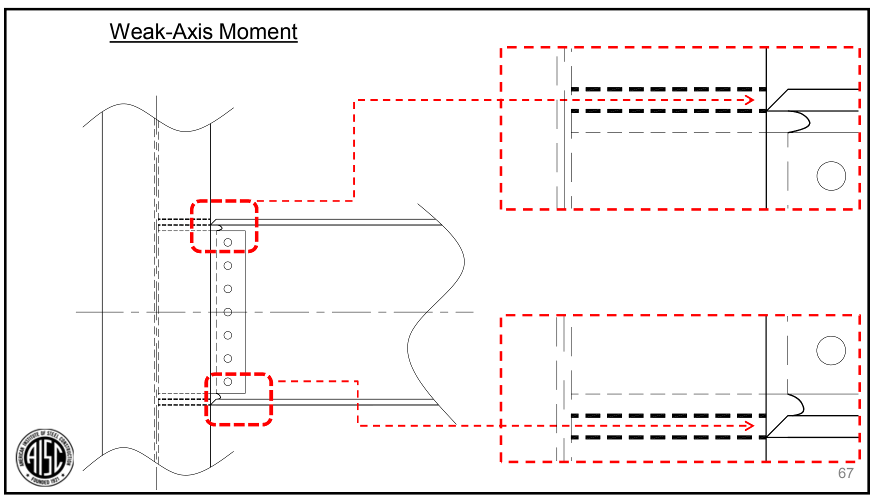
Entering and Tightening

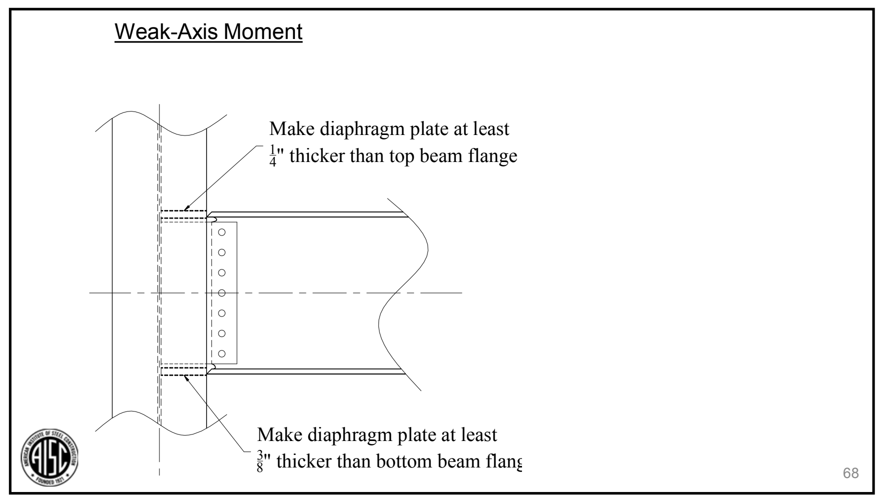


64

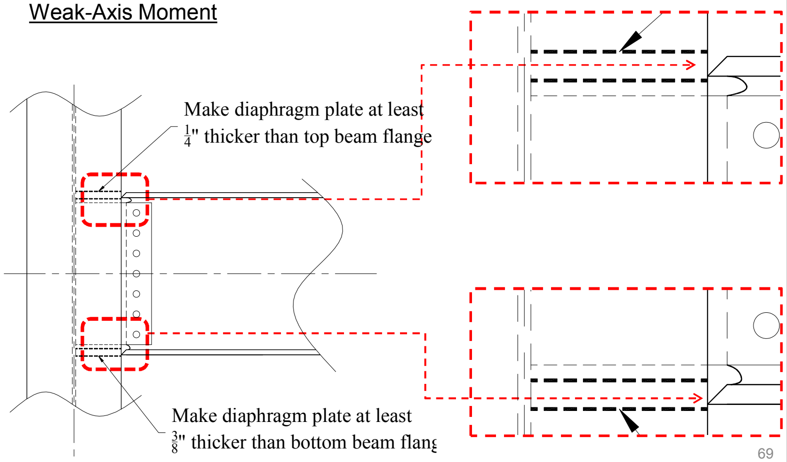

Weak-Axis Moment
65


Weak-Axis Moment
66


Weak-Axis Moment
67


Weak-Axis Moment
Make diaphragm plate at least
$\frac{1}{4}$" thicker than top beam flange
Make diaphragm plate at least
$\frac{3}{8}$" thicker than bottom beam flang
68

