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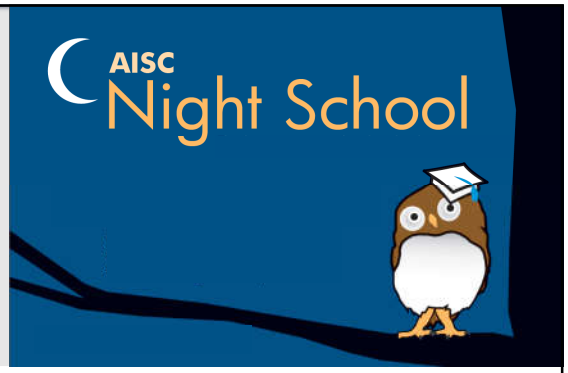

Connection Design

Tips, Tricks, and Lessons Learned

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AISC Night School

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Course Description

19.3 Shear Connections
February 25, 2019

Indisputably, beam end shear connections comprise the bulk of the total connections in a structural steel package. Connection designers have a plethora of options in regard to the type of shear connections to use. Magnitude of load, geometry, main member type, erection ease, shop schedule, and project specifications are just some of the considerations that impact the decision on the type of connection to use.

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

- List several means of communication between the Engineer of Record and the connection design engineer, which facilitate the approval process.
- Explain why the use of AISC *Steel Construction Manual*, Table 3-6, to specify shear connection loads, is a misuse of that table.
- Identify detailing practices that can simplify the fabrication and erection of single plate shear connections, reducing costs and minimizing construction errors.
- Describe cracking issues that can occur at shear connections during galvanizing.

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

Session 3: Shear Connections
February 25, 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	Shear Connections
Beam-to-Column Flange	Single Plate Shear Connection
Beam-to-Column Web	Conventional
One-Sided	Extended
Two-Sided	Single Angle Shear Connection
Beam-to-Beam	Bolted-Bolted
Equal Depth	Welded-Welded
Unequal Depth	Bolted-Welded
One-Sided (spandrel)	Double Angle Shear Connection
Two-Sided	Bolted-Bolted
Beam-to-Wall Edge	Welded-Welded
Beam-to-Wall Face	Bolted-Welded
	Shear End Plate Connection
	Seated Connections
	Knife Connections
	Bracket Connections

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Beam-to-Wall Face	Bolted-Welded
	Shear End Plate Connection
etc., etc., etc. …	Seated Connections
	Knife Connections
	Bracket Connections

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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 – Session 1)
  - Work Flow
  - Effective Communication (Cliff – Session 2)

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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 – Session 1)
  - Work Flow
  - Effective Communication (Cliff – Session 2)
- We'll talk a little about
  - Work Flow
  - Effective Communication
  - General Detailing
  - Specific Issues Related to Shear 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 shear connections

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Delegated Connection Design

Delegated Work

YOU CAN'T DELEGATE RESPONSIBILITY

Sources

- Your State's Rules and Regulations
- Legal Consultants
- Delegated Connection Design: What Are the EOR's Responsibilities? [N41A] NASCC 2014

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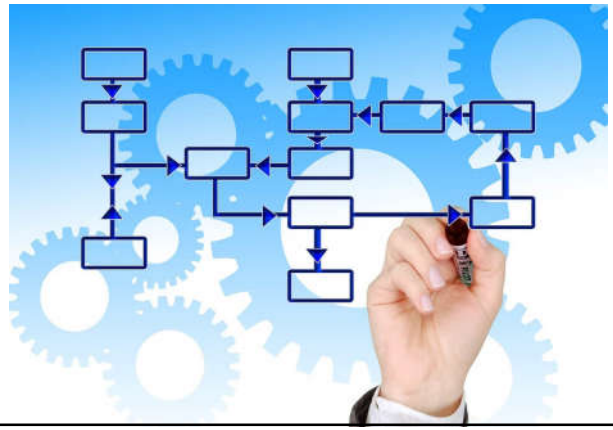
Getting to an IFF (issue for fabrication shop drawing)



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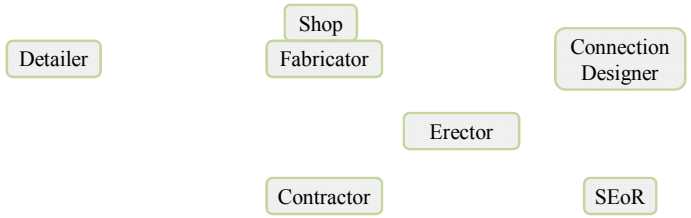
Getting to an IFF



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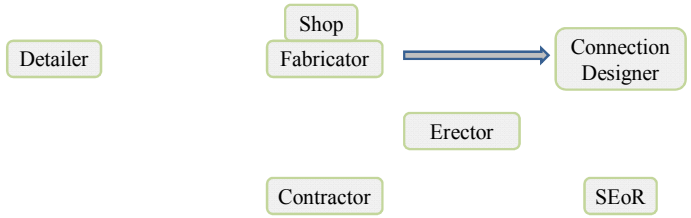
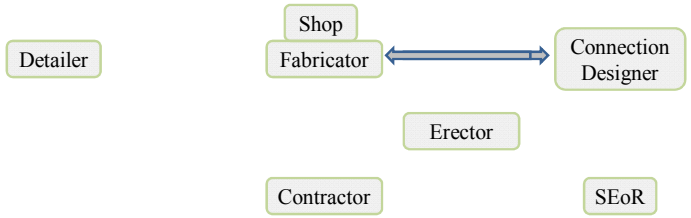
Getting to an IFF



Fabricator establishes preferred connection types and hierarchy and shares with connection designer

18

Getting to an IFF



Connection designer generates sample calculations for each type of connection (a.k.a., P-Sheets, Proposal Sheets, etc.) and submits to Fabricator

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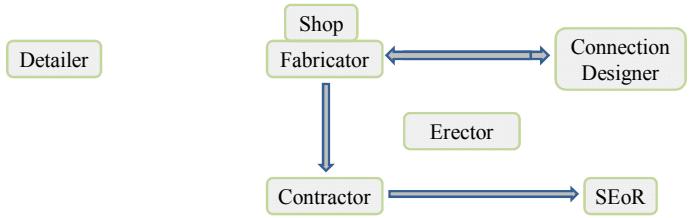
Getting to an IFF



Fabricator submits samples to Contractor, who submits to SEoR

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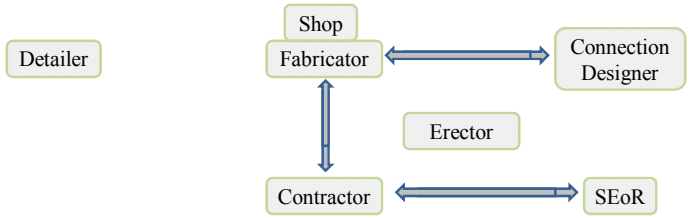
Getting to an IFF



SEoR reviews and either approves or comments on submittals and returns to Contractor who returns to Fabricator/Connection Designer

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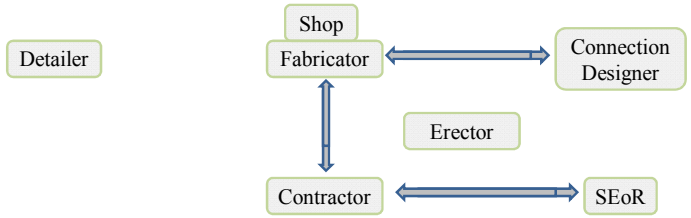
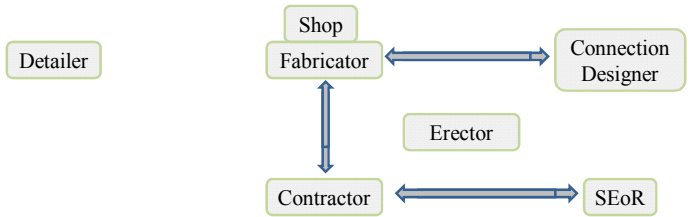
Getting to an IFF



Something is not approved by the SEoR…

22

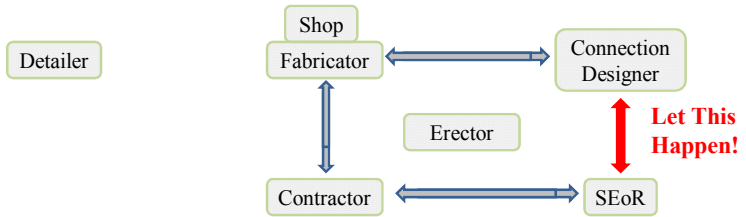
Getting to an IFF



Something is not approved by the SEoR…
This is where things have the potential of breaking down!

23

Getting to an IFF



Something is not approved by the SEoR…
This is where things have the potential of breaking down!

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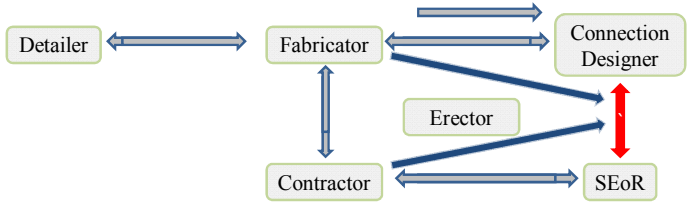
Getting to an IFF



The contractor and fabricator need to be kept in the loop to monitor possible deviations from contract documents…

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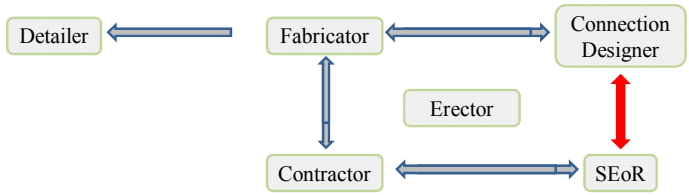
Getting to an IFF



Once approved, connection designer completes connection detail, submits to the Fabricator/Detailer

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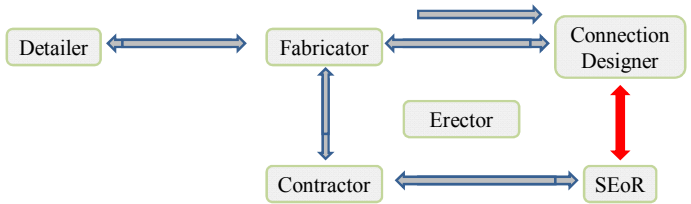
Getting to an IFF



Detailer produces CHECKED shop drawings; submits to Fabricator and Connection Designer for approval.

