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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.





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.



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



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



Joint Types

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Moment Connections

- Direct Welded
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- Embedded

etc., etc., etc. ...



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

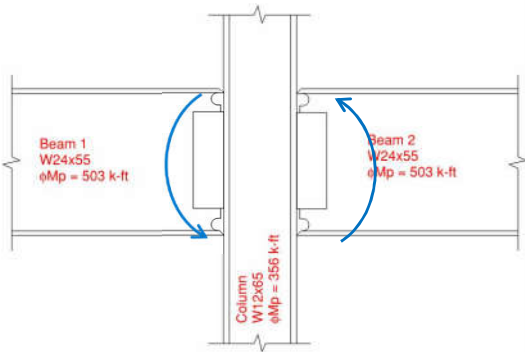
- AISC has offered many webinars and Night Schools on the design of connections
- Dowswell, Fortney, Muir, Murray, Sabelli, Thornton, etc.
 - Specific examples
 - Number crunching
- The SDM and Design Examples manual
- Today, I'd like to look at some detailing and any issues not typically addressed in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



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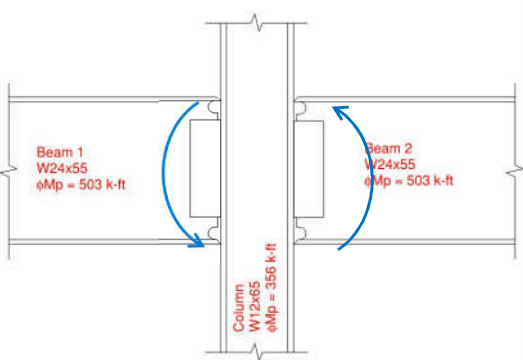
Communication – Unverified Assumptions

Before you
“assume” try this
crazy method
called “asking”.




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



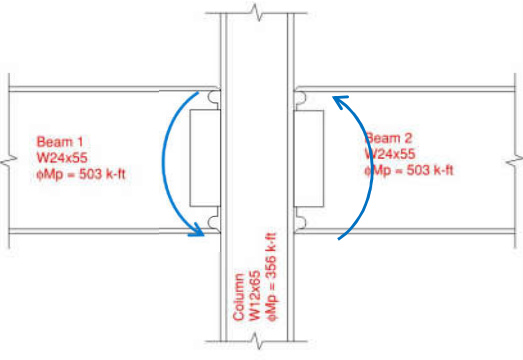


Structural Drawings specify CJP welds at top & bottom beam flanges for fully restrained moment connections

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.



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

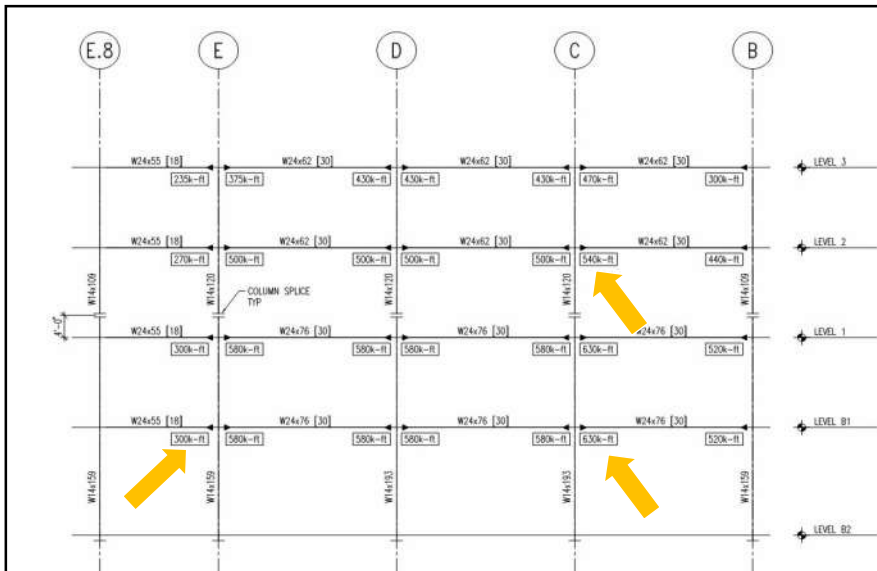
Connection designer assumed beam moments were solely due to lateral load cases such that:
 $\Sigma\phi M_{p_beams} \leq \Sigma\phi M_{p_columns}$
and designed/detailed the beam-to-column connections for only 356 k-ft.

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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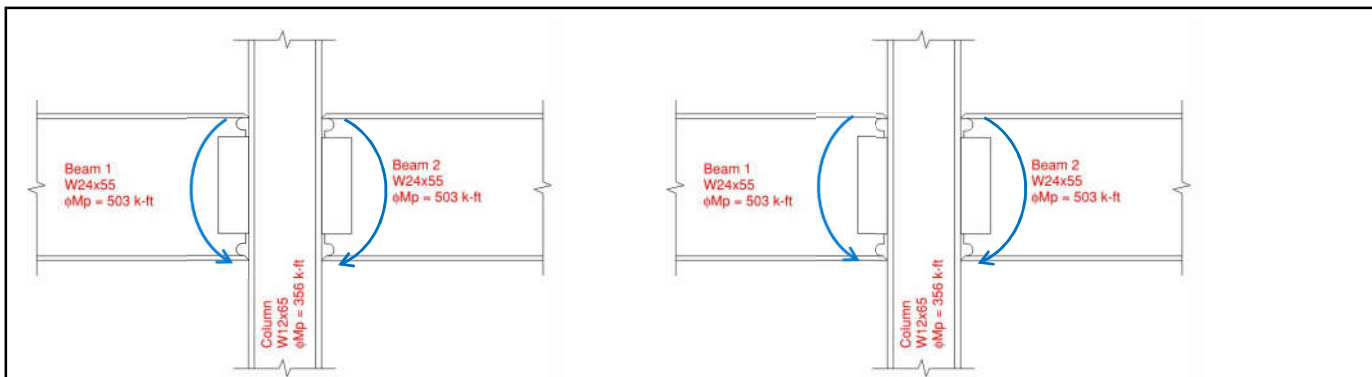
W24x55:
 $\phi M_p = 503$ k-ft
 vs. 300 k-ft

W24x62:
 $\phi M_p = 574$ k-ft
 vs. 540 k-ft

W24x76:
 $\phi M_p = 750$ k-ft
 vs. 630 k-ft

Figure courtesy of Susan Burmeister, P.E.

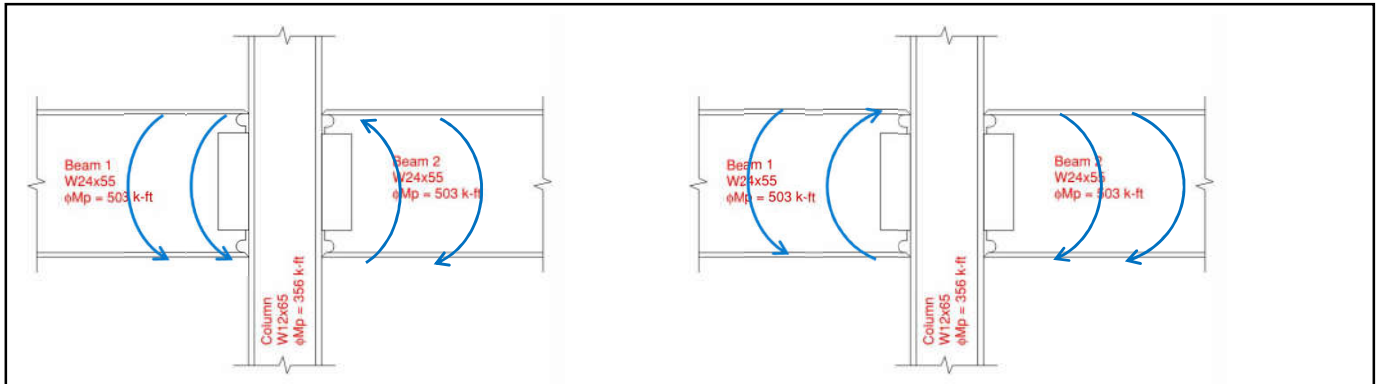
Member capacity design does not just impact the size of the connection – it has a direct impact on evaluations for continuity and web doubler plates



Moral...

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



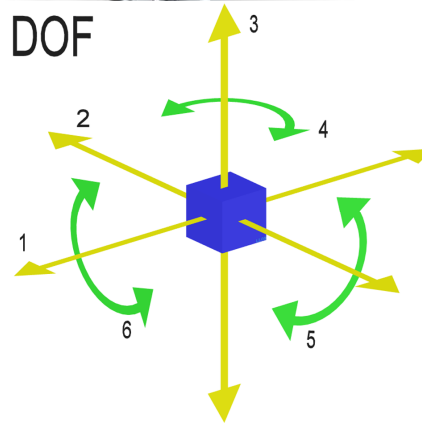


Note that the moment at the face of a beam-column moment joint may be a result of a combination of gravity and lateral...

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



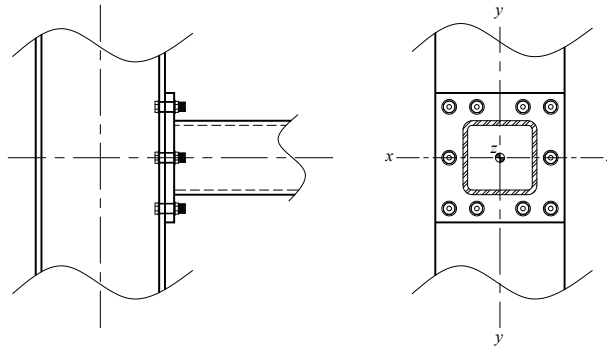
Moment Connections with Combined Loads – Boundary Conditions



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	*	*	*	*	*	*

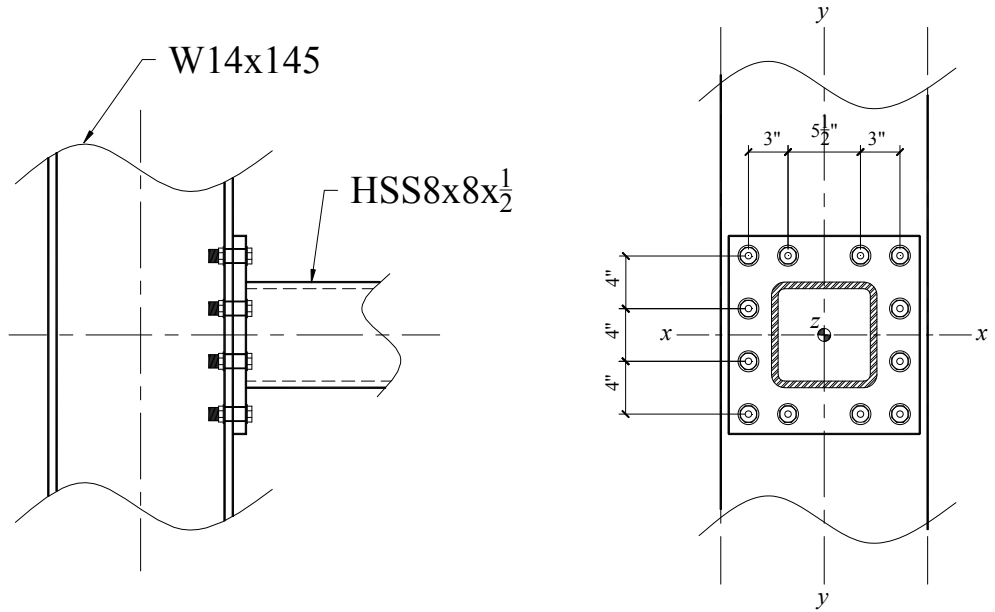
*Provide available axial, shear, or moment strength of HSS

SEoRs: Let's be reasonable...

Connection Designers: It's okay to ask...