Weak-Axis Moment



69

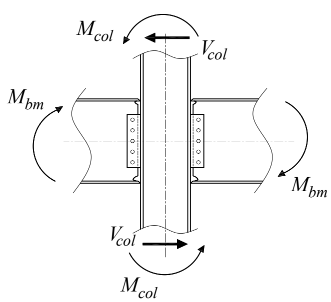
Panel Zones and Column Stiffening

W14x90 Column with 1.5” thick continuity plates

Note distortion in column flanges due to welding

70

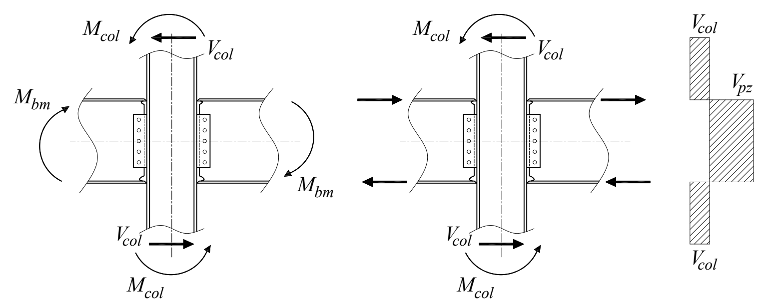
Two-Sided – Lateral Moments



It’s a mistake to assume that panel zone shear only occurs with lateral moments

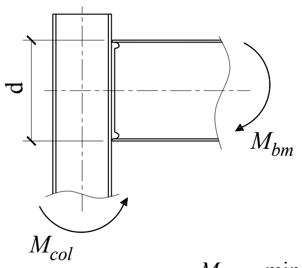
71

Two-Sided – Lateral Moments



72

One-Sided – Gravity or Lateral – T/Column



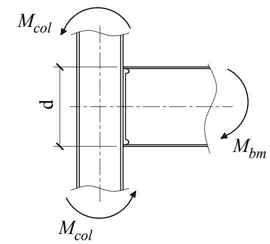
$M_{stiff} = \min(M_{bm},\ M_{col})$

$M_{dbl} = \min(M_{bm},\ M_{col})$
$V_{dbl} = M_{dbl} / d$

Stiffeners Doublers

73

One-Sided – Gravity or Lateral – Column Midspan



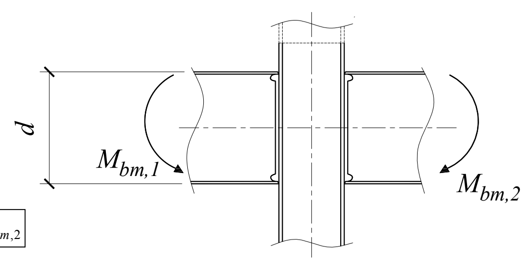
$M_{stiff} = \min(M_{bm},\ 2M_{col})$

$M_{dbl} = \min(M_{bm},\ 2M_{col})$
$V_{dbl} = M_{dbl} / d$

Stiffeners Doublers

74

Two-Sided – Gravity – Equal Beam Moments



$M_{bm,1} = M_{bm,2}$

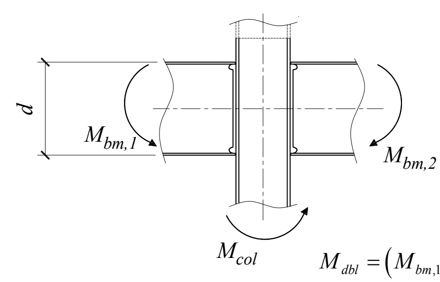
$M_{stiff} = M_{bm,1}$
$M_{stiff} = M_{bm,2}$

No Panel Shear

Stiffeners Doublers

75

Two-Sided – Gravity – Equal Beam Moments



$M_{bm,1} > M_{bm,2}$

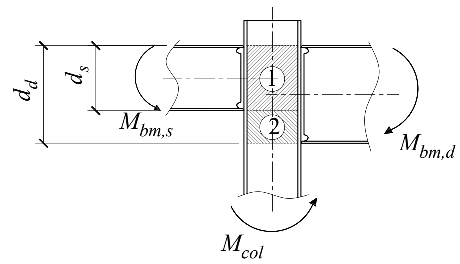
$M_{stiff} = \min(M_{bm,1} - M_{bm,2},\ M_{col})$