COSP now refers to these as "Fabrication Documents"

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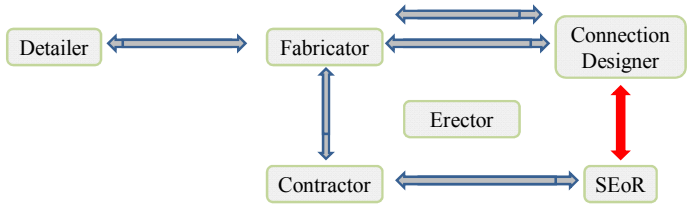
Getting to an IFF



Once Fabricator and Connection Designer approve the shop drawing, Fabricator submits to SEoR through the Contractor for approval

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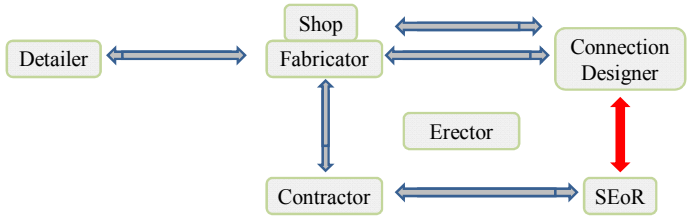
Getting to an IFF



SEoR approves Shop Drawings or comments

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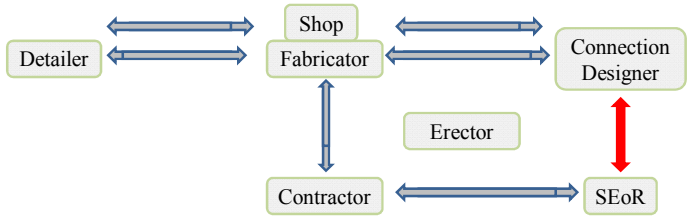
Getting to an IFF



With an approved shop drawing, detailer produces Issued For Fabrication Drawings, submits to Fabricator who submits to shop for fabrication

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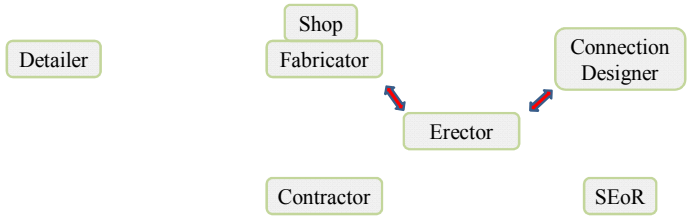
Getting to an IFF



To be really effective, the erector should be in the loop through the entire process. Fabricators and connection Designers should get erector input early on and often.

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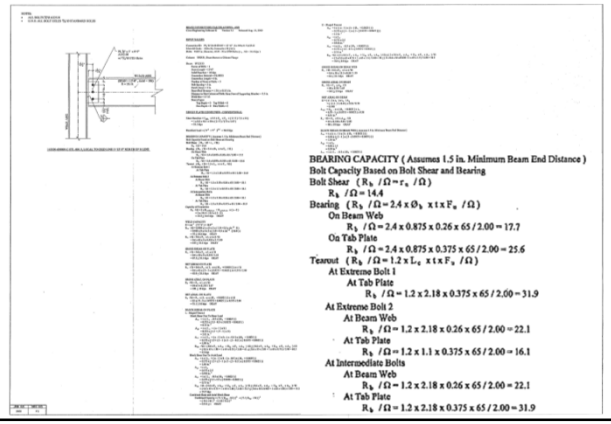
Sample Calculation



BEARING CAPACITY (Assumes 1.5 in. Minimum Beam End Distance)
Bolt Capacity Based on Bolt Shear and Bearing
Bolt Shear ($R_n/\Omega = r_n/\Omega$)
$R_n/\Omega = 14.4$
Bearing ($R_n/\Omega = 2.4 \times \Phi_n \times t \times F_u/\Omega$)
On Beam Web
$R_n/\Omega = 2.4 \times 0.875 \times 0.26 \times 65 / 2.00 = 17.7$
On Tab Plate
$R_n/\Omega = 2.4 \times 0.875 \times 0.375 \times 65 / 2.00 = 25.6$
Tearout ($R_n/\Omega = 1.2 \times L_c \times t \times F_u/\Omega$)
At Extreme Bolt 1
At Tab Plate
$R_n/\Omega = 1.2 \times 2.18 \times 0.375 \times 65 / 2.00 = 31.9$
At Extreme Bolt 2
At Beam Web
$R_n/\Omega = 1.2 \times 2.18 \times 0.26 \times 65 / 2.00 = 22.1$
At Tab Plate
$R_n/\Omega = 1.2 \times 1.1 \times 0.375 \times 65 / 2.00 = 16.1$
At Intermediate Bolts
At Beam Web
$R_n/\Omega = 1.2 \times 2.18 \times 0.26 \times 65 / 2.00 = 22.1$
At Tab Plate
$R_n/\Omega = 1.2 \times 2.18 \times 0.375 \times 65 / 2.00 = 31.9$

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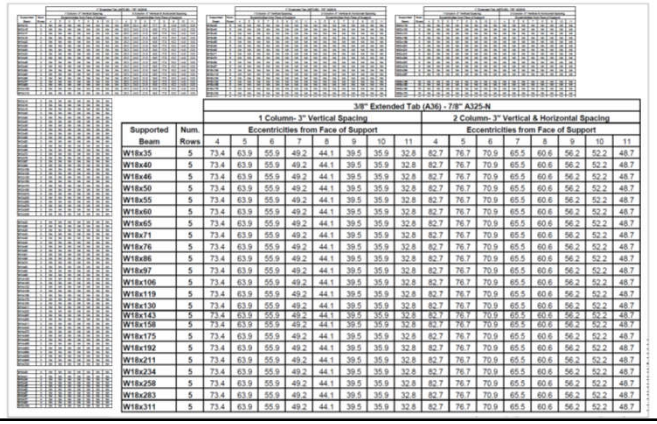
Communication with Detailer



3/8" Extended Tab (A36) - 7/8" A325-N

Supported Beam	Num. Rows	1 Column- 3" Vertical Spacing — Eccentricities from Face of Support								2 Column- 3" Vertical & Horizontal Spacing — Eccentricities from Face of Support							
		4	5	6	7	8	9	10	11	4	5	6	7	8	9	10	11
W18x35	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x40	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x46	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x50	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x55	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x60	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x65	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x71	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x76	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x86	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x97	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x106	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x119	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x130	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x143	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x158	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x175	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x192	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x211	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x234	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x258	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x283	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7
W18x311	5	73.4	63.9	55.9	49.2	44.1	39.5	35.9	32.8	82.7	76.7	70.9	65.5	60.6	56.2	52.2	48.7

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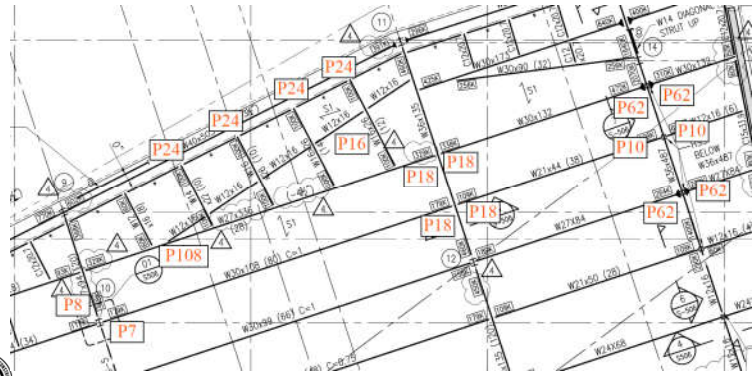
Communication with SEoR



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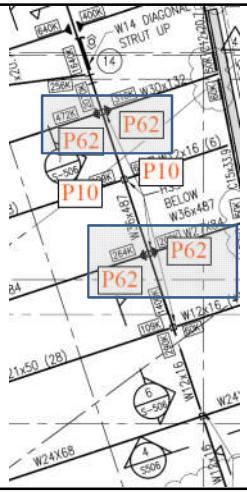
Communication with SEoR

Although the loads are different, the connection type is the same…
…Same P-sheet

…If you require a separate P-sheet for every connection, it is critical that it is specified in the bid drawings!



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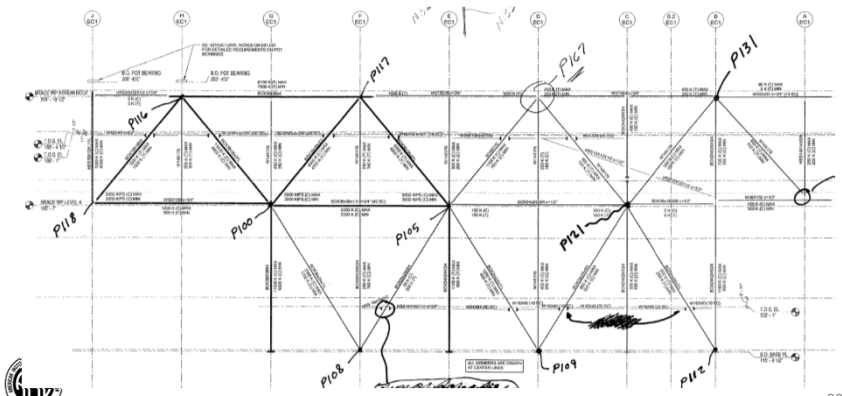
Communication with SEoR



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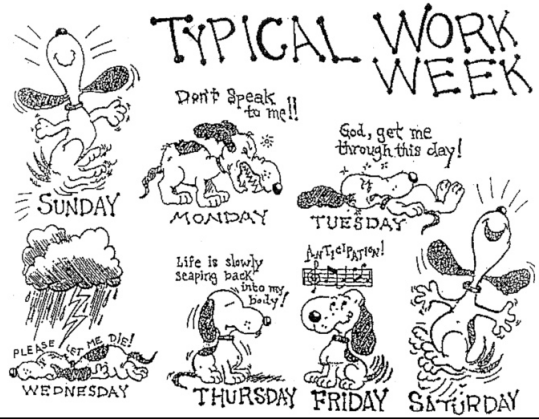
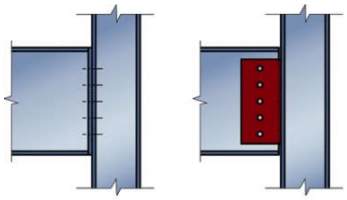
Shear Connections - General