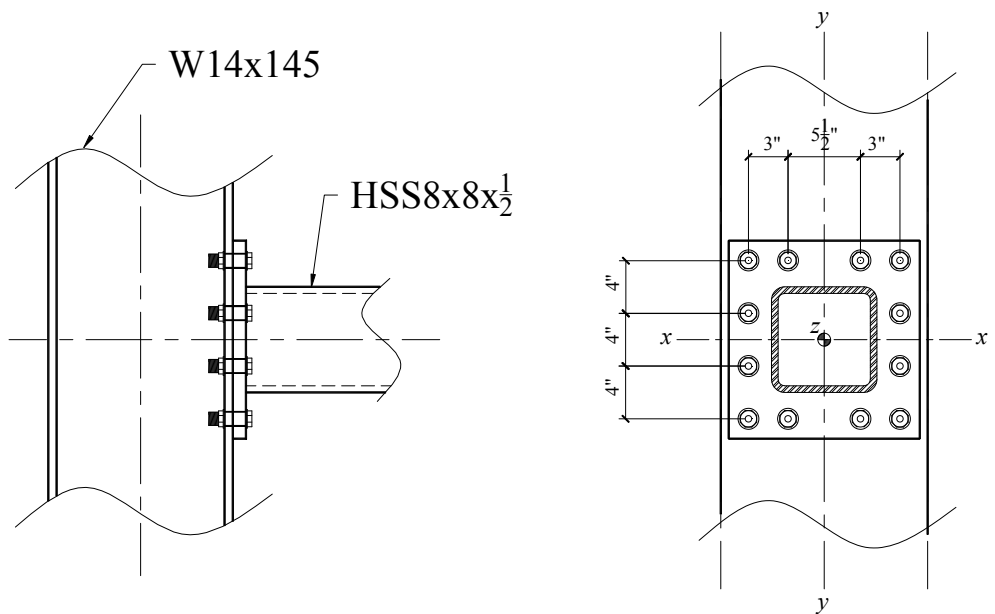


Moment Connections with Combined Loads – Boundary Conditions



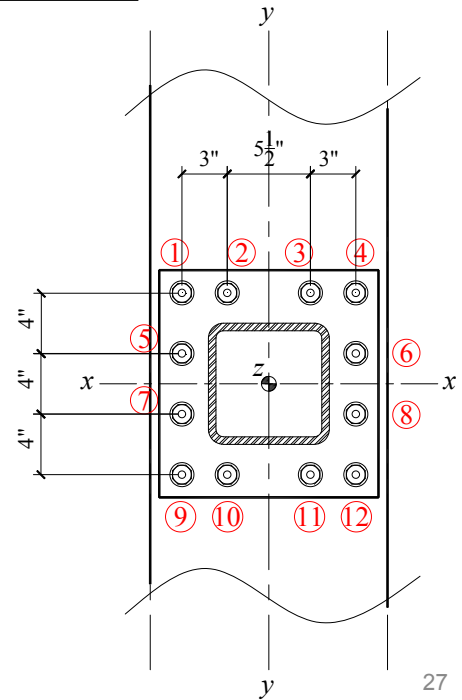
Moment Connections with Combined Loads – Boundary Conditions

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

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

Approach 1

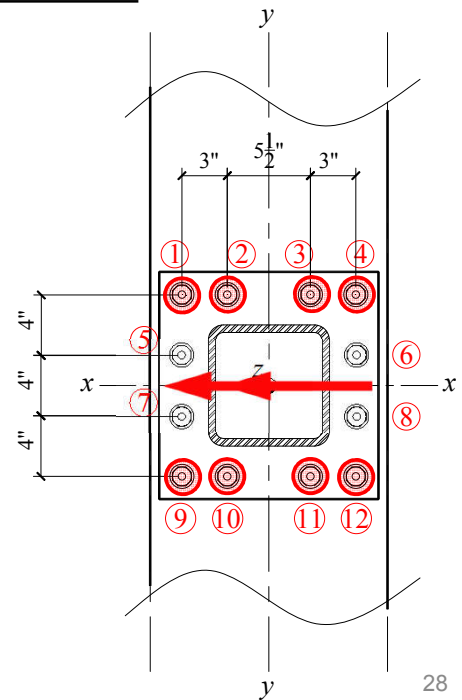
Load	Magnitude (+/-)
	(kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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

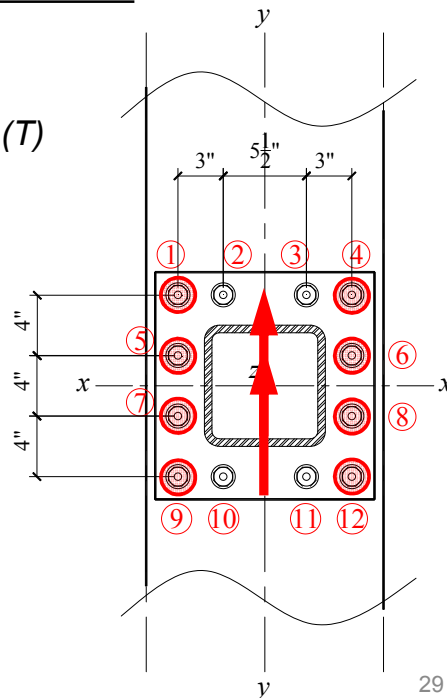
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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)

$e = 12''$

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

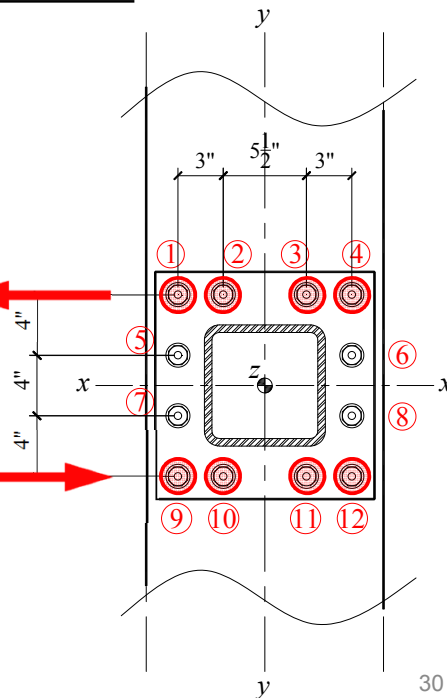
$V_x = 0.729 \text{ kips/bolt}$

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

$C = 2.917 \text{ kips}$

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

$C = 2.917 \text{ kips}$



Moment Connections with Combined Loads – Boundary Conditions

Approach 1

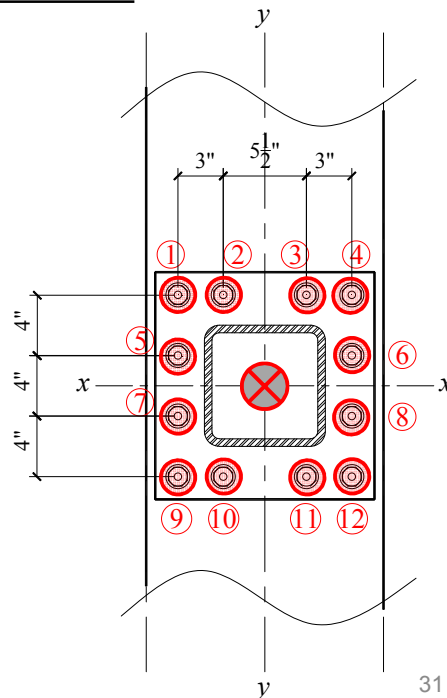
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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

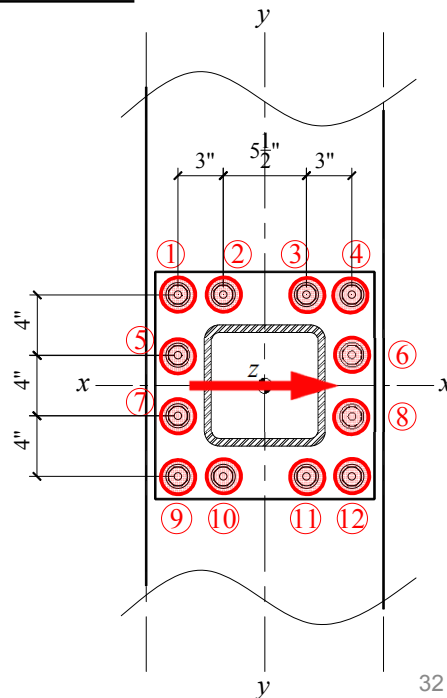
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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

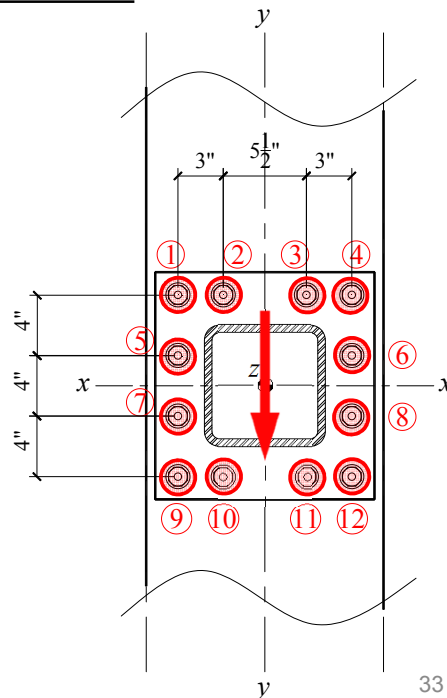
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

All bolts carry V_y (V)

$e = n/a$

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

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



Moment Connections with Combined Loads – Boundary Conditions

Approach 1

Bolt	Load	Load	e	Qty	1	2	3	4	5	6	7	8	9	10	11	12
Load	Type	(kips & inches)	(in)		(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(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



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

Section J3.7

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

$$R_n = F'_t A_b$$

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

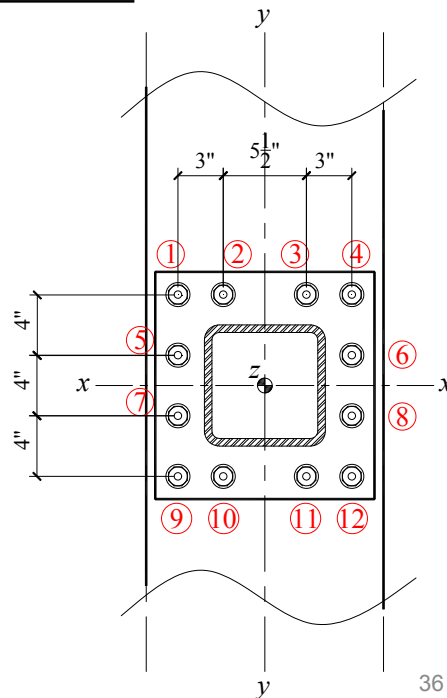


Moment Connections with Combined Loads – Boundary Conditions

Approach 2

No Tension/Shear Interaction

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



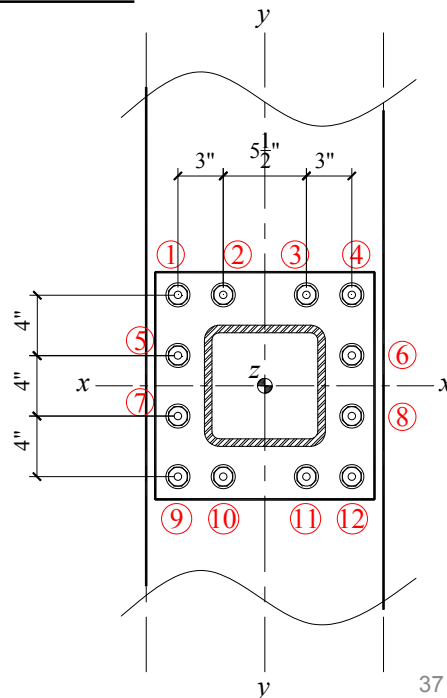
Moment Connections with Combined Loads – Boundary Conditions

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

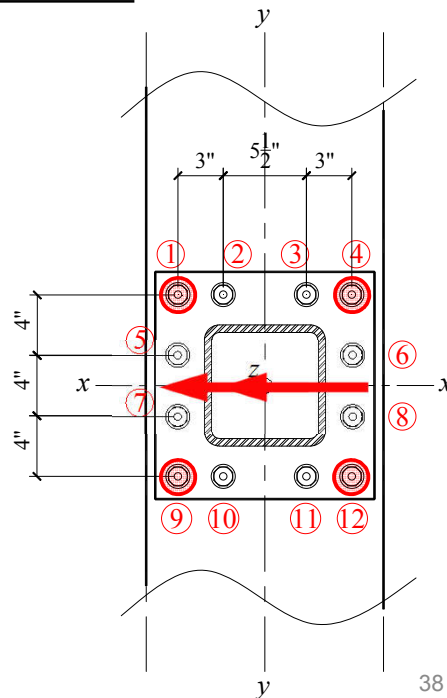
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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

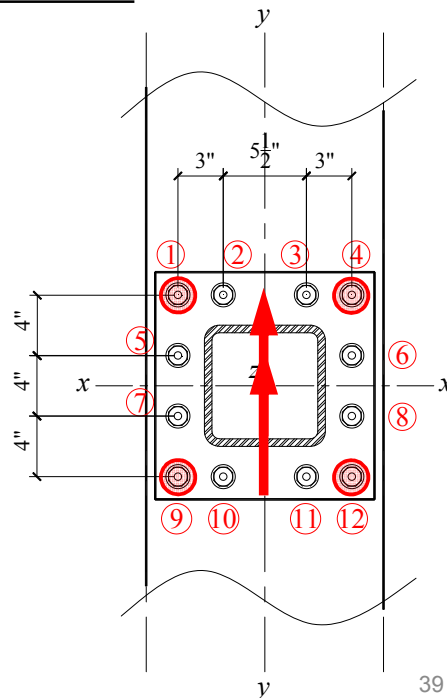
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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)

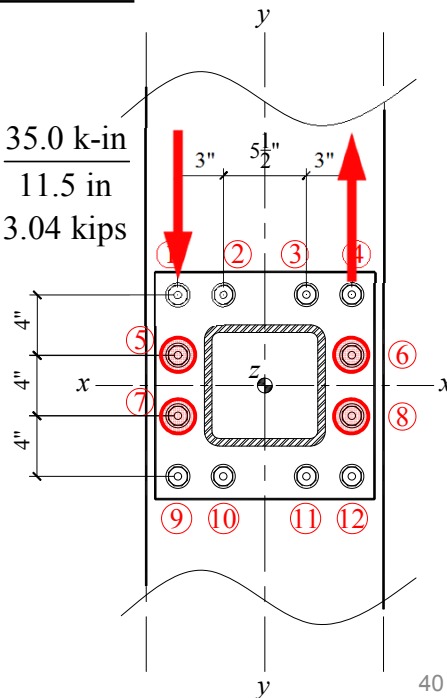
$e = 11.5''$

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

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

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

$C = 3.04 \text{ kips}$



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

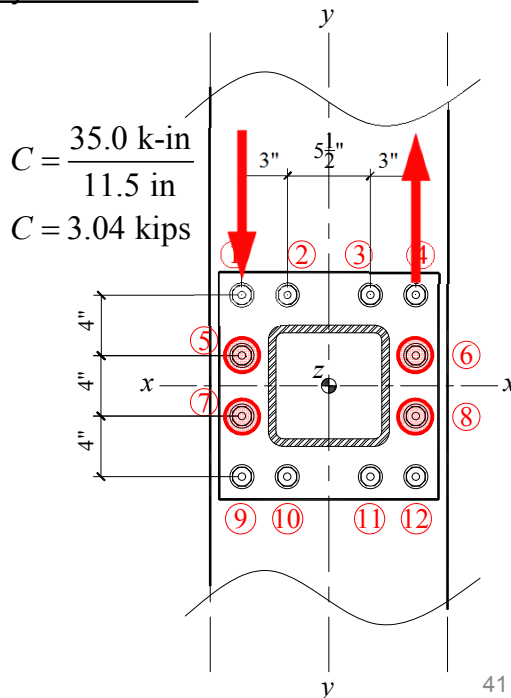
Bolts 5, 6, 7, 8, M_z (V)