$M_{dbl} = (M_{bm,1} - M_{bm,2}) / d$
$V_{dbl} = M_{dbl} / d$

Stiffeners Doublers

76

Two-Sided – Gravity or Lateral – T/Column

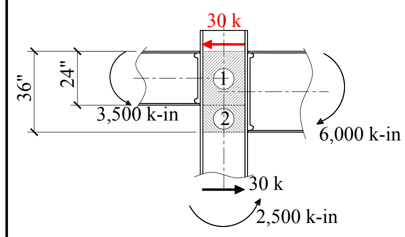


Unequal depth beams can induce more than one panel zone

A FBD of the joint should be generated to understand the demand on the column flanges and web

77

Multiple Panel Zones



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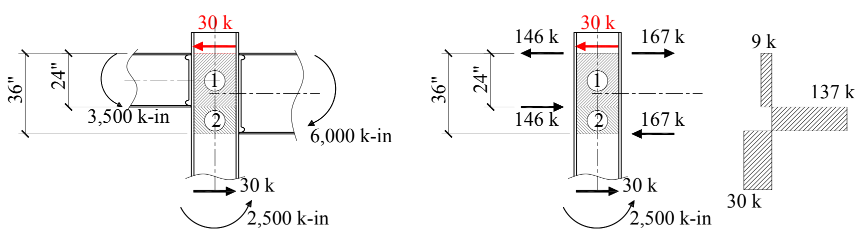
Multiple Panel Zones



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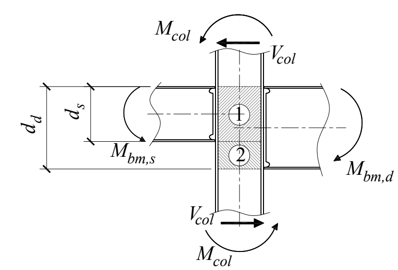
Two-Sided – Gravity or Lateral –Column Mid-Span



Unequal depth beams can induce more than one panel zone

A FBD of the joint should be generated to understand the demand on the column flanges and web

80

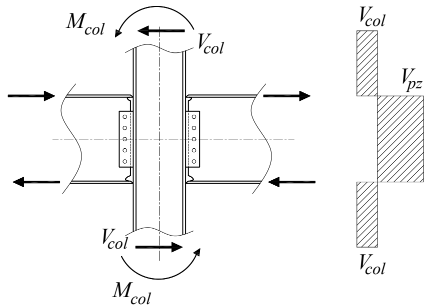
Panel Zone Strength

Section J10.6

(a) …panel zone deformation is not considered…

$\alpha P_r \leq 0.4 P_y$

$R_n = 0.60 F_y d_c t_w$ (EQ. J10.9)



81

Panel Zone Strength

Section J10.6

(a) …panel zone deformation is not considered…

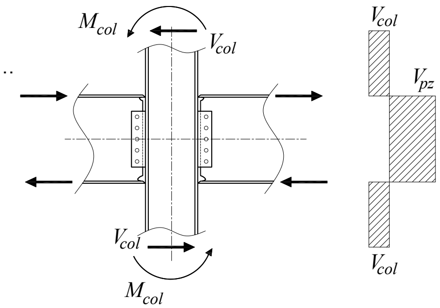
$\alpha P_r \leq 0.4 P_y$

$R_n = 0.60 F_y d_c t_w$ (EQ. J10.9)

(b) …panel zone deformation is considered…

$\alpha P_r \leq 0.75 P_y$

$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)$ (EQ. J10.11)



The parans term accounts for additional shear strength in columns with thicker flanges and inelastic panel zone deformation (up to three times the deformation at panel zone yielding)

82

Panel Zone Strength

Section J10.6

(a) …panel zone deformation is not considered…

$\alpha P_r \leq 0.4 P_y$

$R_n = 0.60 F_y d_c t_w$ (EQ. J10.9)

(b) …panel zone deformation is considered…

$\alpha P_r \leq 0.75 P_y$

$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)$ (EQ. J10.11)