37

Shear Connections - General

- Typically make up the bulk of the connections
- Repetition is key
- For economy and safety, provide the actual design loads
  - Avoid specifying "Maximum Total Uniform Load," .i.e., UDL



38

Shear Connections - General

- Typically make up the bulk of the connections
- Repetition is key
- For economy and safety, provide the actual design loads
  - Avoid specifying UDL
- Consider ease and safety of fabrication
- Consider ease and safety of erection
- Consider fabricator setup and availability
  - Bolted versus welded connections
  - What other projects are running through the shop
  - How is the fabricator set up to operate

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Shear Connections - General

- Consider the required performance of a shear connection
  - Avoid unnecessarily specifying pretensioned or slip-critical joints
- Pretensioned and Slip-Critical joints impact costs
  - Installation verification, surface preparation… time and money
- Pretensioned joints require the same preload as a slip-critical joint
  - It's just that joint slip is not a concern
  - Installation verification is required in a pretensioned joint

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Shear Connections - General

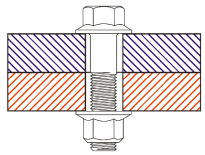
- Consider the required performance of a shear connection
  - Avoid unnecessarily specifying pretensioned or slip-critical joints
- Pretensioned and Slip-Critical joints impact costs
  - Installation verification, surface preparation… time and money
- Assume snug-tight joints
  - No Upper Limit

41

Shear Connections - General

- Consider the required performance of a shear connection
  - Avoid unnecessarily specifying pretensioned and slip-critical joints
- Pretensioned and slip-critical joints impact costs
  - Installation verification … time and money
- Assume snug-tight joints
  - No Upper Limit

Neither RCSC nor AISC places an upper limit on the preload of a bolt in a snug-tight joint



42

Shear Connections - General

- Avoid multi-pass welds in welded joints
  - Varies with process, position and wire/stick; consult with your local fabricator

43

Shear Connections - General

- Avoid multi-pass welds in welded joints
  - Varies with process, position, and wire/stick; consult with your local fabricator
- Or, refer to Table 3.6 of AWS D1.1 2015
  - "Maximum Single Pass Fillet Weld Sizes" are tabulated there
  - Again, consult with local fabricator for process and wire size used in their shop

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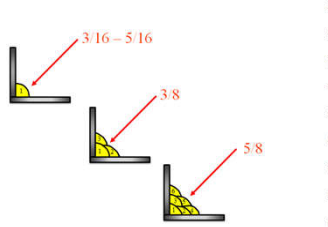
Shear Connections - General

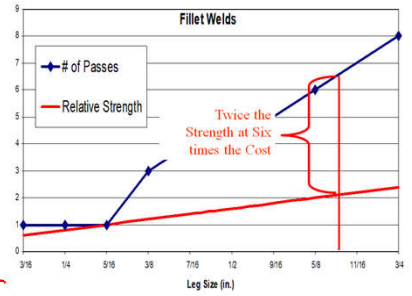
- Avoid multi-pass welds in welded joints
  - Varies with process, position, and wire/stick; consult with your local fabricator
- Or, refer to Table 3.6 of AWS D1.1 2015
  - "Maximum Single Pass Fillet Weld Sizes" are tabulated there
  - Again, consult with local fabricator for process and wire size used in their shop
- Table 8-12 of the *Manual* can be used as a guide; still consult with AWS D1.1 and fabricator for process and position

45

Shear Connections - General

- Avoid multi-pass welds in welded joints
  - Varies with process and wire/stick; consult with your local fabricator



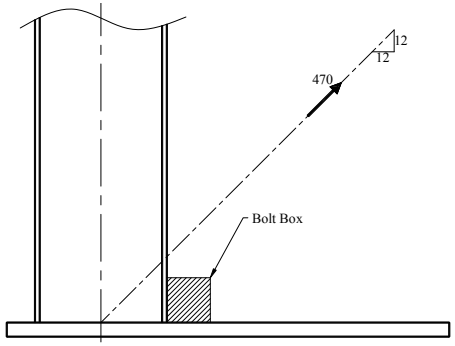


Assumes single pass with 5/16" weld

46

Shear Connections - General

- A slight increase in plate size can significantly reduce the number of passes of welds



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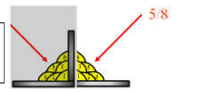
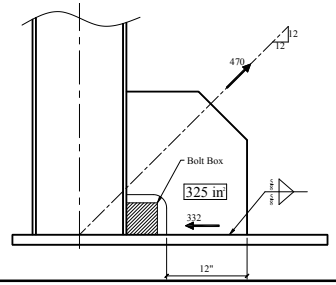
Shear Connections - General

- A slight increase in plate size can significantly reduce the number of passes of welds

(12)-passes at 12"
144 in





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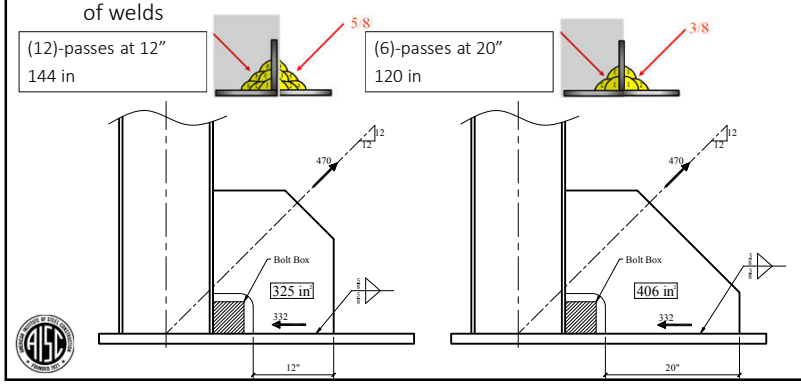
Shear Connections - General

- A slight increase in plate size can significantly reduce the number of passes of welds



49

Poll Question 1

Q. Specifying a pretensioned joint, in lieu of a slip-critical joint, prevents a bolted connection from slipping without having the additional cost associated with installation verification.

a) True: Pretensioned joints have no inspection requirements beyond that of a snug-tight joint

b) True: Pretensioned joints are specified when joint slip is a concern

c) False: Pretensioned joints do not necessarily prevent slip

d) False: Pretensioned joints do not necessarily prevent slip and installation verification is still required.

50

Poll Question 1

Q. Specifying a pretensioned joint, in lieu of a slip-critical joint, prevents a bolted connection from slipping without having the additional cost associated with installation verification.

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b) True: Pretensioned joints are specified when joint slip is a concern

c) False: Pretensioned joints do not necessarily prevent slip

d) False: Pretensioned joints do not necessarily prevent slip and installation verification is still required.

Select your answer!

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Specifications - Notes

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Specifications - Notes

- Provide required material standards
- Allow for equivalent substitutions
- Allow for dual grades

53

Specifications - Notes

- Provide required material standards
- Allow for equivalent substitutions
- Allow for dual grades
- Be aware of the various correction factors for materials

TABLE A3.1
R_y and R_t Values for Steel and Steel Reinforcement Materials

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M	1.5	1.2
• ASTM A1043/A1043M Gr. 36 (250)	1.3	1.1
• ASTM A992/A992M	1.1	1.1
• ASTM A572/A572M Gr. 50 (345) or 55 (380)	1.1	1.1
• ASTM A913/A913M Gr. 50 (345), 60 (415), 65 (450), or 70 (485)	1.1	1.1
• ASTM A588/A588M	1.1	1.1
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
• ASTM A529 Gr. 50 (345)	1.2	1.2
• ASTM A529 Gr. 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500/A500M Gr. B	1.4	1.3
• ASTM A500/A500M Gr. C	1.3	1.2
• ASTM A501/A501M	1.4	1.3
• ASTM A53/A53M	1.6	1.2
• ASTM A1085/A1085M	1.25	1.15
Plates, Strips and Sheets:		
• ASTM A36/A36M	1.3	1.2
• ASTM A1043/A1043M Gr. 36 (250)	1.3	1.1
• ASTM A1011/A1011M HSLAS Gr. 55 (380)	1.1	1.1
• ASTM A572/A572M Gr. 42 (290)	1.3	1.0
• ASTM A572/A572M Gr. 50 (345), Gr. 55 (380)	1.1	1.2
• ASTM A588/A588M	1.1	1.2
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
Steel Reinforcement:		
• ASTM A615/A615M Gr. 60 (420)	1.2	1.2
• ASTM A615/A615M Gr. 75 (520) and Gr. 80 (550)	1.1	1.2
• ASTM A706/A706M Gr. 60 (420) and Gr. 80 (550)	1.2	1.2

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Specifications - Notes

- Provide required material standards
- Allow for equivalent substitutions
- Allow for dual grades
- Be aware of the various correction factors for materials

ASTM A36

$R_y F_y = (1.3)(36.0 \text{ ksi})$

$R_y F_y = 46.8 \text{ ksi}$

ASTM A572-50

$R_y F_y = (1.1)(50.0 \text{ ksi})$

$R_y F_y = 55.0 \text{ ksi}$

TABLE A3.1
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Specifications - Notes

- Note that we do not use correction factors in typical connection design out of the Specification
- But be aware…
- …e.g., protecting welds and bolts in single plate shear connections…

$$t_{\max} = \frac{6M_{\max}}{F_y l^2}$$

- …e.g., shear plate to HSS wall and relative yield strengths of plate and wall…

TABLE A3.1
R_y and R_t Values for Steel and Steel Reinforcement Materials

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Specifications - Notes

- If a particular type of connection is not permitted, clearly note that in the Specifications/Notes/Drawings
- During the approval process is not the time!
- Common source of disputes!