$e = 11.5''$

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

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

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



Moment Connections with Combined Loads – Boundary Conditions

Approach 2

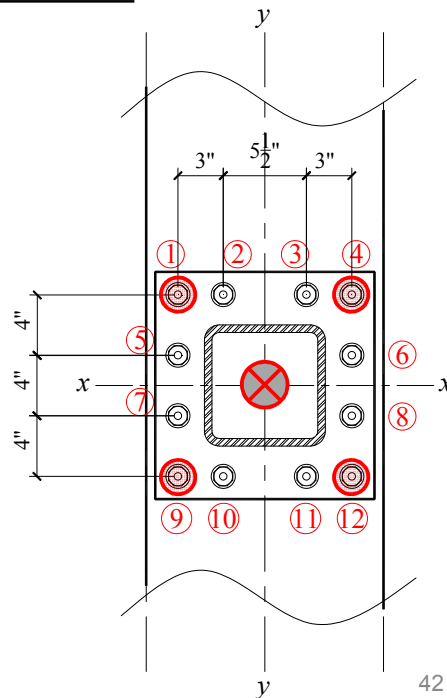
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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

$e = n/a$

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

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



Moment Connections with Combined Loads – Boundary Conditions

Approach 2

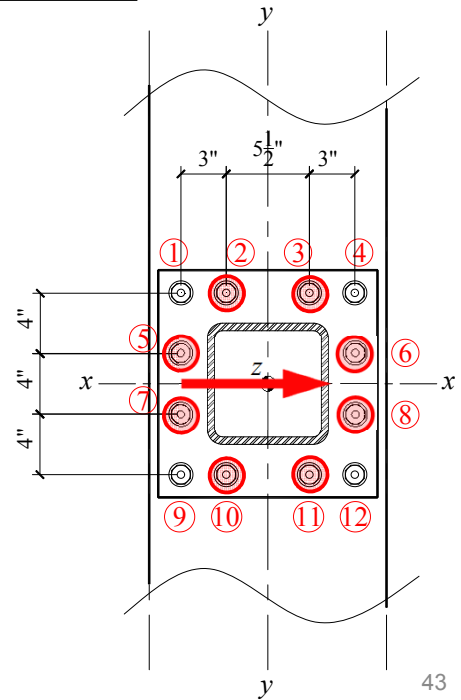
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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

$e = n/a$

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

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



Moment Connections with Combined Loads – Boundary Conditions

Approach 2

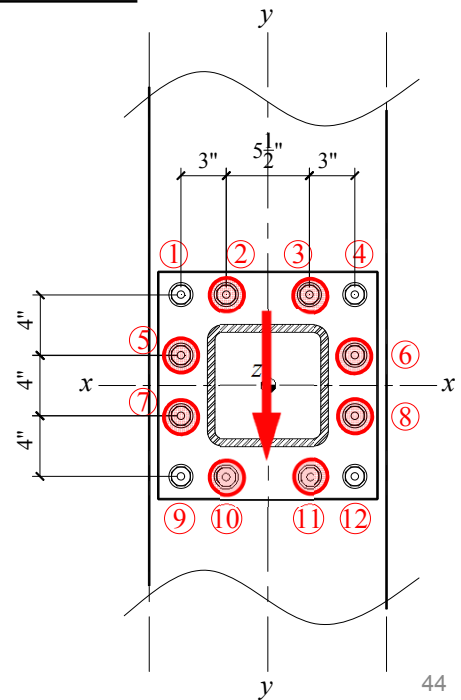
Load	Magnitude (+/-) (kips & inches)
M_x	250
M_y	45.0
M_z	35.0
A	15.0
V_x	10.0
V_y	65.0

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

$e = n/a$

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

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



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	-



Moment Connections with Combined Loads – Boundary Conditions

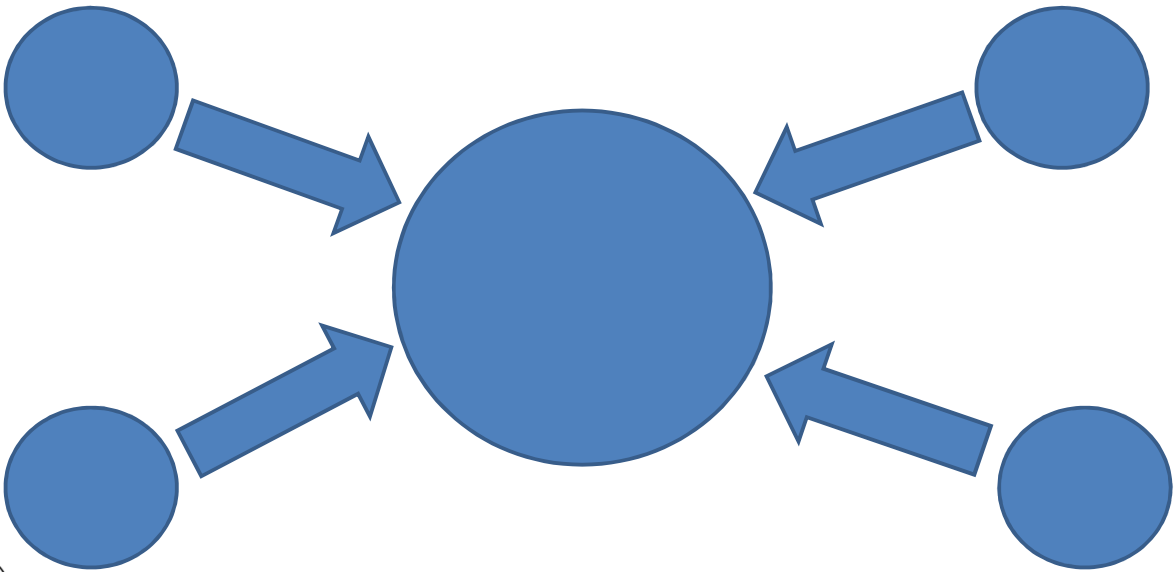
Approach 2

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	-

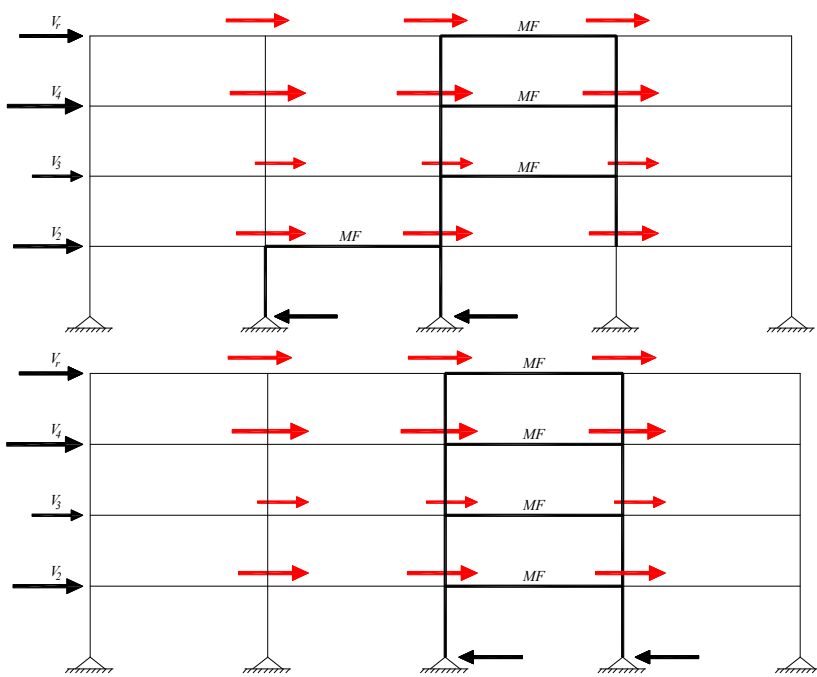
Shear/tension interaction is not required for any bolt

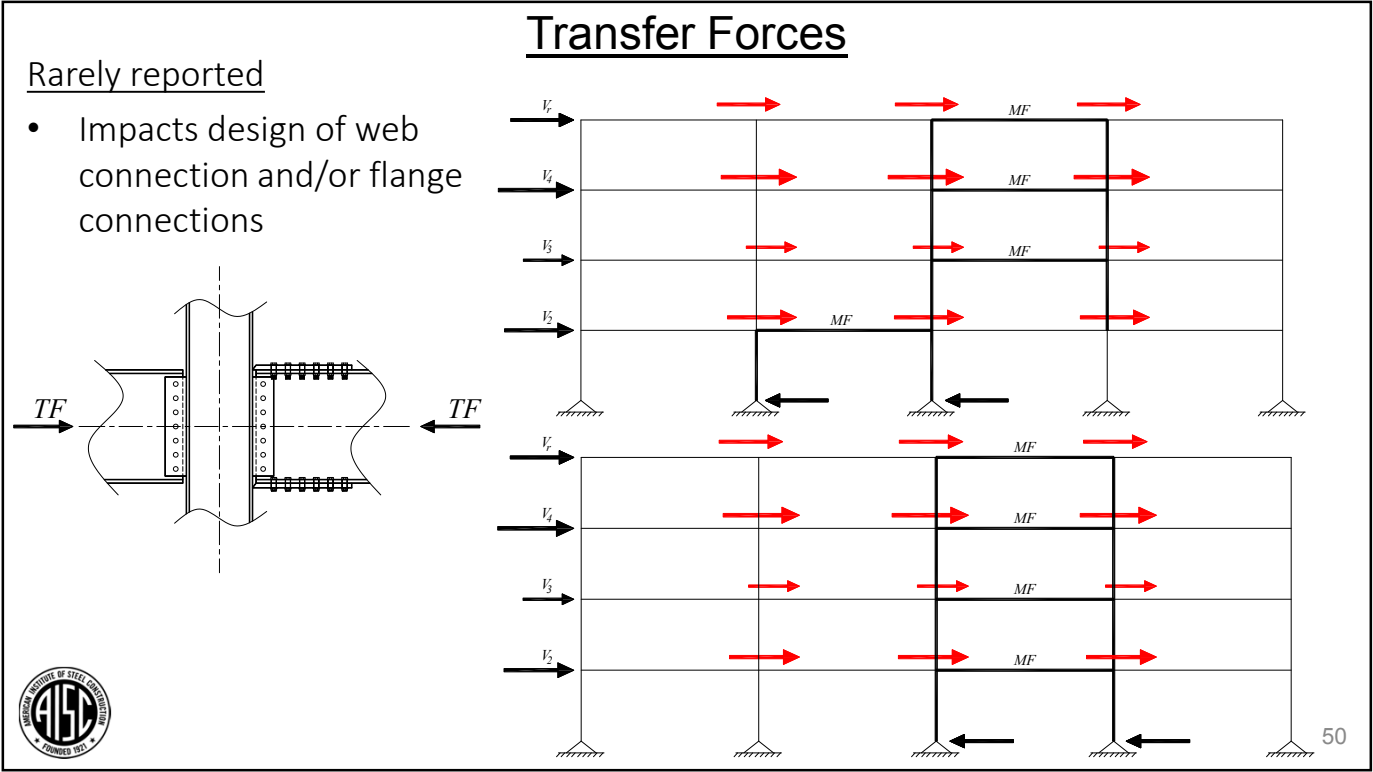
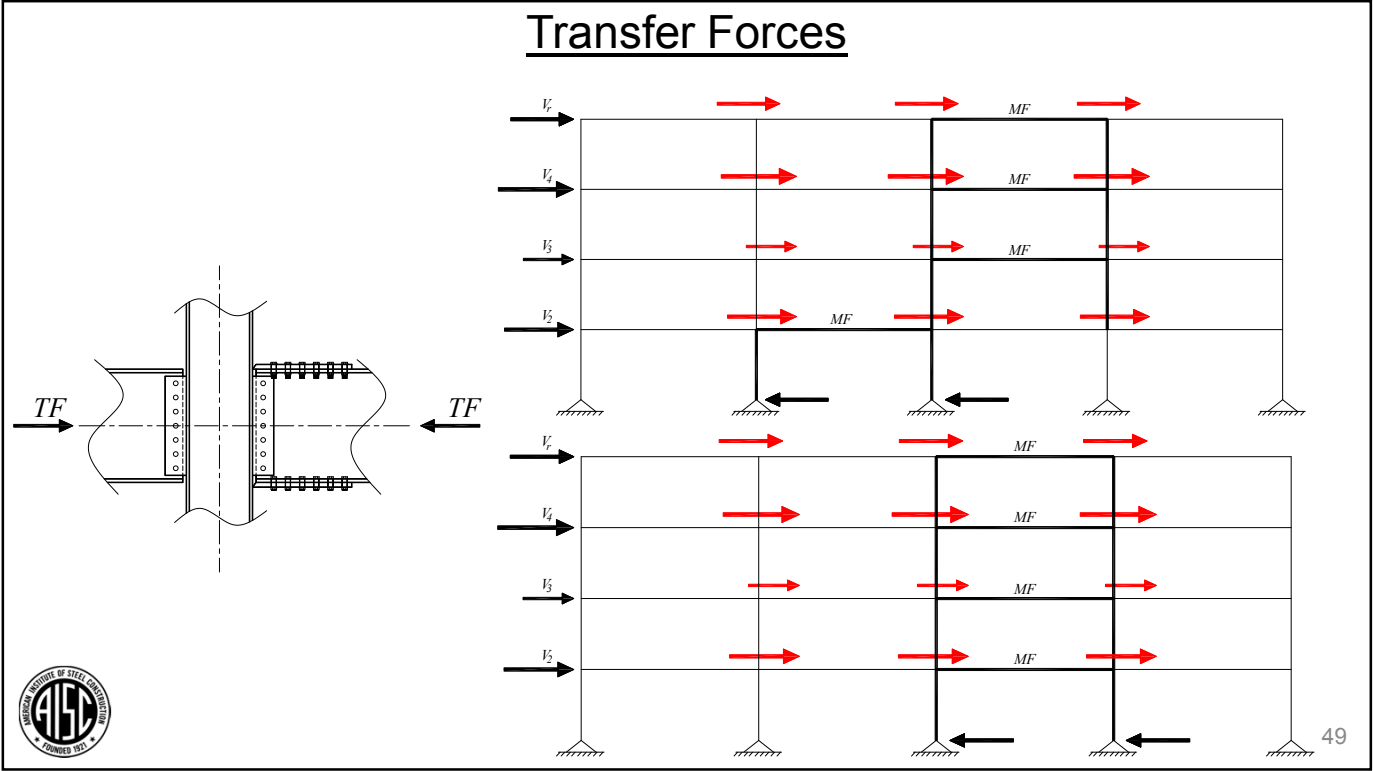


Transfer Forces and Collectors



Transfer Forces





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.