- Research (e.g., SAC) shows that when centerline models are used for analysis with no rigid offsets, it is reasonable to assume that effects of panel zone deformation has been considered.
- The contribution of additional term in J10.11 is relatively less in thinner flanged members and in deeper column sections (i.e., deeper webs)

83

Panel Zone Strength

Section J10.6

(a) …panel zone deformation is not considered…

$\alpha P_r \leq 0.4 P_y$

$R_n = 0.60 F_y d_c t_w$ (EQ. J10.9)

(b) …panel zone deformation is considered…

$\alpha P_r \leq 0.75 P_y$

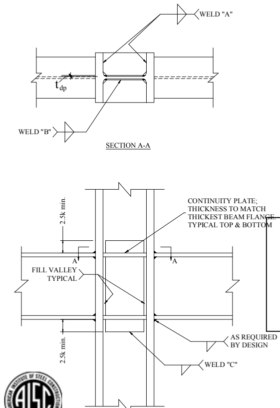
$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)$ (EQ. J10.11)

Column Size	t_{fc} (inches)	*Approximate Strength Increase from with J10.11
W14x730	4.91	50%
W14x370	2.66	35%
W14x145	1.09	20%
W14x26	0.42	5%
W24x370	2.72	20%
W24x146	1.09	10%
W24x62	0.59	3%

*Varies based on beam depth

84

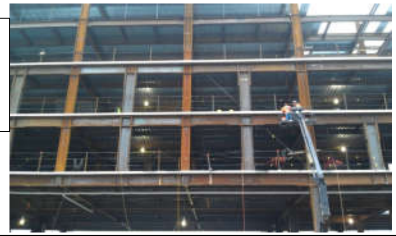
Two-Sided Connection with Continuity and Web Doubler Plates



BEAM	COLUMN	WP t_p (in)	CP	WELD "A" (16^{ths})	WELD "B" (16^{ths})	WELD "C" (16^{ths})
W16X26	W14x90	(2)-x	Y	x	x	x
W16X26	W14x132	-	-	-	-	-
W18X71	W14x90	(2)-x"	Y	x	x	x
W18X71	W14x132	-	Y	x	x	-

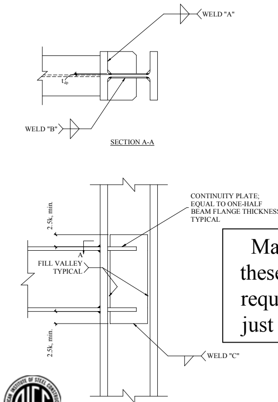
Note that the information provided should include interaction for "shared columns."

Make clear if these are design requirements or just for bidding



85

One-Sided Connection with Continuity and Web Doubler Plates



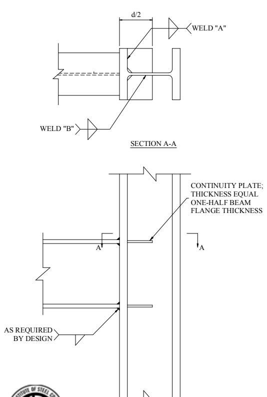
BEAM	COLUMN	WP t_p (in)	CP	WELD "A" (16^{ths})	WELD "B" (16^{ths})	WELD "C" (16^{ths})
W16X26	W14x90	(2)-x	Y	x	x	x
W16X26	W14x132	-	-	-	-	-
W18X71	W14x90	(2)-x"	Y	x	x	x
W18X71	W14x132	-	Y	x	x	-

Note that the information provided should include interaction for "shared columns."

Make clear if these are design requirements or just for bidding

86

One-Sided Connections with Continuity Plates – No Web Doublers



BEAM	COLUMN	WP t_p (in)	CP	WELD "A" (16^{ths})	WELD "B" (16^{ths})	WELD "C" (16^{ths})
W16X26	W14x90	(2)-x	Y	x	x	x
W16X26	W14x132	-	-	-	-	-
W18X71	W14x90	(2)-x"	Y	x	x	x
W18X71	W14x132	-	Y	x	x	-