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Specifications - Notes

- Try not to over-constrain with conceptual details

- Allow the fabricator and connection designer flexibility

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Specifications - Notes

- Typically,
  - Calculations for one location for each repetitive connection type are submitted
  - Calculations for every "one-off" connection are submitted

If a full set of calculations for every connection is required, note that clearly; fabricators and their estimators need to capture the costs associated with such a requirement during bid preparation

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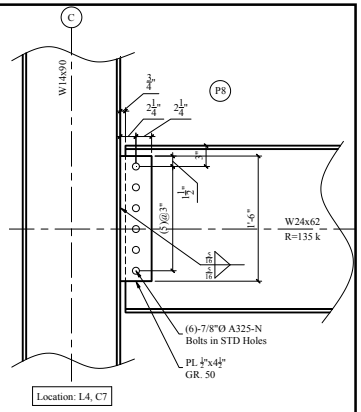
Providing Loads

- Design every connection
- Every bolt, inch of weld, and 1/8" of plate thickness adds cost, weight, and time



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Providing Loads

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Providing Loads

- Cliff spoke a bit about UDL loads (maximum Uniform Design Load) in Session 2
- This is important enough to reinforce that discussion.

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Providing Loads

Excellent Example

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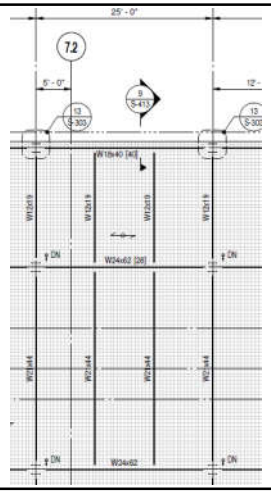
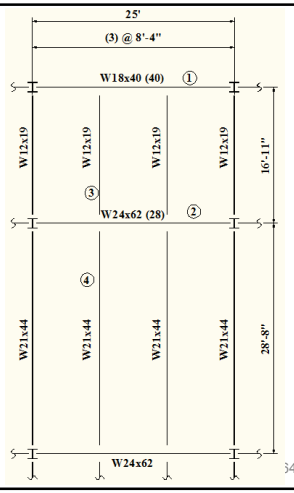
Providing Loads





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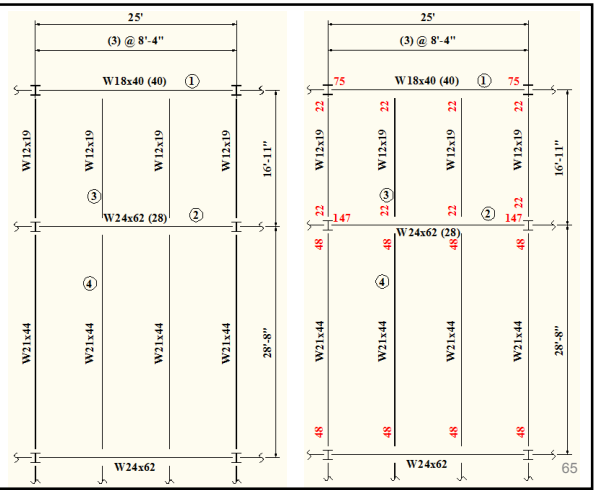
Providing Loads

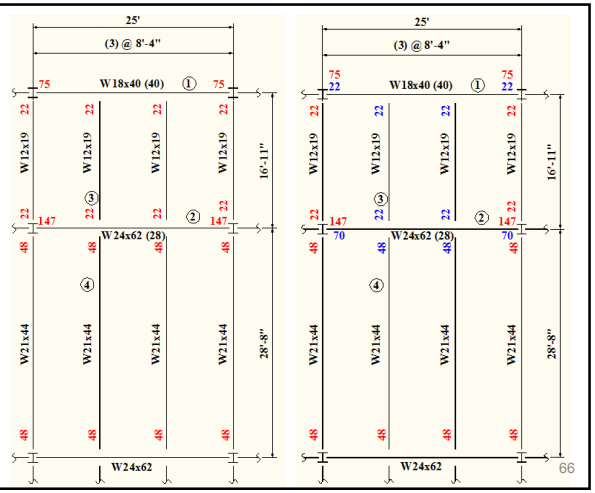
The **red values** shown at beam ends are the required strengths pulled from Table 3-6 of the AISC *Manual* (14th ED.)



Providing Loads

The **blue values** shown at beam ends are maximum girder reactions that the beams can deliver.

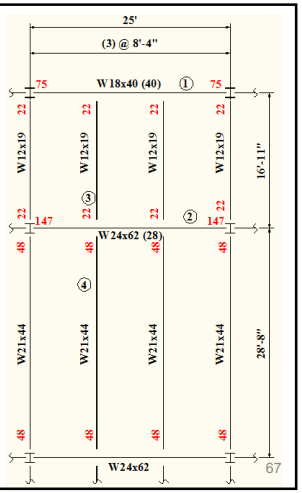
The W12x19 and W21x44 beam end connection strengths are significantly undersized for delivering 147 kips to the girder ends.



Providing Loads

Let q_i = maximum floor load based on provided shear strength:

Girder 1: $q_1 = \frac{2R_1}{L_1(TL)_1} = \frac{(2)(75kips)(1{,}000lbs/kip)}{(25ft)(8.46ft)} = 709psf$



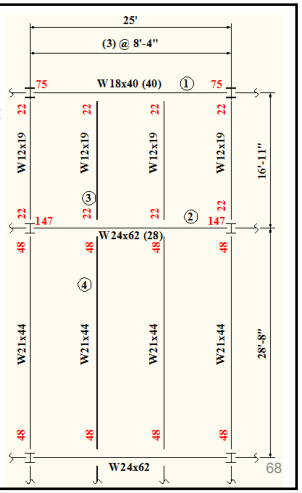
Providing Loads

Let q_i = maximum floor load based on provided shear strength:

Girder 1: $q_1 = \frac{2R_1}{L_1(TL)_1} = \frac{(2)(75kips)(1{,}000lbs/kip)}{(25ft)(8.46ft)} = 709psf$

Girder 2: $q_2 = \frac{2R_2}{L_2(TL)_2} = \frac{(2)(147kips)(1{,}000lbs/kip)}{(25ft)(22.8ft)} = 516psf$

It's possible, but unlikely, that the design floor load for this commercial building approached 516 *psf* let alone 709 *psf*.



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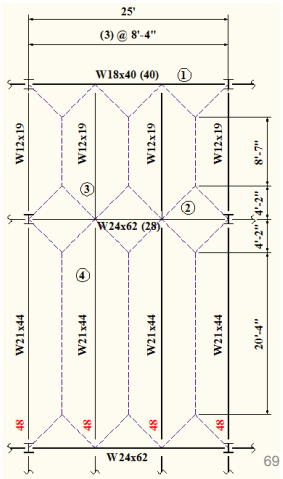
Providing Loads

Evaluate q_i based on a more detailed tributary area (two-way action)



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Providing Loads

Let q_i = maximum floor load based on provided shear strength:

Girder 1:

$$q_1 = \frac{R_1}{(TA)_1} = \frac{(75kips)(1{,}000lbs/kip)}{(8.46ft)(8.33ft)+(0.5)(4.167ft)(4.167ft)}$$

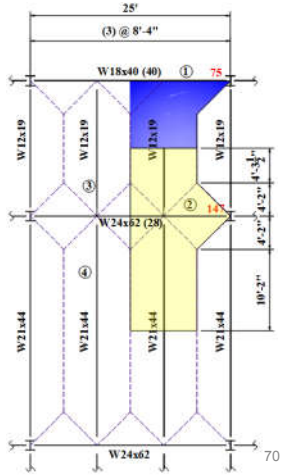
$$q_1 = 947psf$$

Girder 2:

$$q_2 = \frac{R_2}{(TA)_2} = \frac{(147kips)(1{,}000lbs/kip)}{(22.8ft)(8.33ft)+(4.167ft)(4.167ft)}$$

$$q_2 = 709psf$$

Produces a relatively larger design floor load!



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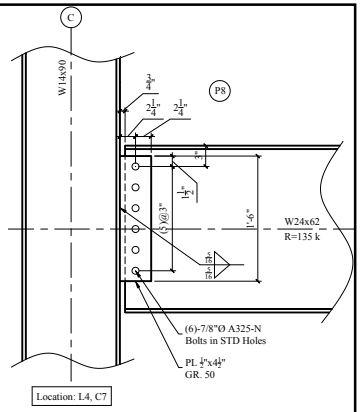
Single Plate Shear Connections

- Design every connection
- Every bolt, inch of weld, and 1/8" of plate thickness adds cost, weight, and time



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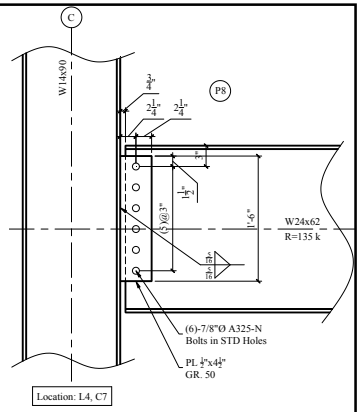
Single Plate Shear Connections

- Well defined analysis and design procedure
- Relatively ease of fabrication
- Very safe for erection



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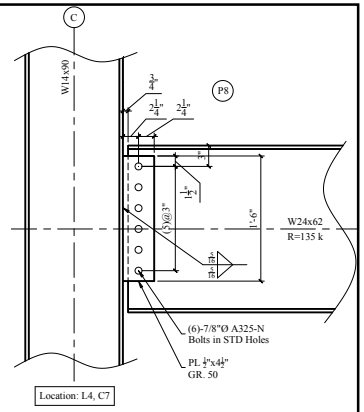
Single Plate Shear Connections

- Make the plate symmetrical
  - Eliminates fitter installing plate in incorrect orientation
- Punch-down is typically 3”
- Make sure proud dimension is sufficient to clear fillet welds
- Avoid connection plates with thickness less than 3/8” (personal rule of thumb)



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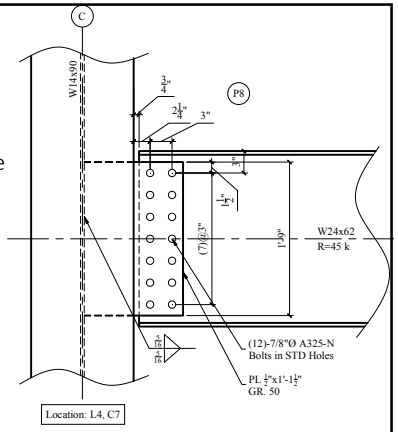
Extended Single Plate Shear Connections

- Design every connection
- Every bolt, inch of weld, and 1/8” of plate thickness adds cost, weight, and time