- True
- False
- It depends on source of the moment (gravity or lateral)
- 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.

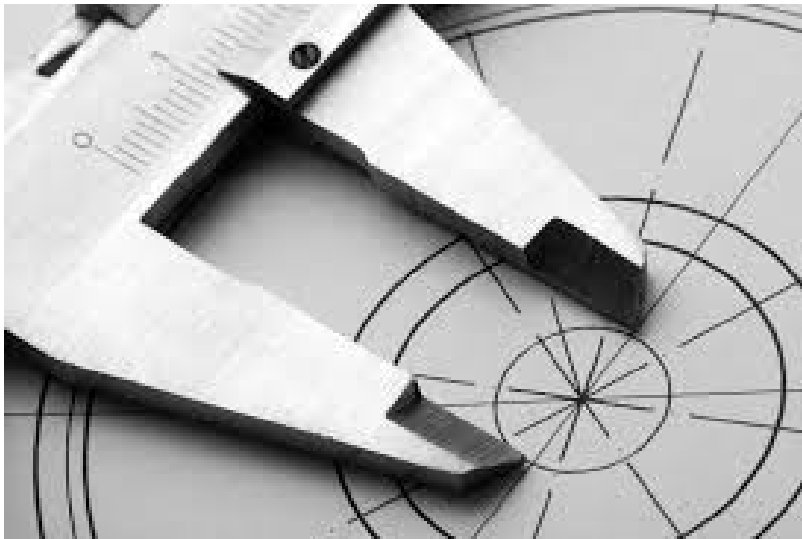
- a) True
- b) False
- c) It depends on source of the moment (gravity or lateral)
- d) Only if the source of the moments is gravity load only



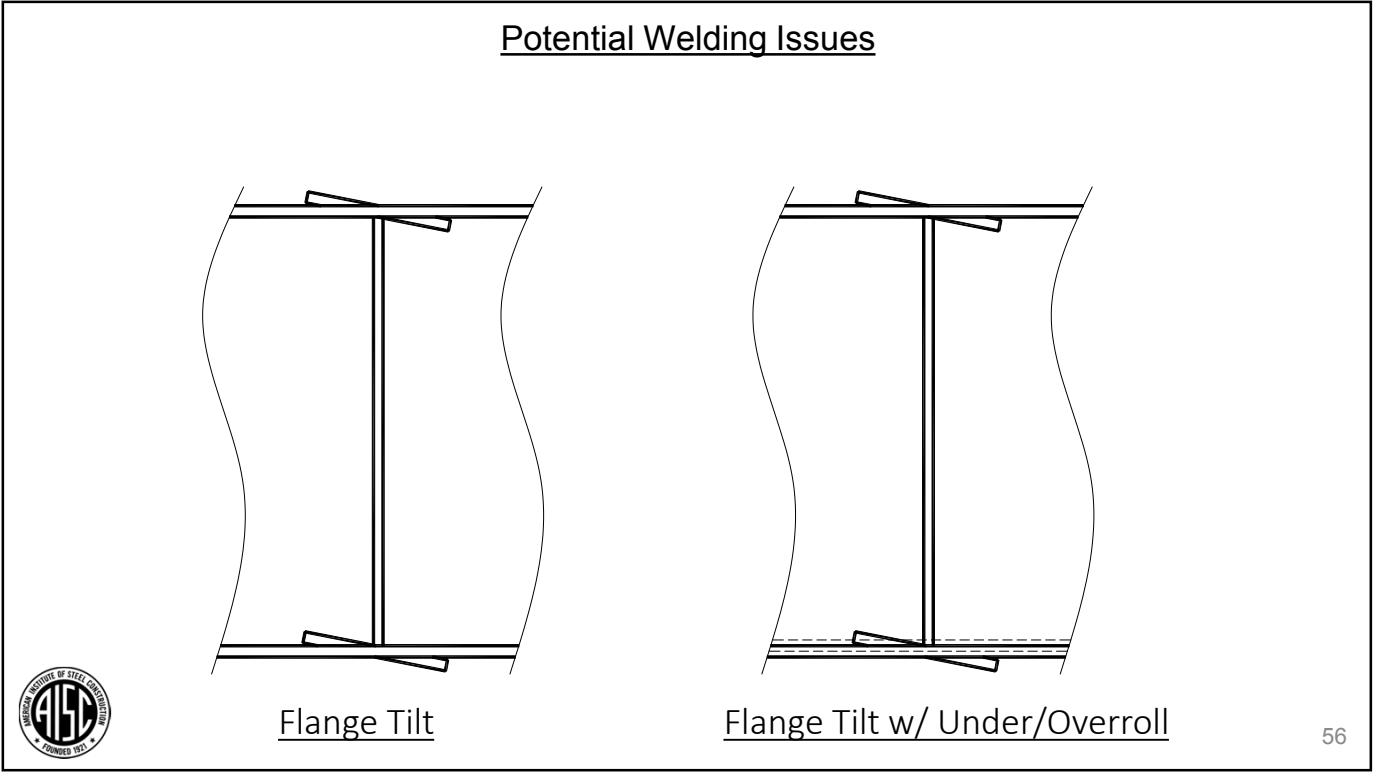
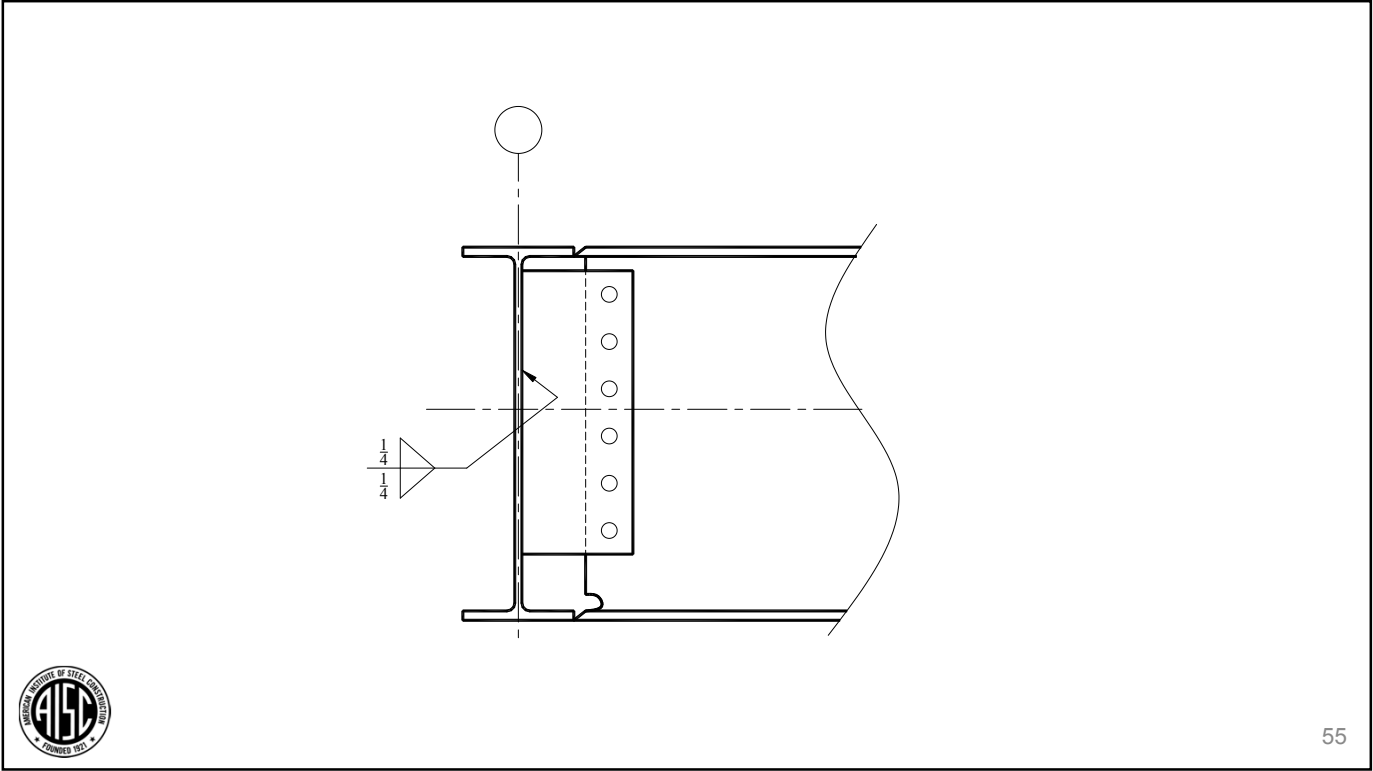
Select your answer!

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Erection and Mill Tolerances



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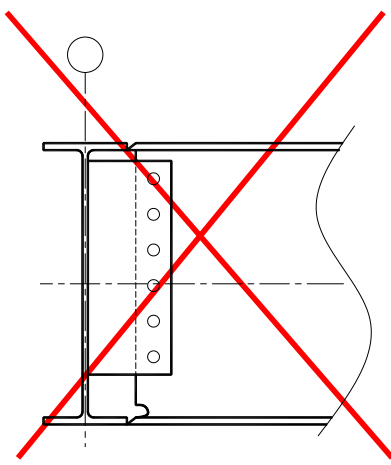