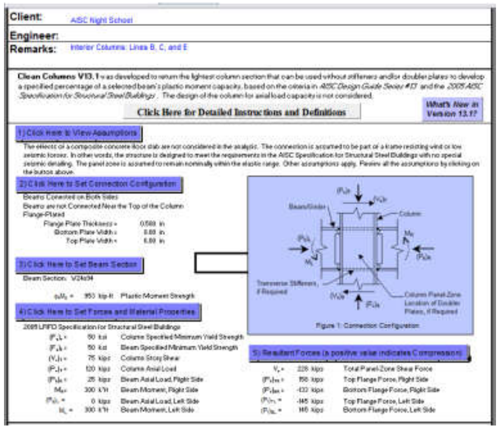
Make clear if these are design requirements or just for bidding

87

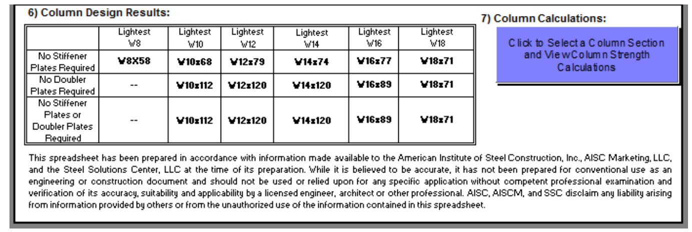
Clean Column Worksheet

https://www.steeltools.org/

88



89



6) Column Design Results:

	Lightest W8	Lightest W10	Lightest W12	Lightest W14	Lightest W16	Lightest W18
No Stiffener Plates Required	W8X58	W10x68	W12x79	W14x74	W16x77	W18x71
No Doubler Plates Required	--	W10x112	W12x120	W14x120	W16x89	W18x71
No Stiffener Plates or Doubler Plates Required	--	W10x112	W12x120	W14x120	W16x89	W18x71

7) Column Calculations:

Click to Select a Column Section and View Column Strength Calculations

This spreadsheet has been prepared in accordance with information made available to the American Institute of Steel Construction, Inc., AISC Marketing, LLC, and the Steel Solutions Center, LLC at the time of its preparation. While it is believed to be accurate, it has not been prepared for conventional use as an engineering or construction document and should not be used or relied upon for any specific application without competent professional examination and verification of its accuracy, suitability and applicability by a licensed engineer, architect or other professional. AISC, AISCM, and SSC disclaim any liability arising from information provided by others or from the unauthorized use of the information contained in this spreadsheet.

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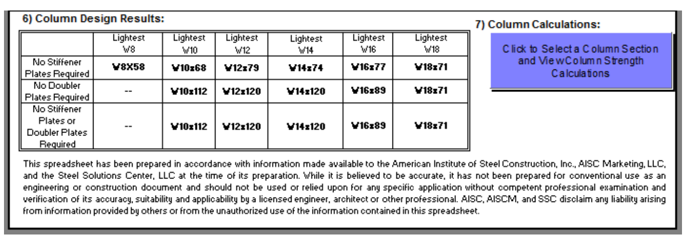
Most commercially available software evaluates stiffening requirements of wide flange columns



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No Stiffener Plates or Doubler Plates Required	--	W10x112	W12x120	W14x120	W16x89	W18x71

7) Column Calculations:

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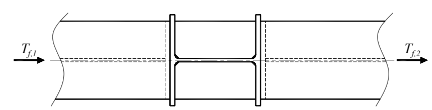
If not, I strongly recommend this worksheet!

91

A Word on Continuity Plate Welds

Consider the lateral moment connection shown…



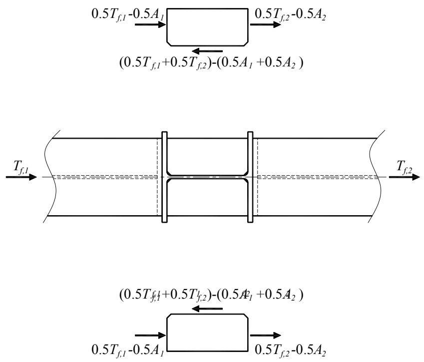
92

A Word on Continuity Plate Welds

Consider the lateral moment connection shown…

$T_{f,i}$ = Tension Flange Force

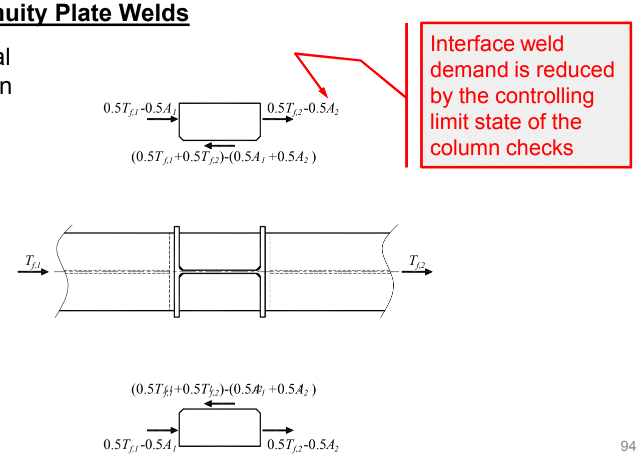
A_i = Controlling Limit State for C.P. Checks