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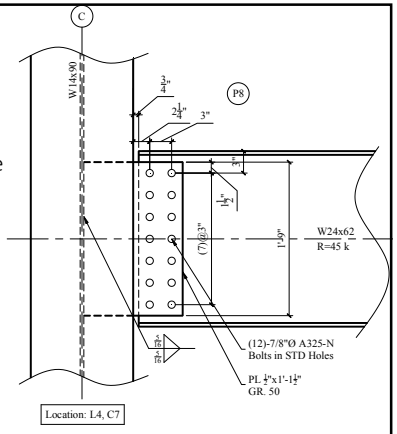
Extended Single Plate Shear Connections

- Design every connection
- Every bolt, inch of weld, and 1/8” of plate thickness adds cost weight, and time
- Take advantage of depth
  - Reduce eccentricity
  - Reduce bolt columns
  - Reduce plate thickness
  - Reduce plate-to-web weld



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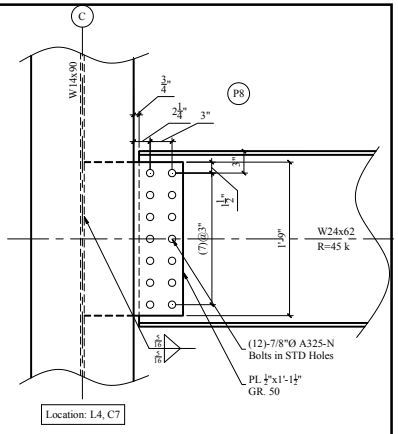
Extended Single Plate Shear Connections

- Use Gr 50 plate material
  - Reduces thickness required for bending and shear relative to A36
    - Reduces weld size



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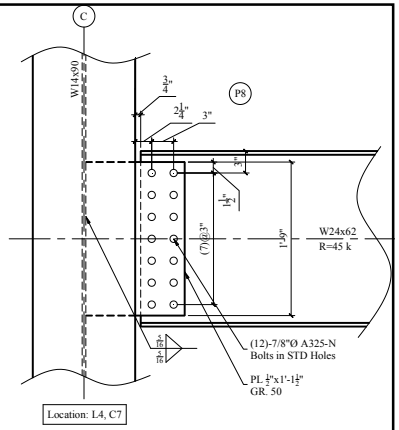
Extended Single Plate Shear Connections

- Use Gr 50 plate material
  - Reduces thickness required for bending and shear relative to A36
    - Reduces weld size
  - It does, however, reduce t_{max}
    - Again, reduces weld size



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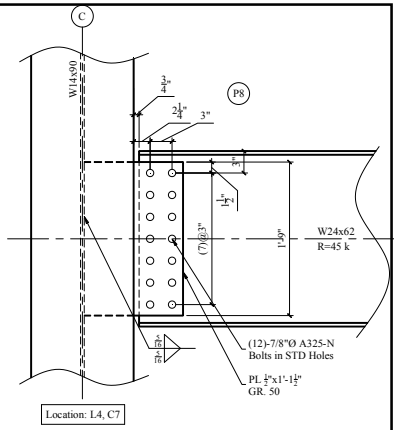
Extended Single Plate Shear Connections

- Single plate shear connections are disputably the most desirable shear connection with fabricators, designers, and erectors



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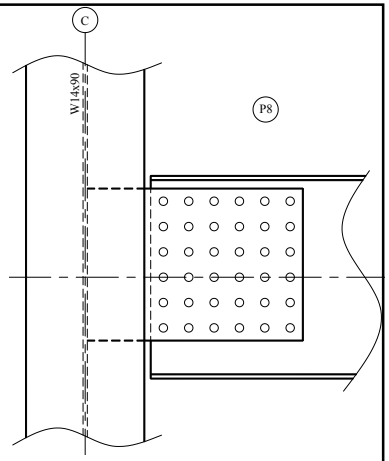
Extended Single Plate Shear Connections

- Single plate shear connections are disputably the most desirable shear connection with fabricators, designers, and erectors
- But, we need to be reasonable!
- We have other options…



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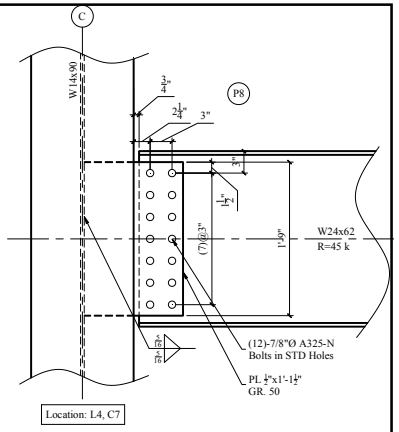
Extended Single Plate Shear Connections

- For those of you concerned about stability in one-sided connections…
- Refer to *Thornton and Fortney
- This is a connection that has been tested extensively (conventional and extended)…
  - …Search on the EJ website, www.aisc.org; dozens of articles



*Thornton, W.A., and Fortney, P.J. (2011). "On the Need for Stiffeners for and the Effect of Lap Eccentricity on Extended Shear Tabs," Engineering Journal, *American Institute of Steel Construction*, Chicago, Ill., V. 48, No. 2, pp.117-125

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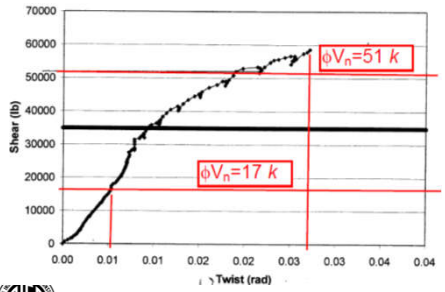
Single Plate Shear Connections

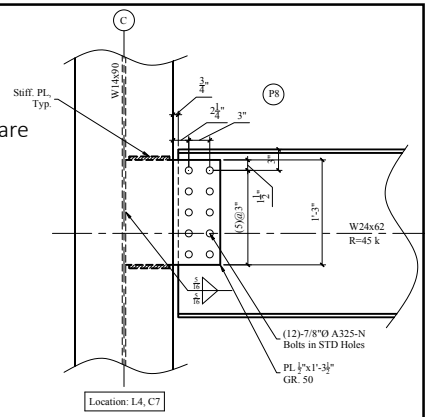
- For those of you concerned about stability in one-sided connections…



Sherman and Ghorbanpoor (2002)

Extended Single Plate Shear Connections

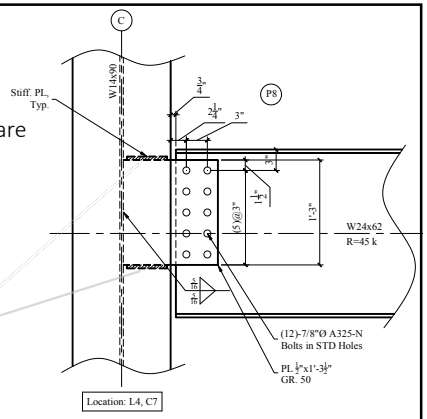
- I've worked projects where stiffeners are required without discussion
  - Specifying stiff. PL up to one inch thick…



Extended Single Plate Shear Connections

- I've worked projects where stiffeners are required without discussion
  - Specifying stiff. PL up to one inch thick…

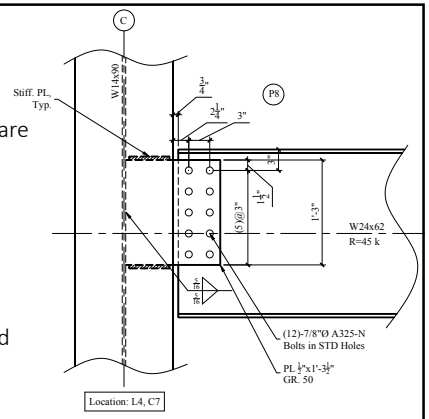
These plates are sometimes referred to as "stabilizer" plates.



Extended Single Plate Shear Connections

- I've worked projects where stiffeners are required without discussion
  - Specifying stiff. PL up to one inch thick…
- Rational checks show they are not required…
- Thornton and Fortney (2011) provided methods for limit state checks
  - LTB, Lap Eccentricity

If you have to use stiffeners, do not connect to column web



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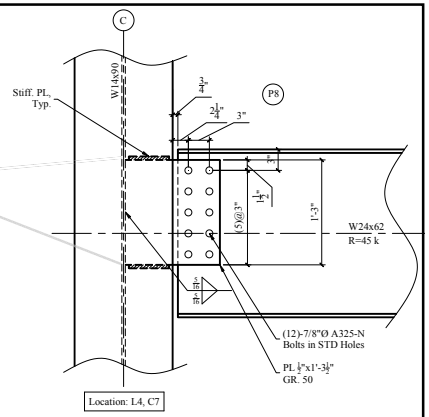
Extended Single Plate Shear Connections

If you simply can't live without "stabilizer" plates, do not attach to column web

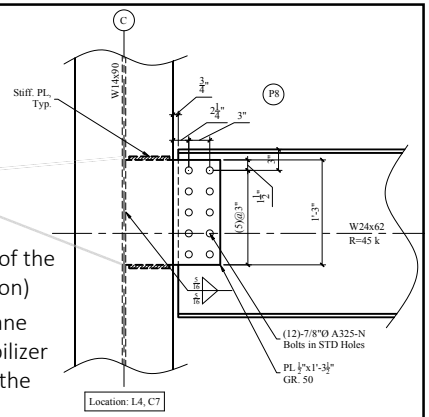


85

Extended Single Plate Shear Connections

If you simply can't live without "stabilizer" plates, do not attach to column web

- The stabilizer plates change the behavior of the connection (change the boundary condition)
  - To what degree depends on the in-plane flexural and shear stiffness of the stabilizer plate (the thicker the plate the stiffer the plate)

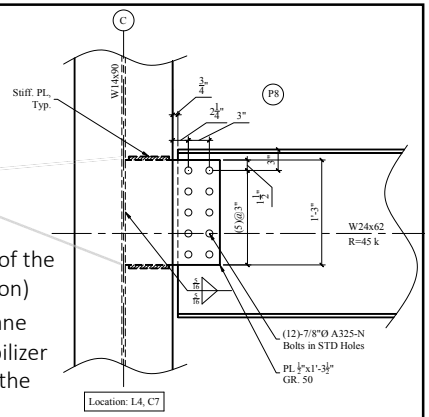


86

Extended Single Plate Shear Connections

If you simply can't live without "stabilizer" plates, do not attach to column web