Flange Tilt

Flange Tilt w/ Under/Overroll



Possible Options



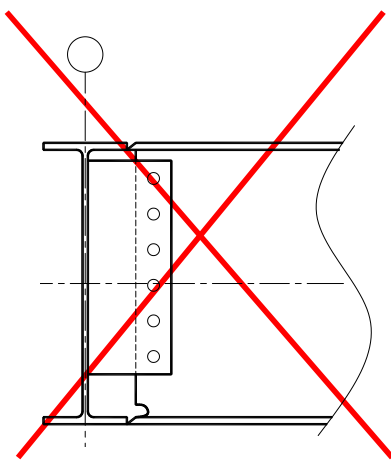
Moment Connection

Option 1

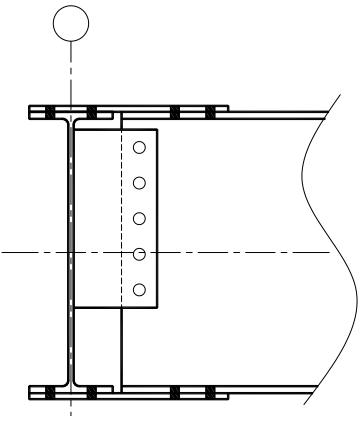
Option 2



Possible Options



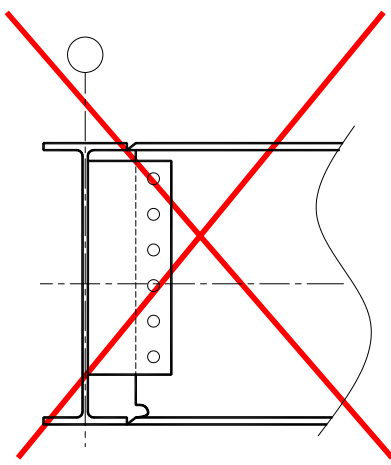
Moment Connection



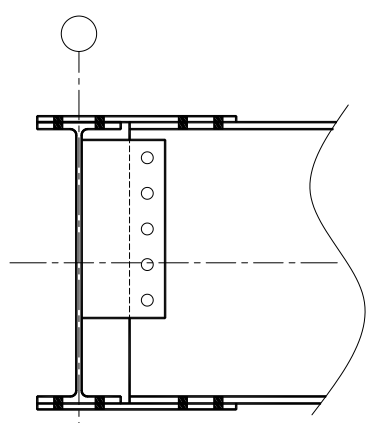
Option 1



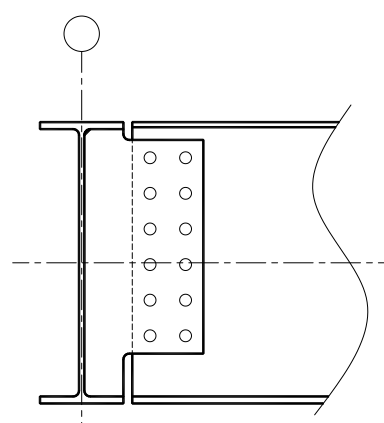
Possible Options



Moment Connection



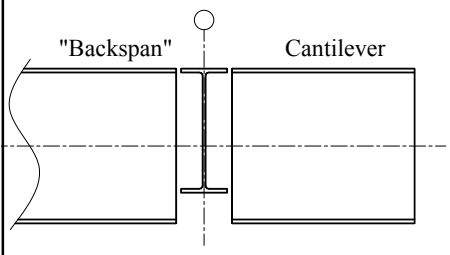
Option 1



Option 2



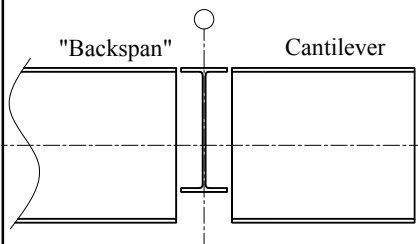
Shallow Supporting Members



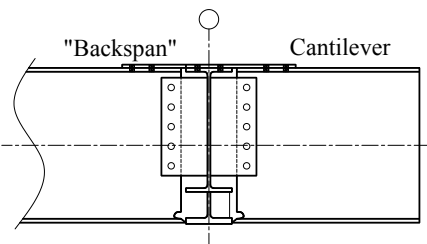
Beam-to-Beam



Shallow Supporting Members



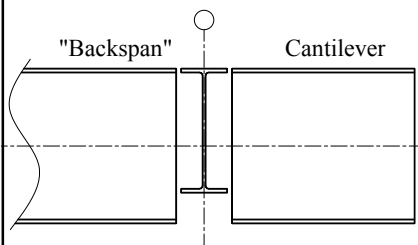
Beam-to-Beam



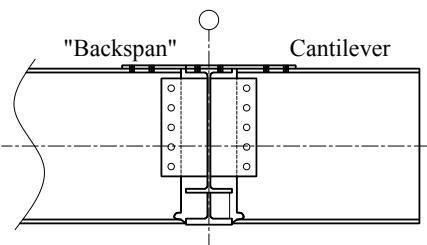
Flange Plate and Tee



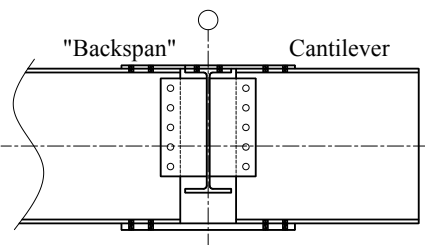
Shallow Supporting Members



Beam-to-Beam



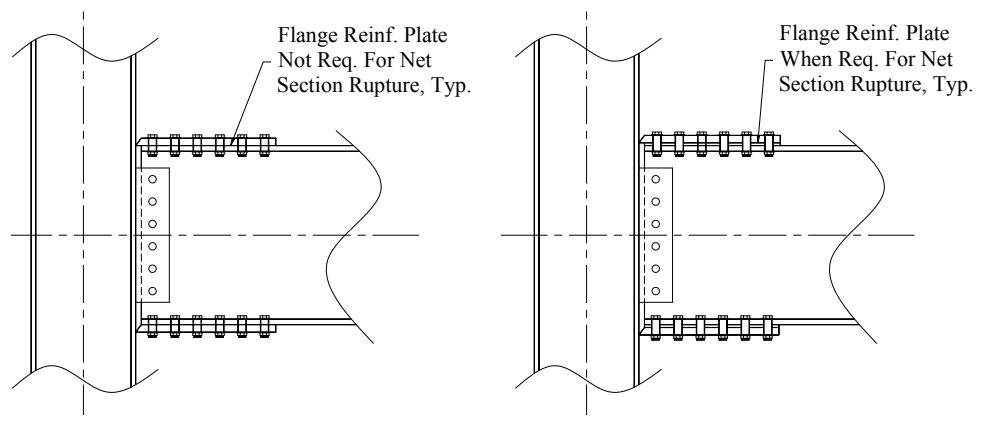
Flange Plate and Tee



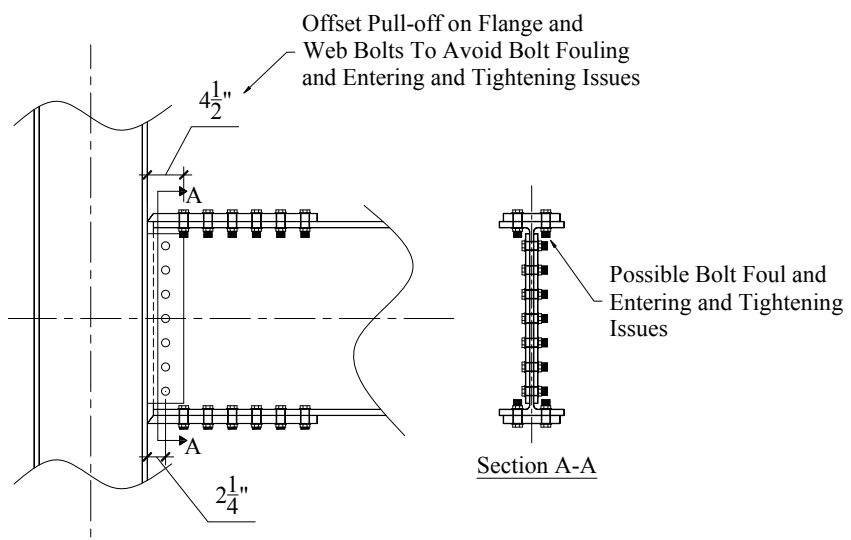
Flange Plates

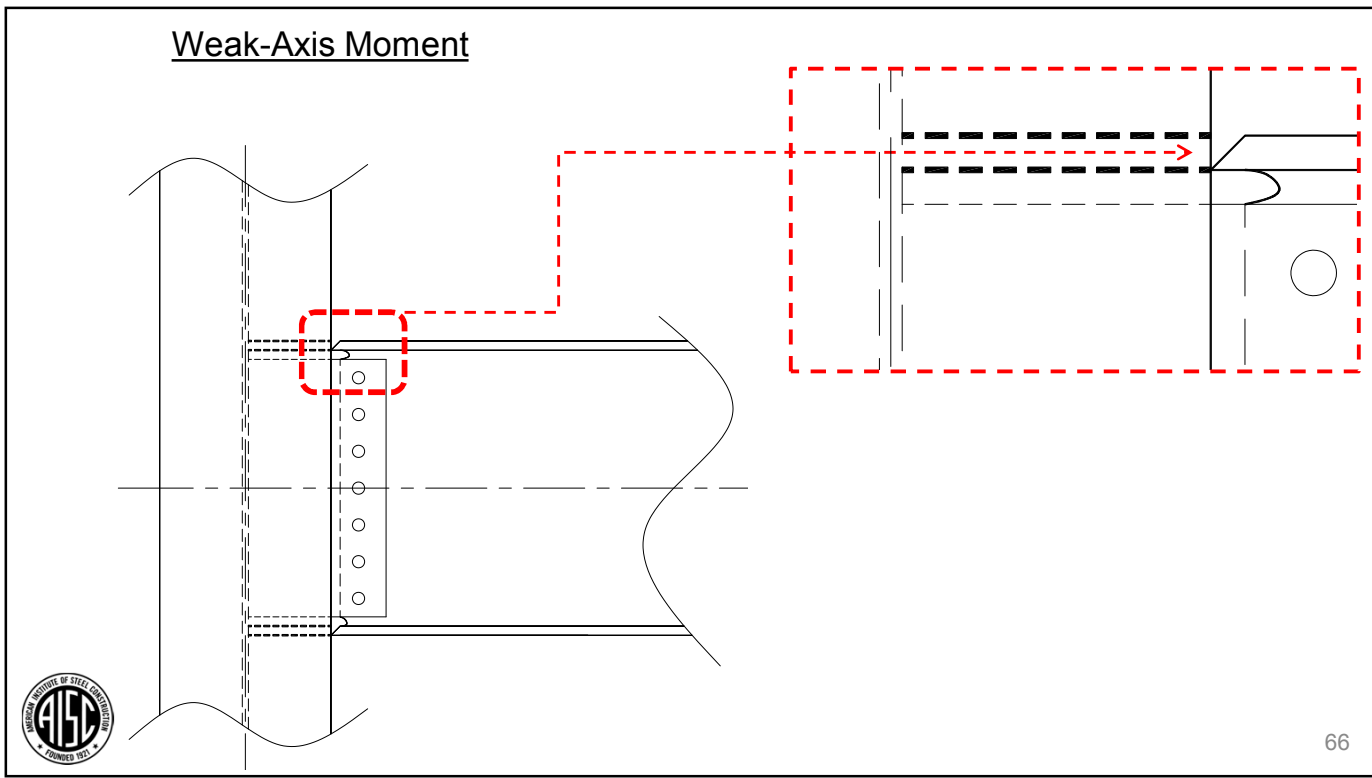
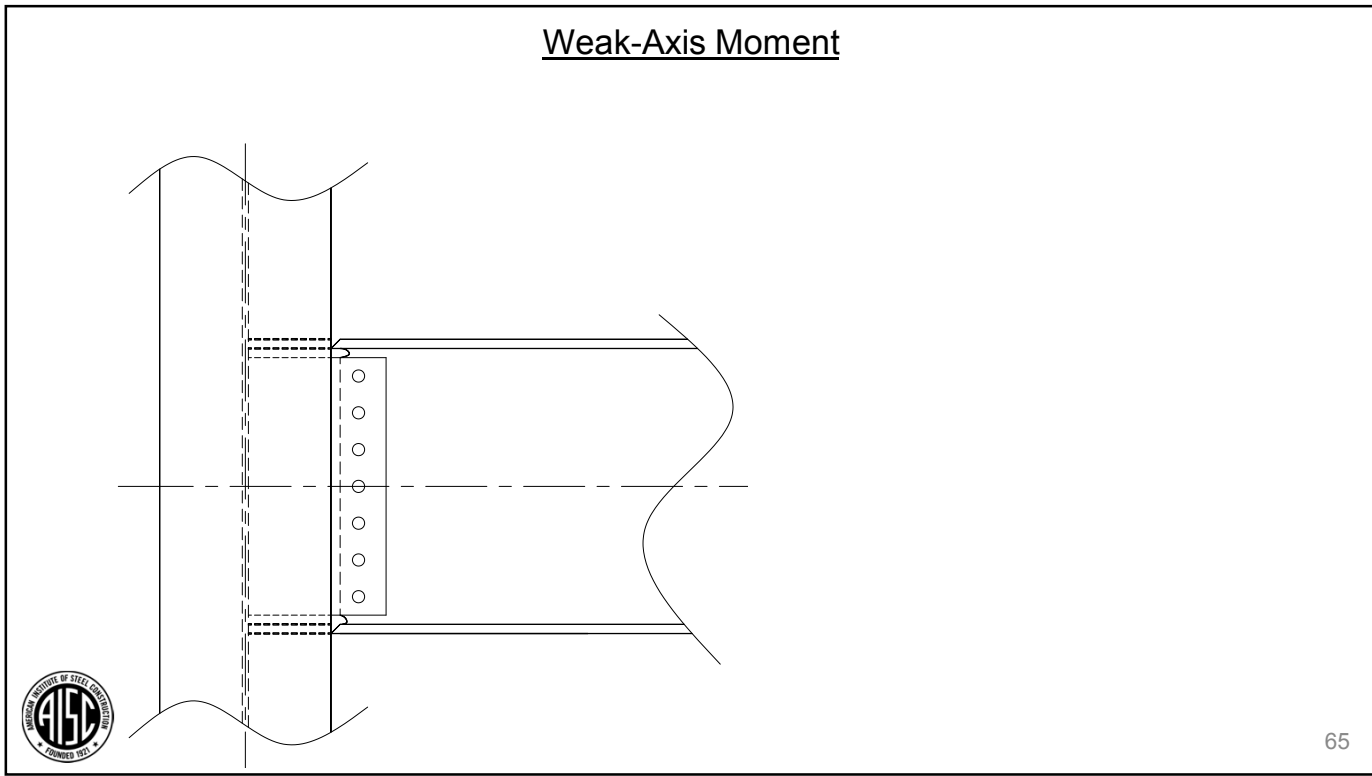