93

A Word on Continuity Plate Welds

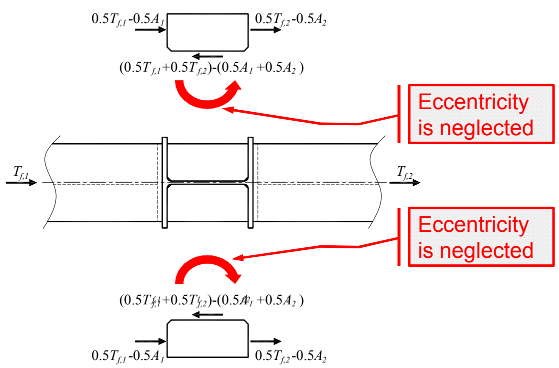
Consider the lateral moment connection shown…



94

A Word on Continuity Plate Welds

Consider the lateral moment connection shown…



95

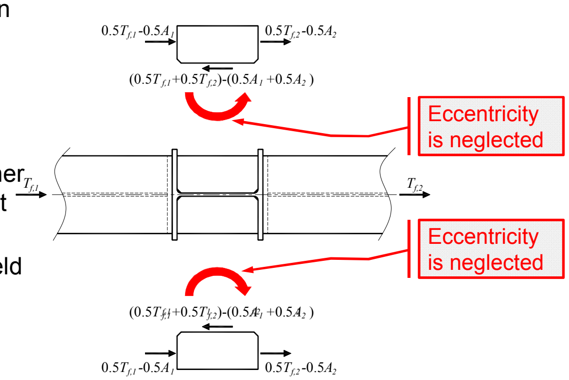
A Word on Continuity Plate Welds

Consider the lateral moment connection shown…

The effect of *e* is negligible…

…several ways to account for this other than the moment at the web…

…no effect final weld size

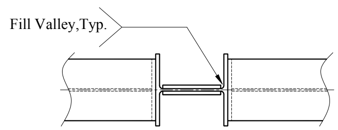


96

A Word on Double Plate Welds

"Fill Valley" Welds

CJP???
Mag. Particle Testing???



97

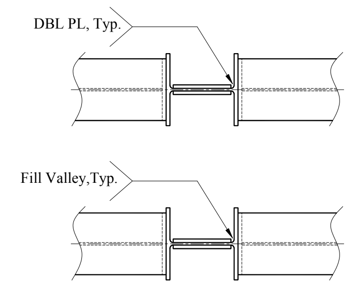
A Word on Double Plate Welds

"Fill Valley" Welds

CJP???
Mag. Particle Testing???

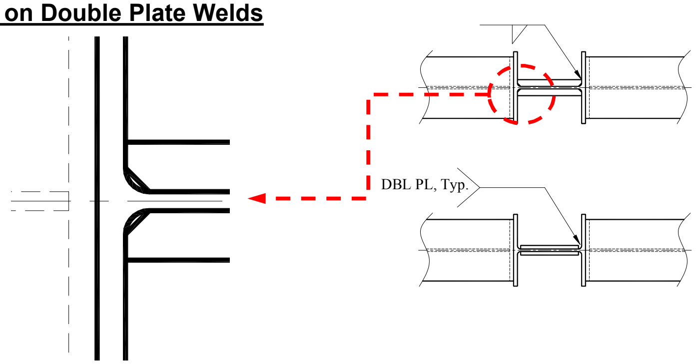
Addressed in AWS D1.8 2016…

…but really applies to all dbl. plate welds



98

A Word on Double Plate Welds



Increase plate thickness to account for bevel

99

Poll Question 2

Q: The welds used to install continuity plates in wide flange columns are typically based on an admissible and equilibrated force distribution on the continuity plate FBD.

a) True
b) False

100

Poll Question 2

Q: The welds used to install continuity plates in wide flange columns are typically based on an admissible and equilibrated force distribution on the continuity plate FBD.

a) True
b) False

Select your answer!