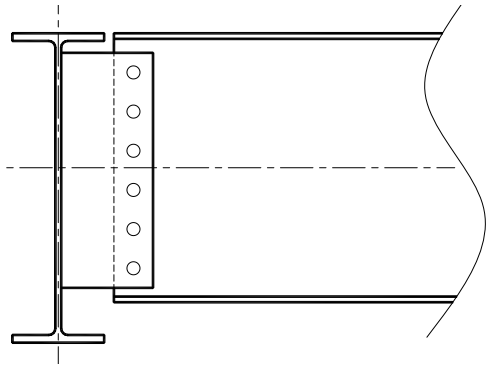
- The stabilizer plates change the behavior of the connection (change the boundary condition)
  - To what degree depends on the in-plane flexural and shear stiffness of the stabilizer plate (the thicker the plate the stiffer the plate



Attaching the stabilizer plate to the web significantly exacerbates the issue

87

Extended Single Plate Shear Connections – One-Sided/Spandrel



88

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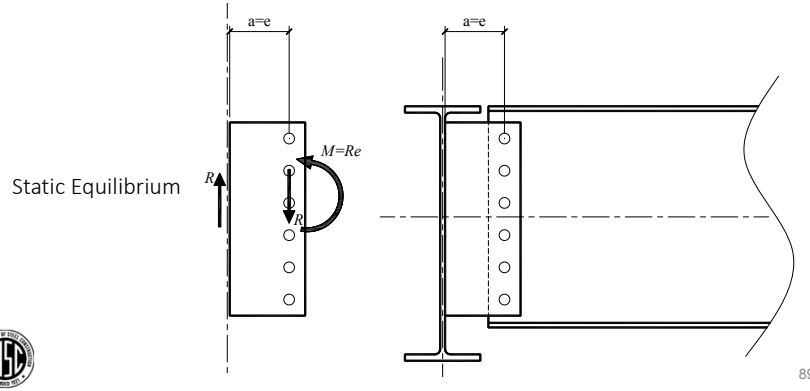
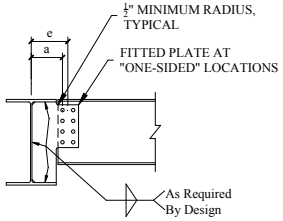
Extended Single Plate Shear Connections – One-Sided/Spandrel



89

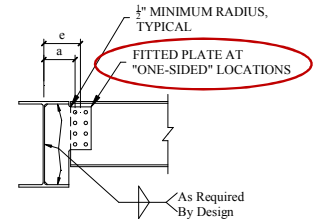
Extended Single Plate Shear Connections – One-Sided/Spandrel



1. A ONE-SIDED LOCATION IS:
   a. ANY BEAM FRAMING TO A PERIMETER BEAM
   b. ANY BEAM FRAMING TO A GIRDER WHERE NO OPPOSITE BEAM IS 36" OR FURTHER FROM BEAM LOCATION.
2. THE FULL ECCENTRICITY, "e" OF THE CONNECTION MUST BE CONSIDERED IN DESIGN OF CONNECTION.

90

Extended Single Plate Shear Connections – One-Sided/Spandrel

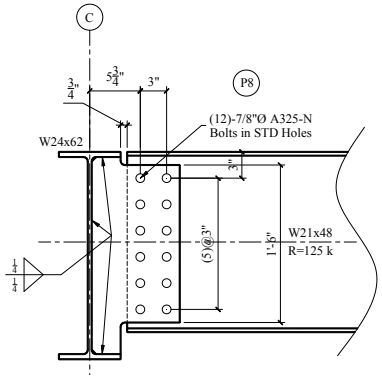


1. A ONE-SIDED LOCATION IS:
   a. ANY BEAM FRAMING TO A PERIMETER BEAM
   b. ANY BEAM FRAMING TO A GIRDER WHERE NO OPPOSITE BEAM IS 36" OR FURTHER FROM BEAM LOCATION.
2. THE FULL ECCENTRICITY, "e" OF THE CONNECTION MUST BE CONSIDERED IN DESIGN OF CONNECTION.

91

Extended Single Plate Shear Connections – One-Sided/Spandrel

- Note that deck and slab, not shown in the sketches provide restraint…



92

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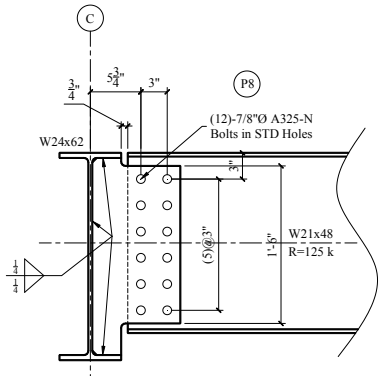
Extended Single Plate Shear Connections – One-Sided/Spandrel

The boundary conditions have been changed…

…this is no longer a simple shear connection

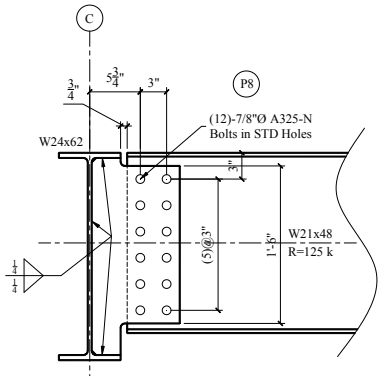


93

Extended Single Plate Shear Connections – One-Sided/Spandrel

The boundary conditions have been changed…

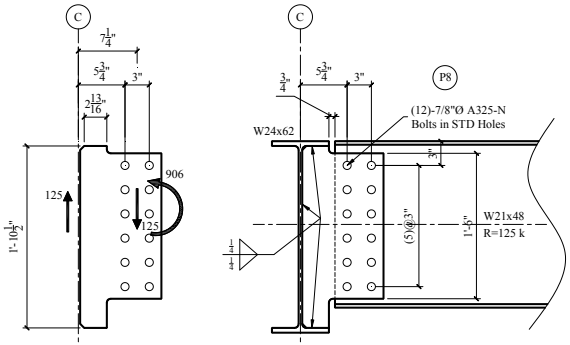
…this is no longer a simple shear connection



Has the SEoR accounted for this in the design of the supporting (spandrel) beam?

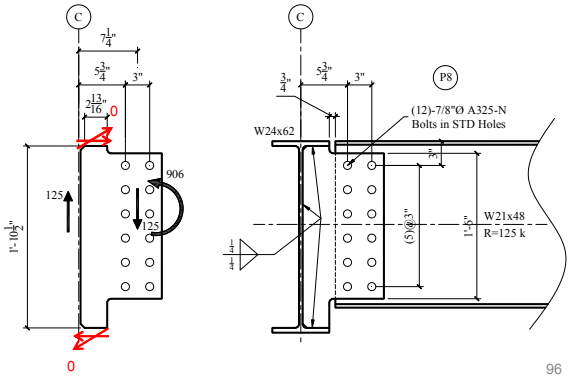
94

Extended Single Plate Shear Connections – One-Sided/Spandrel



95

Extended Single Plate Shear Connections – One-Sided/Spandrel



96

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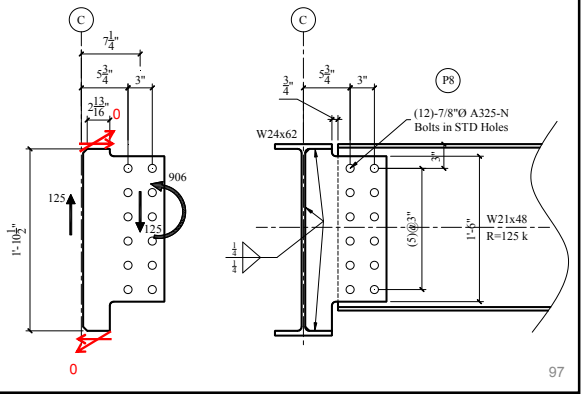
Extended Single Plate Shear Connections – One-Sided/Spandrel

$\phi R_w = 1.392DL$

$\phi R_w = (1.392)(4)(2)(2.5625)$

$\phi R_w = 28.5$ k



97

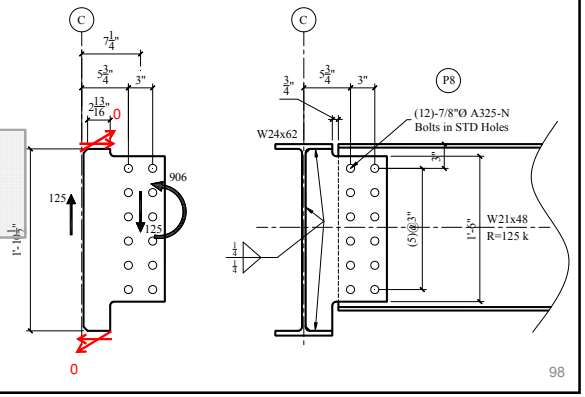
Extended Single Plate Shear Connections – One-Sided/Spandrel