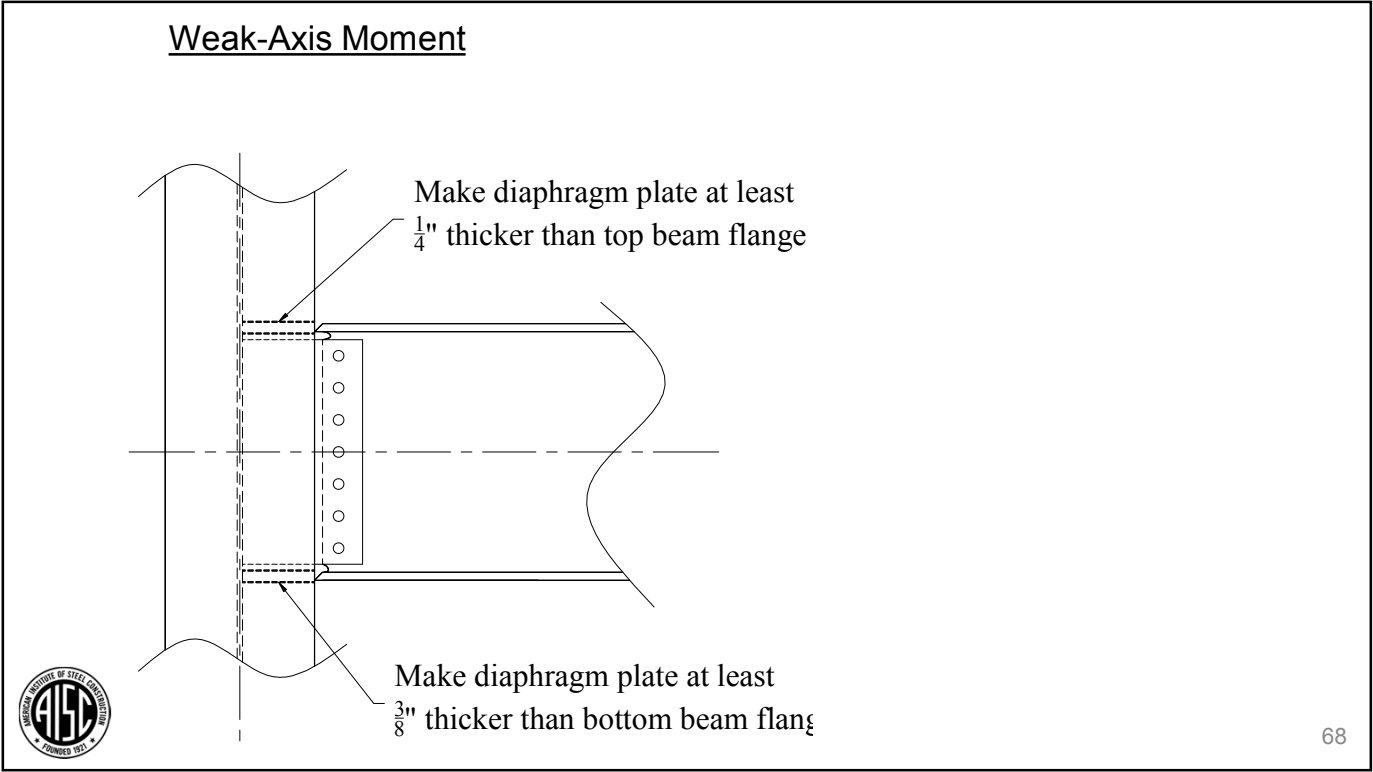
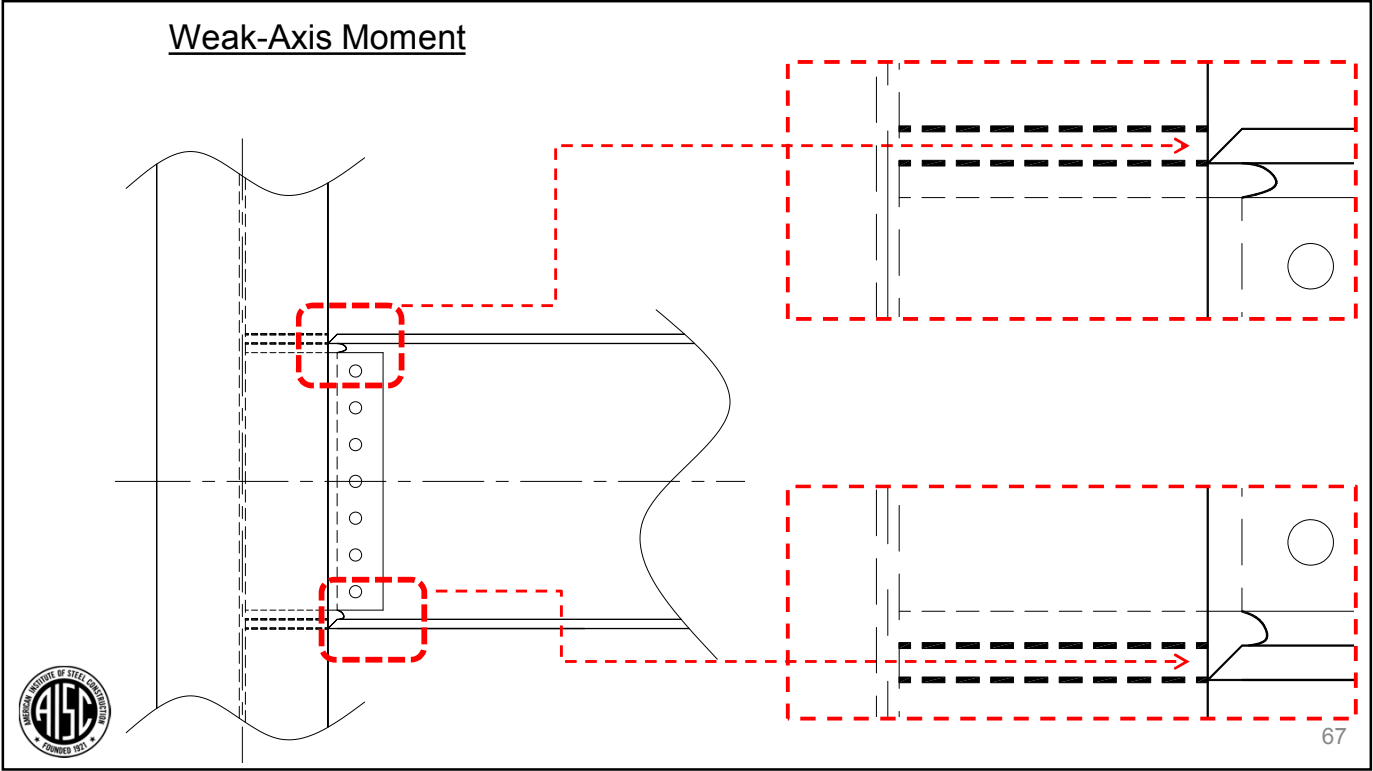
Account for Bolting – Limits on Stress Ratios



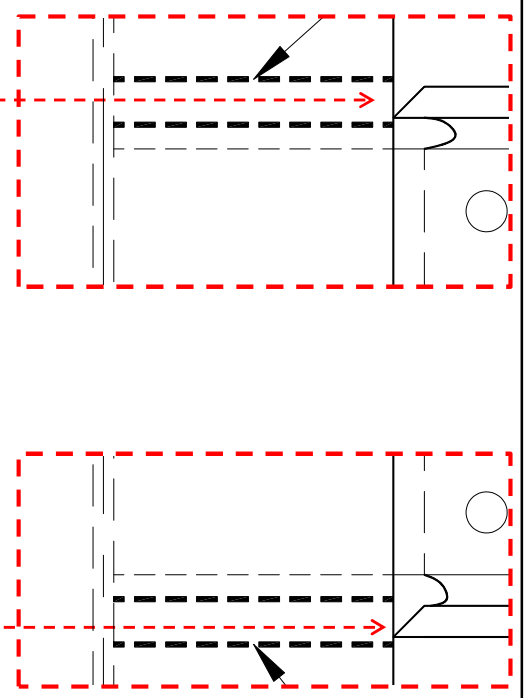
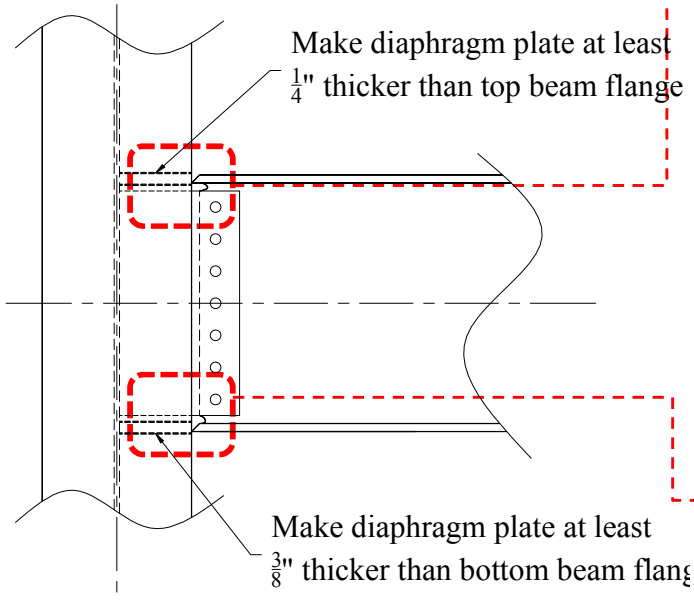
Entering and Tightening



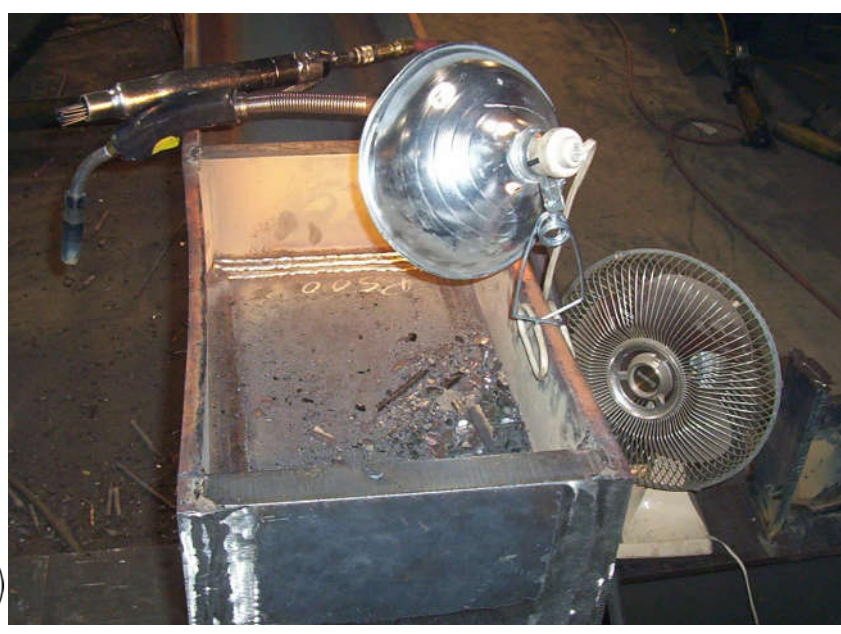




Weak-Axis Moment



Panel Zones and Column Stiffening

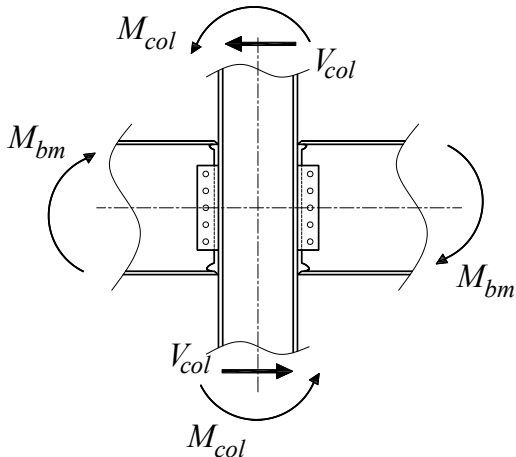


W14x90 Column
with 1.5" thick
continuity plates

Note distortion in
column flanges due to
welding



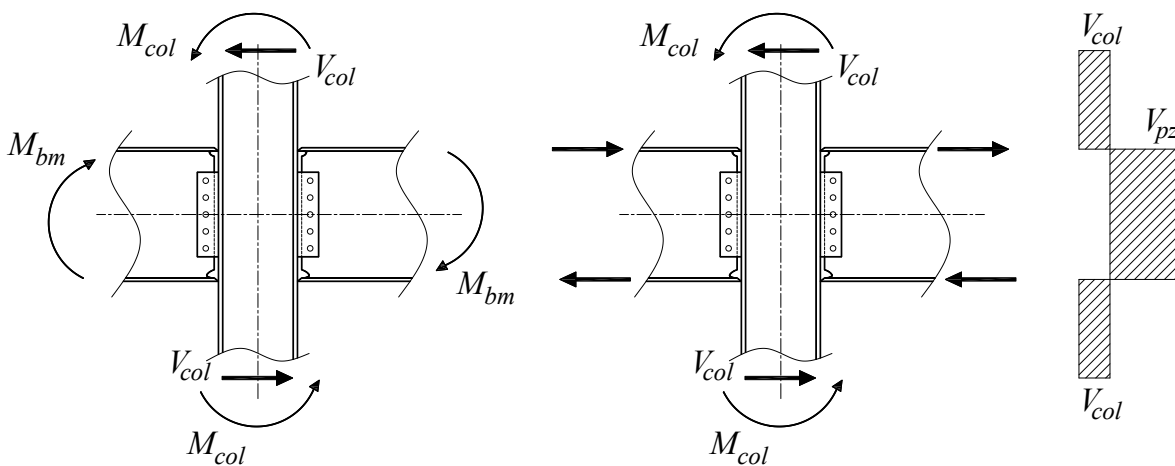
Two-Sided – Lateral Moments



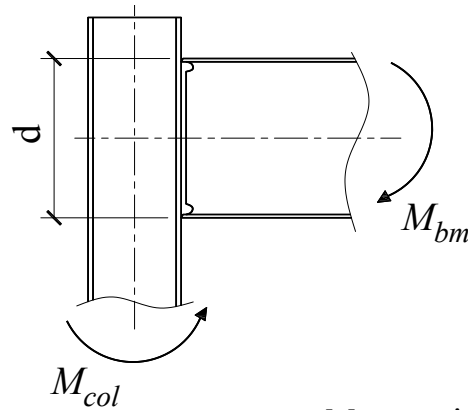
It's a mistake to assume that panel zone shear only occurs with lateral moments