101

Moment Connections

AISC | Questions?

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Individual 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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Individual 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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8-Session Registrants

CEU / PDH Certificates

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

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8-Session Registrants

Access to the quiz

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

Quiz and attendance records

Posted Tuesday mornings. www.aisc.org/nightschool -- Click on Current Course Details.

Reasons for quiz

- EEU – You must take all quizzes and the final exam to receive EEU.
- CEUs/PDHs – If you watch a recorded session, you must pass quiz for CEUs/PDHs.
- REINFORCEMENT – Reinforce what you learn 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

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8-Session Registrants

Access to the recording

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

CEUs / PDHs via recording

If you watch a recorded session, you must take *and pass* the quiz for CEUs/PDHs.

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8-Session Registrants

Night School Resources

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

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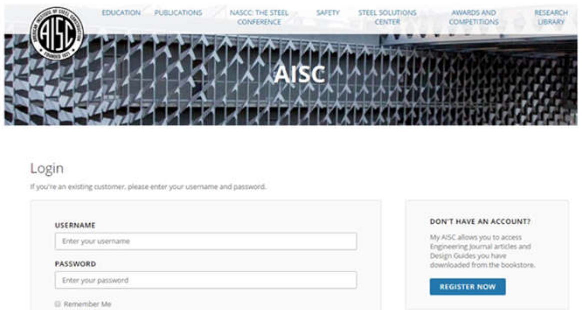
8-Session Registrants

Night School Resources

Go to www.aisc.org and sign in.



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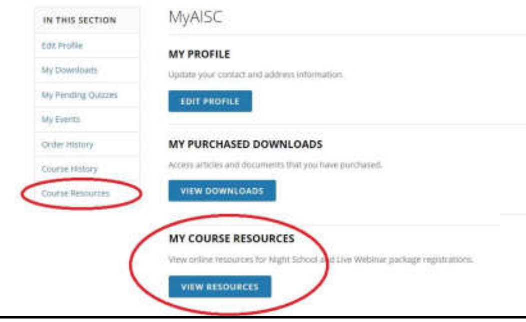
8-Session Registrants

Night School Resources

Go to www.aisc.org and sign in.



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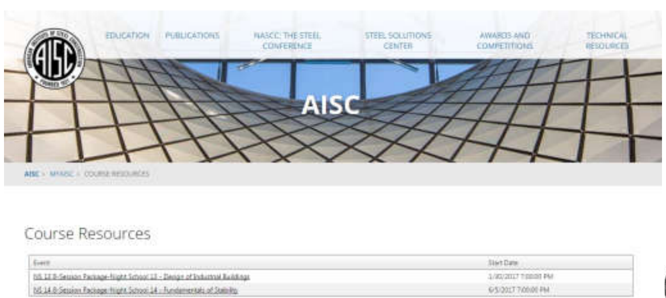
8-Session Registrants

Night School Resources



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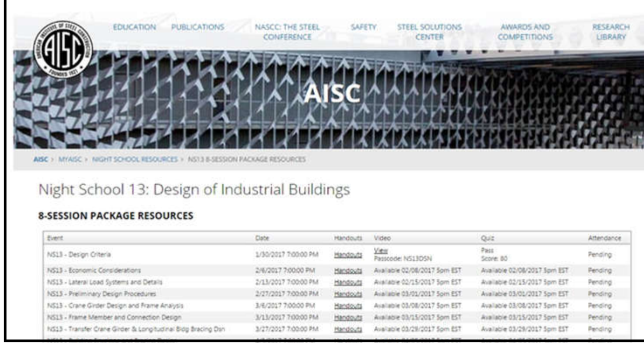
8-Session Registrants

Night School Resources



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Night School Resources

- Weekly "quiz and recording" email.
- Weekly updates of the master quiz and attendance record, found at www.aisc.org/nightschool19. Scroll down to Quiz and Attendance records.
  - Updated on Tuesday mornings.

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8-Session Registrants

Night School Resources

- Webinar connection information
  - Found in your registration confirmation / receipt
  - Reminder email sent out Monday mornings
- Links to handouts also found here

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AISC | Thank you

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