$\phi R_w = 1.392DL$

$\phi R_w = (1.392)(4)(2)(2.5625)$

$\phi R_w = 28.5$ k

No Design Load… …minimum Weld Size



98

Extended Single Plate Shear Connections – One-Sided/Spandrel

$\phi R_w = 1.392DL$

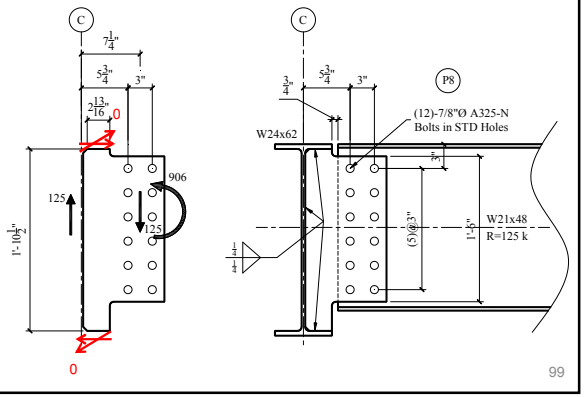
$\phi R_w = (1.392)(4)(2)(2.5625)$

$\phi R_w = 28.5$ k

$\phi R_b = 24.3$ k/bolt

$C = 6.31$ bolts

$\phi R_b = 153$ k



99

Extended Single Plate Shear Connections – One-Sided/Spandrel

$\phi R_w = 1.392DL$

$\phi R_w = (1.392)(4)(2)(2.5625)$

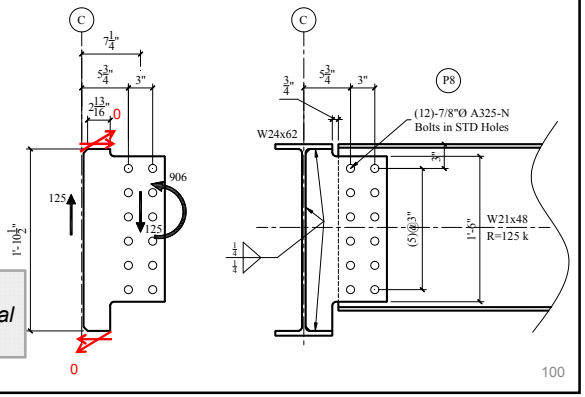
$\phi R_w = 28.5$ k

$\phi R_b = 24.3$ k/bolt

$C = 6.31$ bolts

$\phi R_b = 153$ k

Effective number of bolts; refer to *Manual* Table 7-7



100

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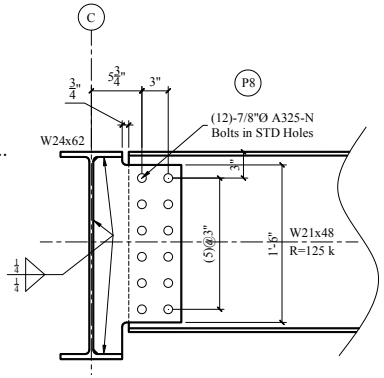
Extended Single Plate Shear Connections – One-Sided/Spandrel

Assumption:

Given the change of boundary condition…

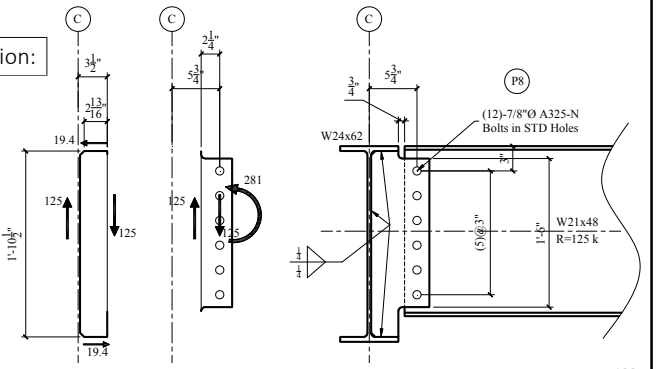
…can we look at this differently?



101

Extended Single Plate Shear Connections – One-Sided/Spandrel

Unverified Assumption:



102

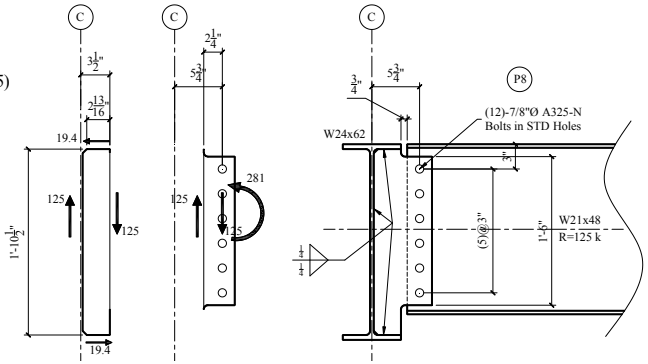
Extended Single Plate Shear Connections – One-Sided/Spandrel

$\phi R_w = 1.392DL$

$\phi R_w = (1.392)(4)(2)(2.5625)$

$\phi R_w = 28.5$ k

$\phi R_w = 28.5 \text{ k} > 19.4 \text{ k}$



103

Extended Single Plate Shear Connections – One-Sided/Spandrel

$\phi R_w = 1.392DL$

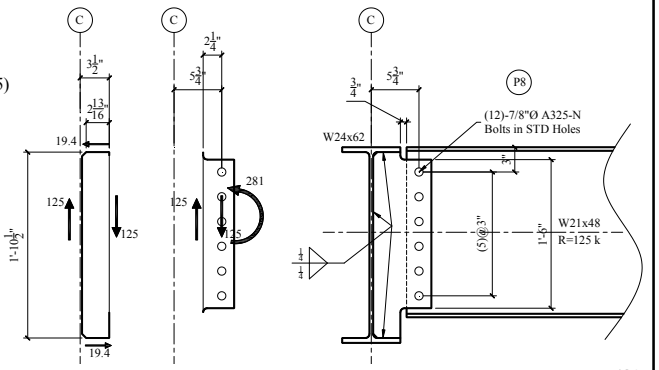
$\phi R_w = (1.392)(4)(2)(2.5625)$

$\phi R_w = 28.5$ k

$\phi R_b = 24.3$ k/bolt

$C = 5.39$ bolts

$\phi R_b = 130$ k



104

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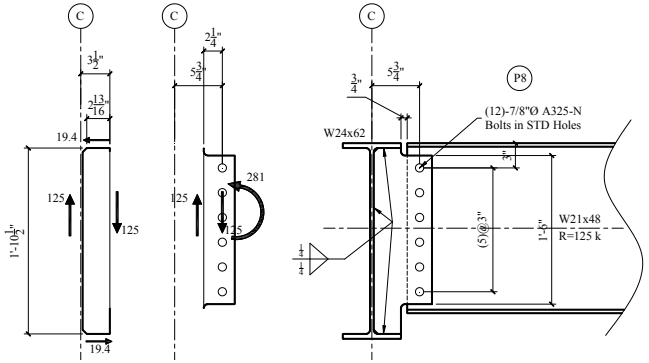
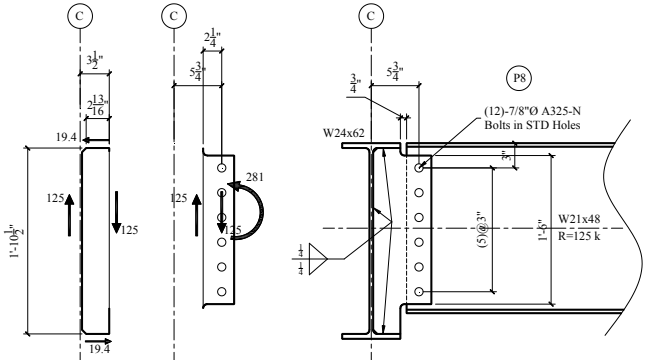
Extended Single Plate Shear Connections – One-Sided/Spandrel



105

Extended Single Plate Shear Connections – One-Sided/Spandrel

If this approach were taken, the SEoR should surely be consulted

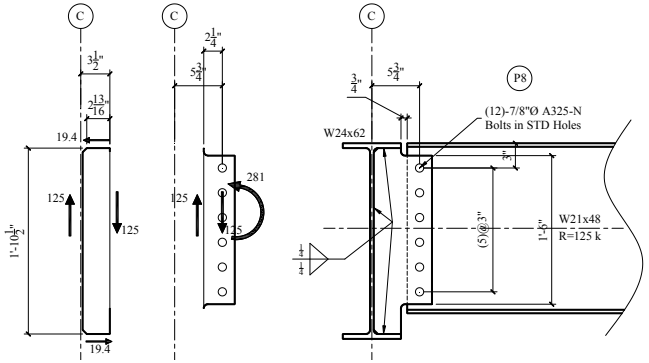


106

Extended Single Plate Shear Connections – One-Sided/Spandrel

If this approach were taken, the SEoR should surely be consulted

These FBDs will be shown on sample calculations submitted to the SEoR

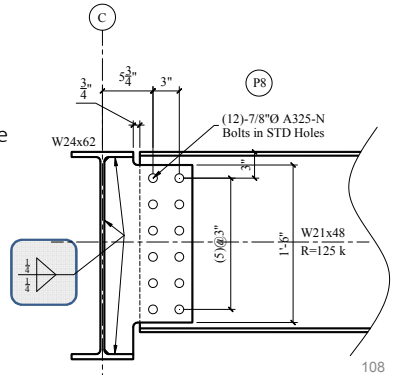


107

Extended Single Plate Shear Connections – One-Sided/Spandrel

Regardless of assumed force distribution…

- Note that a $(5/8)t_p$ weld is not needed here
- Size the vertical and horizontal welds for the loads



108

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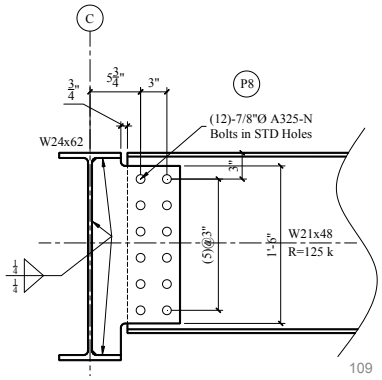
Extended Single Plate Shear Connections – One-Sided/Spandrel

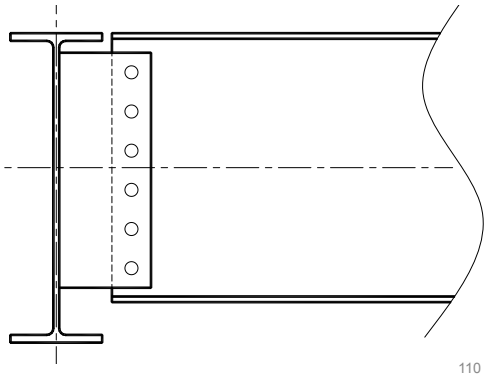
- Fitted plates are expensive and time consuming
  - Shaping, corner snips, radius,
  - Grinding (depending on how cut)…
    - Fit-up issues due to flange tilt, etc.
- Note that deck and slab, not shown in the sketches provide restraint…



109

Extended Single Plate Shear Connections – One-Sided/Spandrel

- Fitted plates are unnecessary
- A typical single plate shear connection is best practice
- Note that deck and slab, not shown in the sketches provide restraint…



110

Poll Question 2

Q. In a conventional single plate shear connection used in a beam-to-column flange connection, if the proud dimension is ¾" and the horizontal edge distance to the bolt hole in the beam web is 1-1/2", good detailing would suggest the width of the connection plate to be…

a) ¾" + 1½" + 1½" = 3¾"
b) ¾" + 1½" + 1½" + ¾" = 4½"
c) At least 3 times the bolt diameter
d) At least 5 times the bolt diameter

111

Poll Question 2

Q. In a conventional single plate shear connection used in a beam-to-column flange connection, if the proud dimension is ¾" and the horizontal edge distance to the bolt hole in the beam web is 1-1/2", good detailing would suggest the width of the connection plate to be…

a) ¾" + 1½" + 1½" = 3¾"
b) ¾" + 1½" + 1½" + ¾" = 4½"
c) At least 3 times the bolt diameter
d) At least 5 times the bolt diameter

Select your answer!