Two-Sided – Lateral Moments



One-Sided – Gravity or Lateral – T/Column



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

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

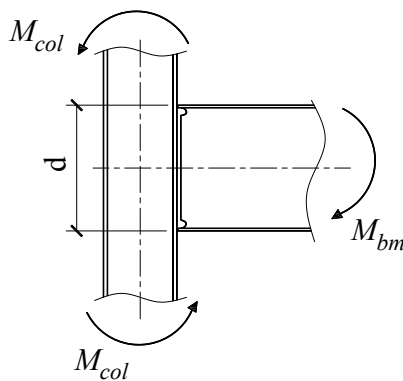
$$V_{dbl} = M_{dbl} / d$$



Stiffeners

Doublers

One-Sided – Gravity or Lateral – Column Midspan



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

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

$$V_{dbl} = M_{dbl} / d$$

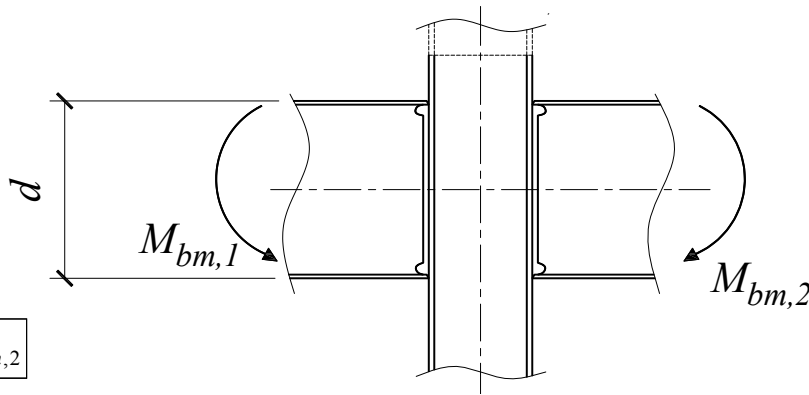


Stiffeners

Doublers



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

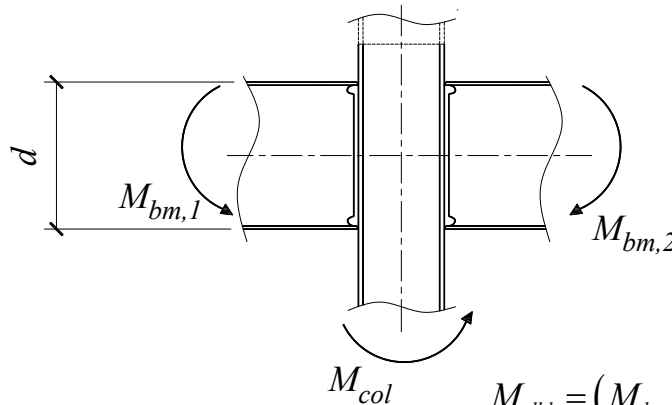
Stiffeners

Doublers



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Two-Sided – Gravity – Equal Beam Moments



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

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

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

$$V_{dbl} = M_{dbl} / d$$

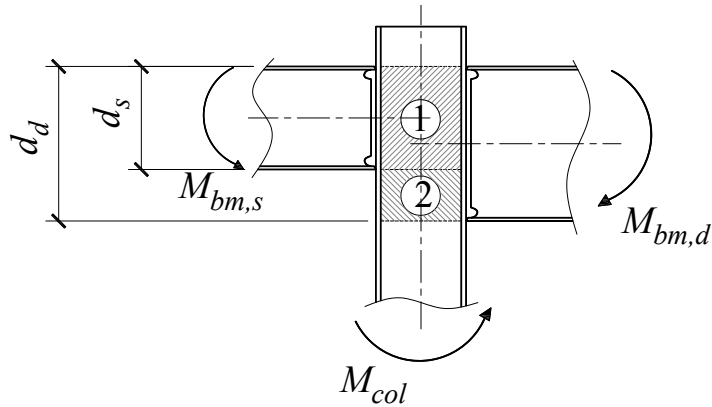
Stiffeners

Doublers



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Two-Sided – Gravity or Lateral – T/Column



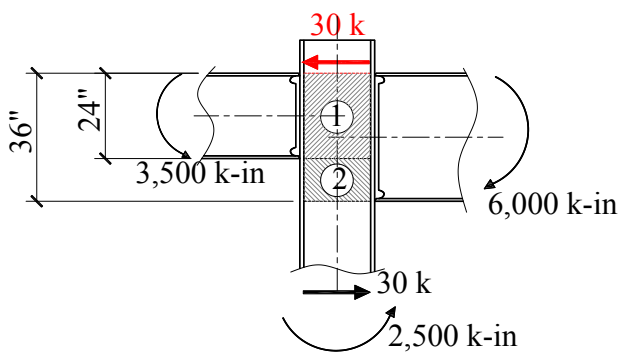
Unequal depth beams can induce more than one panel zone

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



77

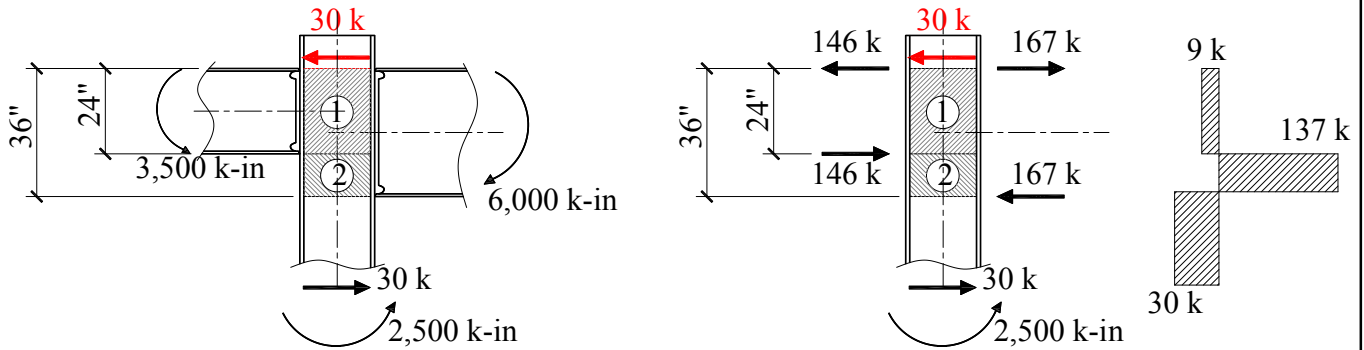
Multiple Panel Zones



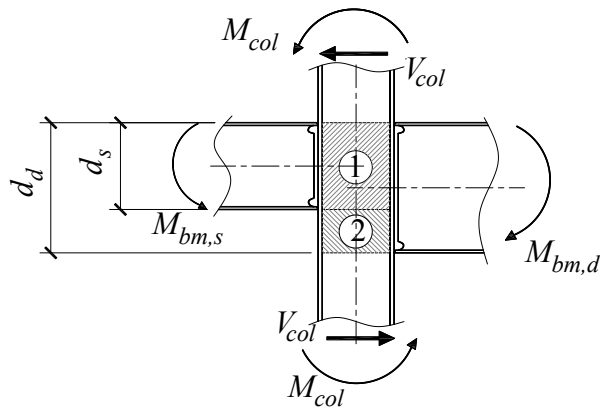
78



Multiple Panel Zones



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



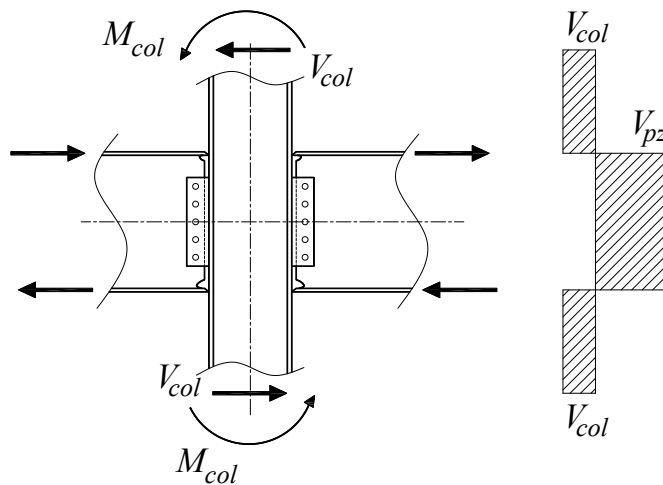
Panel Zone Strength

Section J10.6

(a) ...panel zone deformation is not considered...

$$\alpha P_r \leq 0.4P_y$$

$$R_n = 0.60F_y d_c t_w \quad (\text{EQ. J10.9})$$



Panel Zone Strength

Section J10.6

(a) ...panel zone deformation is not considered...

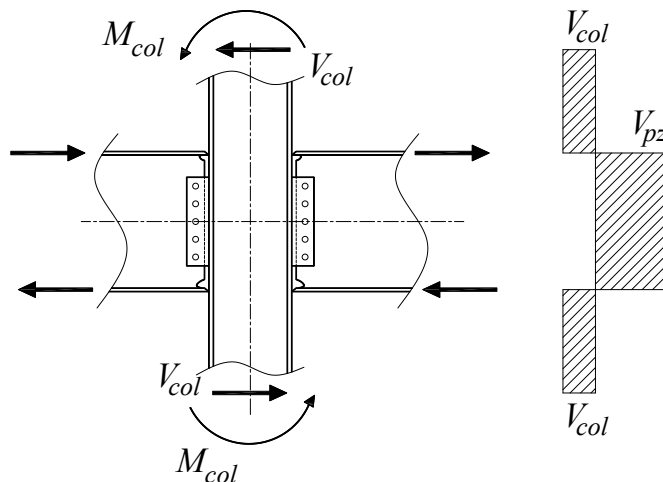
$$\alpha P_r \leq 0.4P_y$$

$$R_n = 0.60F_y d_c t_w \quad (\text{EQ. J10.9})$$

(b) ...panel zone deformation is considered...

$$\alpha P_r \leq 0.75P_y$$

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



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)



Panel Zone Strength

Section J10.6

(a) ...panel zone deformation is not considered...

$$\alpha P_r \leq 0.4P_y$$

$$R_n = 0.60F_y d_c t_w \quad (\text{EQ. J10.9})$$

- 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.

(b) ...panel zone deformation is considered...

$$\alpha P_r \leq 0.75P_y$$

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

- The contribution of additional term in J10.11 is relatively less in thinner flanged members and in deeper column sections (i.e., deeper webs)



Panel Zone Strength

Section J10.6

(a) ...panel zone deformation is not considered...

$$\alpha P_r \leq 0.4P_y$$

$$R_n = 0.60F_y d_c t_w \quad (\text{EQ. J10.9})$$

(b) ...panel zone deformation is considered...

$$\alpha P_r \leq 0.75P_y$$

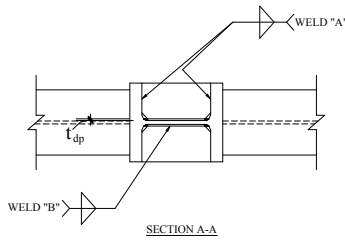
$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w} \right) \quad (\text{EQ. J10.11})$$

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

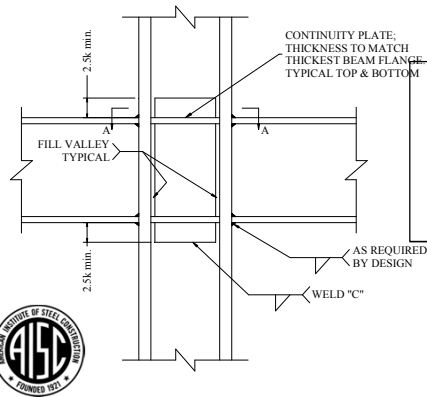


Two-Sided Connection with Continuity and Web Doubler Plates



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	-

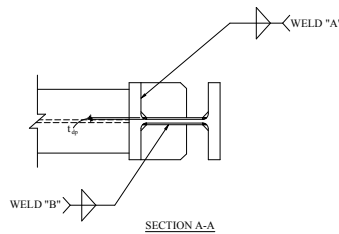
Note that the information provided should include interaction for "shared columns."



Make clear if these are design requirements or just for bidding

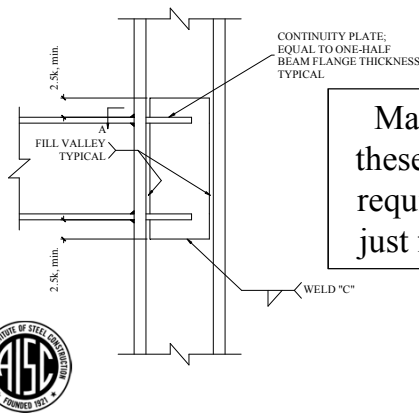


One-Sided Connection with Continuity and Web Doubler Plates



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	-

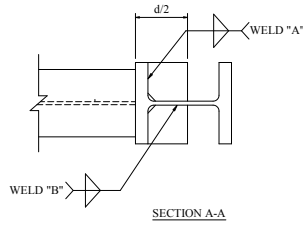
Note that the information provided should include interaction for "shared columns."



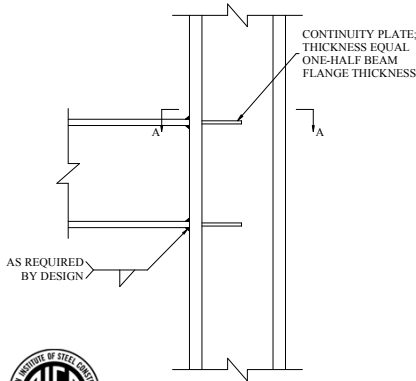
Make clear if these are design requirements or just for bidding



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



Clean Column Worksheet

<https://www.steeltools.org/>



Client: AISC Night School

Engineer:

Remarks: Interior Columns: Lines B, C, and E

Clean Columns V13.1 was developed to return the lightest column section that can be used without stiffeners and/or doubler plates to develop a specified percentage of a selected beam's plastic moment capacity, based on the criteria in *AISC Design Guide Series #13* and the *2005 AISC Specification for Structural Steel Buildings*. The design of the column for axial load capacity is not considered.

[Click Here for Detailed Instructions and Definitions](#) [What's New in Version 13.1?](#)

[1\) Click Here to View Assumptions](#)

The effects of a composite concrete floor slab are not considered in the analysis. The connection is assumed to be part of a frame resisting wind or low seismic forces. In other words, the structure is designed to meet the requirements in the AISC Specification for Structural Steel Buildings with no special seismic detailing. The panel zone is assumed to remain nominally within the elastic range. Other assumptions apply. Review all the assumptions by clicking on the button above.

[2\) Click Here to Set Connection Configuration](#)

Beams Connected on Both Sides
Beams are not Connected Near the Top of the Column
Flange-Plated

Flange Plate Thickness = 0.500 in
Bottom Plate Width = 8.00 in
Top Plate Width = 8.00 in

[3\) Click Here to Set Beam Section](#)

Beam Section: W24x94

$\phi_b M_p = 953$ kip-ft Plastic Moment Strength

[4\) Click Here to Set Forces and Material Properties](#)

2005 LRFD Specification for Structural Steel Buildings

$(F_y)_c = 50$ ksi	Column Specified Minimum Yield Strength
$(F_y)_b = 50$ ksi	Beam Specified Minimum Yield Strength
$(V_y)_c = 75$ kips	Column Story Shear
$(P_y)_c = 120$ kips	Column Axial Load
$(P_y)_b = 25$ kips	Beam Axial Load, Right Side
$M_b = 300$ k-ft	Beam Moment, Right Side
$(P_y)_l = 0$ kips	Beam Axial Load, Left Side
$M_l = 300$ k-ft	Beam Moment, Left Side

Figure 1: Connection Configuration

[5\) Resultant Forces \(a positive value indicates C compression\)](#)

$V_c = 228$ kips	Total Panel-Zone Shear Force
$(P_y)_{re} = 158$ kips	Top Flange Force, Right Side
$(P_y)_{le} = -133$ kips	Bottom Flange Force, Right Side
$(P_y)_{rl} = -145$ kips	Top Flange Force, Left Side
$(P_y)_{ll} = 145$ kips	Bottom Flange Force, Left Side

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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, AISC/M, 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

6) Column Design Results:

	Lightest W8	Lightest W10	Lightest W12	Lightest W14	Lightest W16	Lightest W18
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7) Column Calculations:

Click to Select a Column Section and View Column Strength Calculations

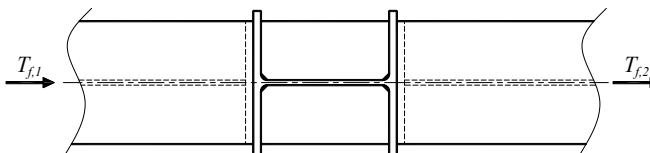
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If not, I strongly recommend this worksheet!



A Word on Continuity Plate Welds

Consider the lateral moment connection shown...

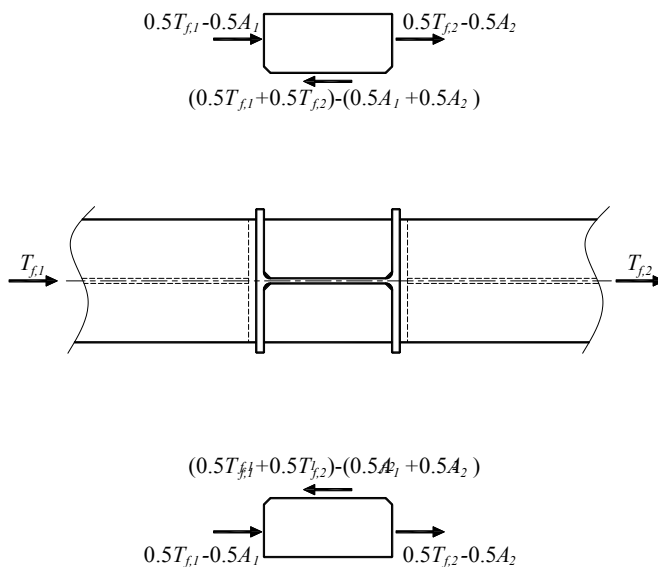


A Word on Continuity Plate Welds

Consider the lateral moment connection shown...

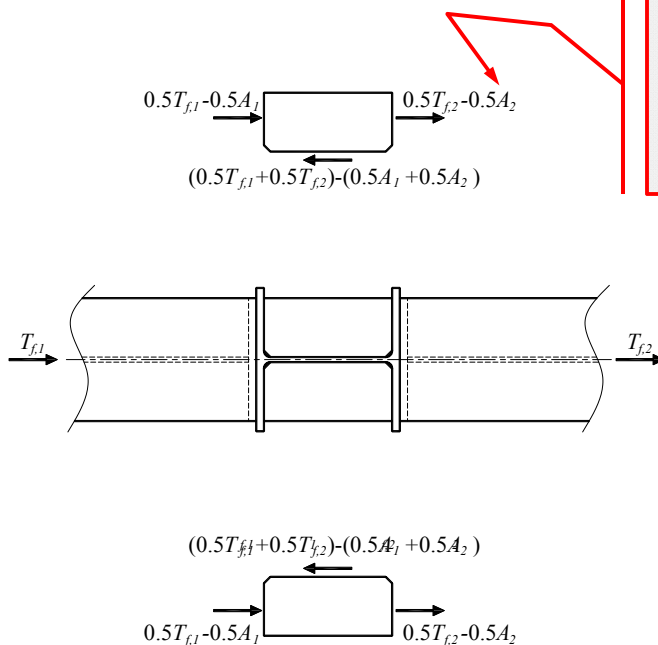
$T_{f,i}$ = Tension Flange Force

A_i = Controlling Limit State for C.P. Checks



A Word on Continuity Plate Welds

Consider the lateral moment connection shown...

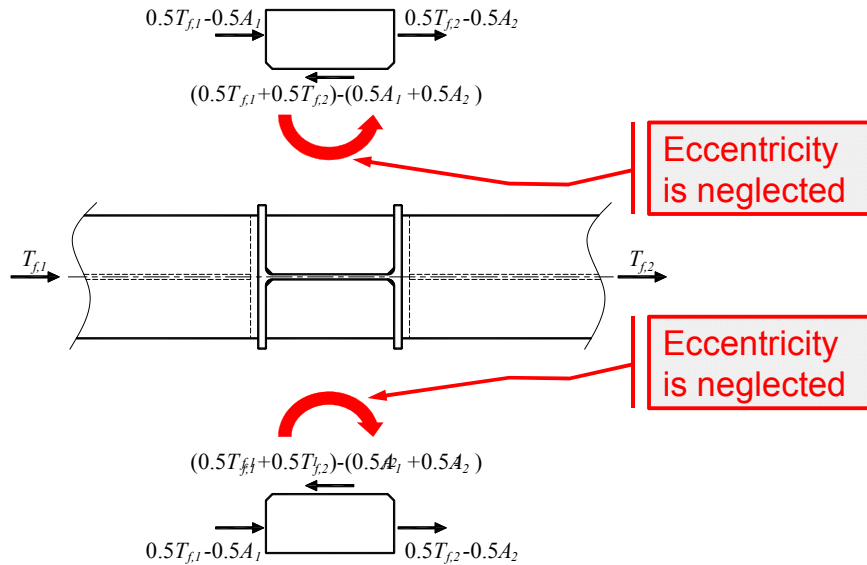


Interface weld demand is reduced by the controlling limit state of the column checks



A Word on Continuity Plate Welds

Consider the lateral moment connection shown...



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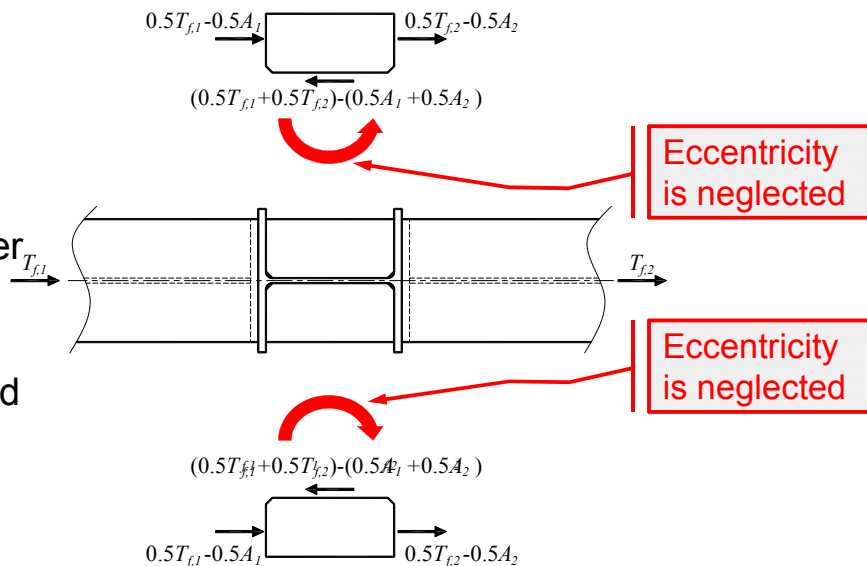
A Word on Continuity Plate Welds

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



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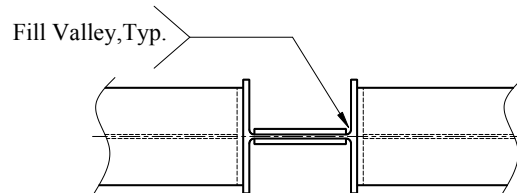


A Word on Double Plate Welds

“Fill Valley” Welds

CJP???

Mag. Particle Testing???



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A Word on Double Plate Welds

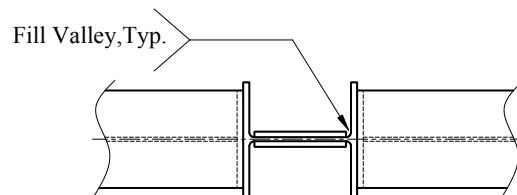
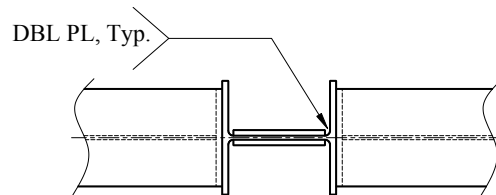
“Fill Valley” Welds

CJP???

Mag. Particle Testing???

Addressed in AWS
D1.8 2016...

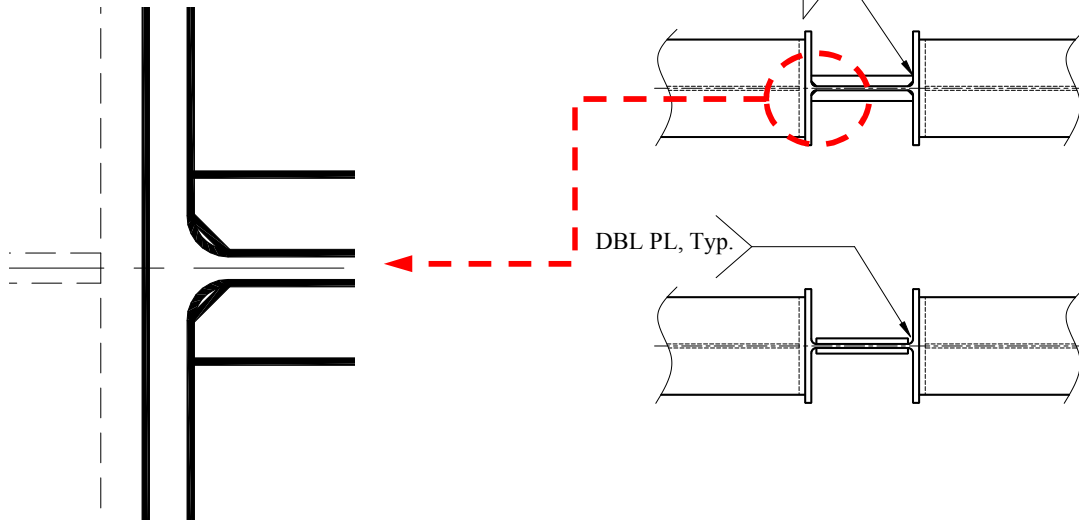
...but really applies
to all dbl. plate
welds



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A Word on Double Plate Welds



Increase plate
thickness to
account for bevel



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



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!

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Moment Connections

AISC | Questions?



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!



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.



8-Session Registrants

CEU / PDH Certificates

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



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Stronger.
Steel.

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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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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Night School Resources

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



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Night School Resources

Go to www.aisc.org and sign in.



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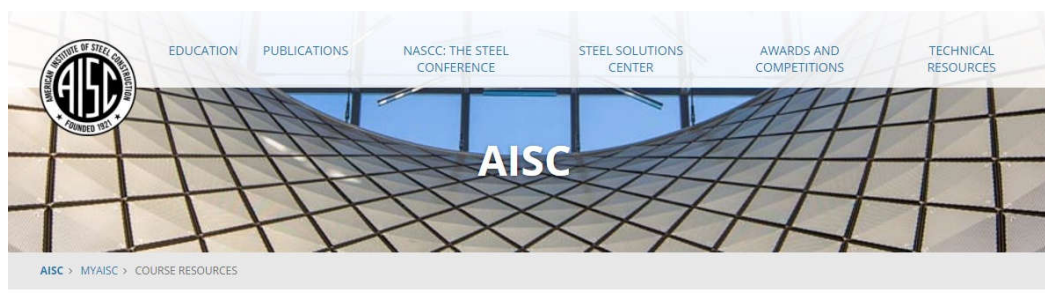
View online resources for Night School and Live Webinar package registrations.

[VIEW RESOURCES](#)



8-Session Registrants

Night School 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



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Night School 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 - Buildings 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



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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