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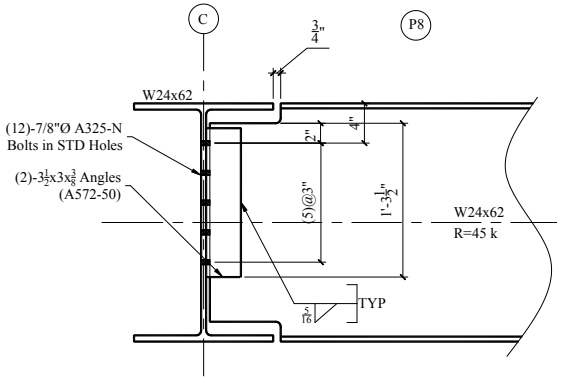
Connections Susceptible to Cracking During Galvanizing

113

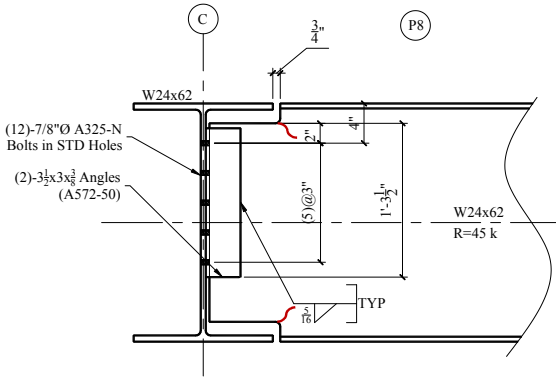
Connections Susceptible to Cracking During Galvanizing



Location: L4, C7

114

Connections Susceptible to Cracking During Galvanizing



Location: L4, C7

115

Connections Susceptible to Cracking During Galvanizing

NOTE that these cracks may not show themselves until under service loads!

"A Synthesis of the Available Technical Literature and Collective Experience for the American Institute of steel Construction," Thomas J. Kinstler, GalvaScience LLC

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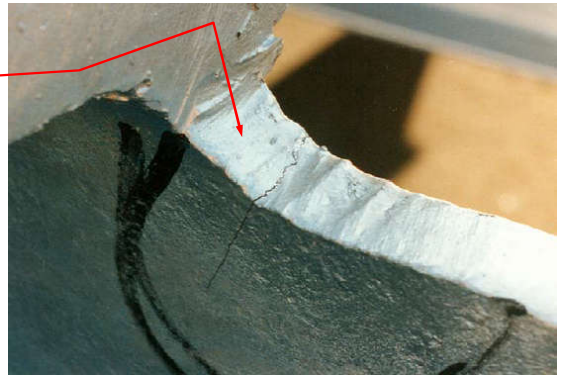
Connections Susceptible to Cracking During Galvanizing

Roughness??
Even when meeting roughness requirements, cracks are still experienced

NOTE that these cracks may not show themselves until under service loads!



"A Synthesis of the Available Technical Literature and Collective Experience for the American Institute of steel Construction," Thomas J. Kinstler, GalvaScience LLC

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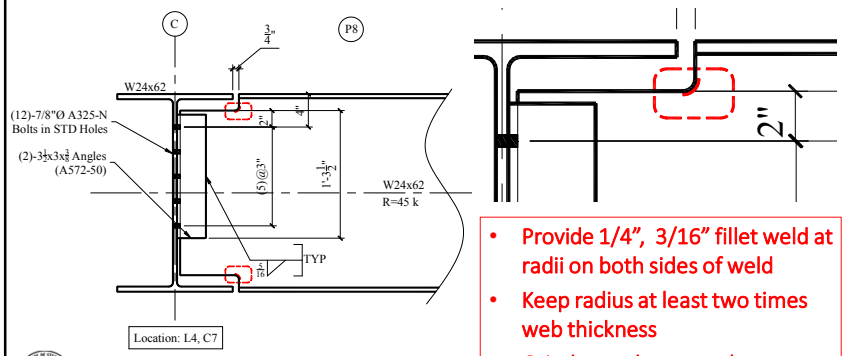
Connections Susceptible to Cracking During Galvanizing

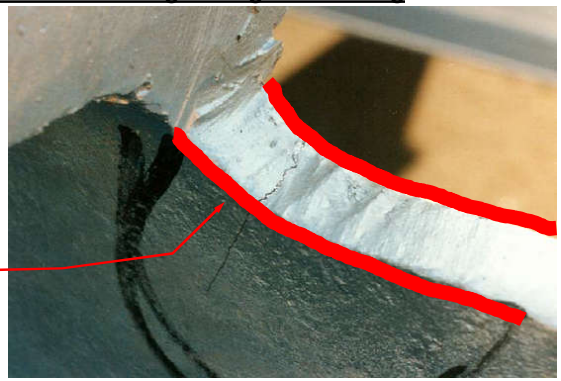


- Provide 1/4", 3/16" fillet weld at radii on both sides of weld
- Keep radius at least two times web thickness
- Grind cut edge smooth

118

Connections Susceptible to Cracking During Galvanizing

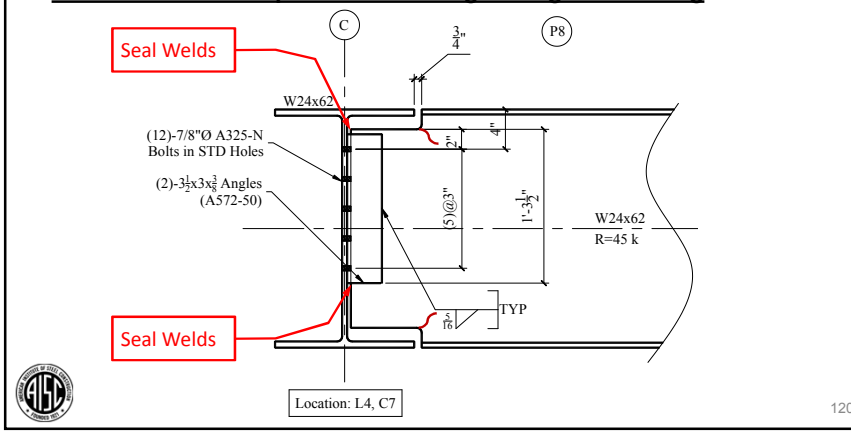
Reinforcement at cope



"A Synthesis of the Available Technical Literature and Collective Experience for the American Institute of steel Construction," Thomas J. Kinstler, GalvaScience LLC

119

Connections Susceptible to Cracking During Galvanizing



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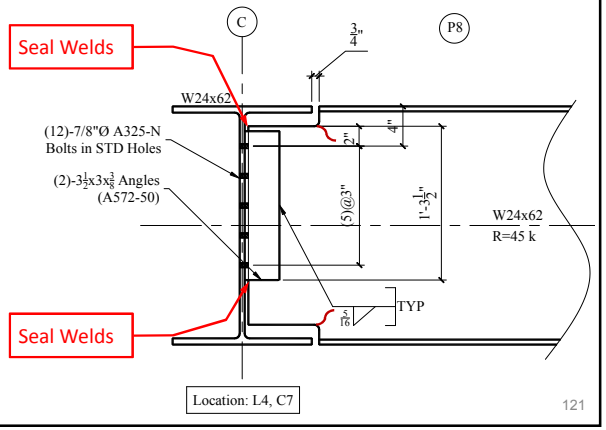
Connections Susceptible to Cracking During Galvanizing

NOTE that wrapping welds around an opposing corner used to be explicitly prohibited in AWS D1.1



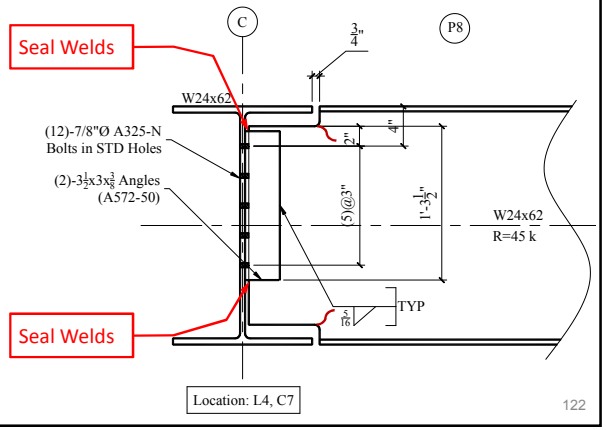
121

Connections Susceptible to Cracking During Galvanizing

NOTE that wrapping welds around an opposing corner used to be explicitly prohibited in AWS D1.1

This is no longer the case!

D1.1 simply cautions in regard to gouging

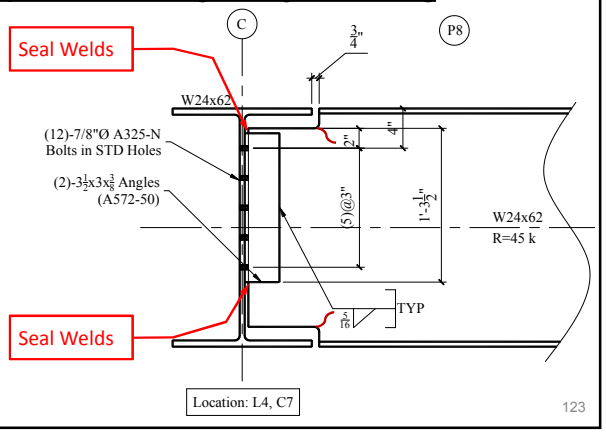


122

Connections Susceptible to Cracking During Galvanizing

The problem still exists, however…

The only change is the fabricator or galvanizer is more susceptible to damages for something mostly out of their control



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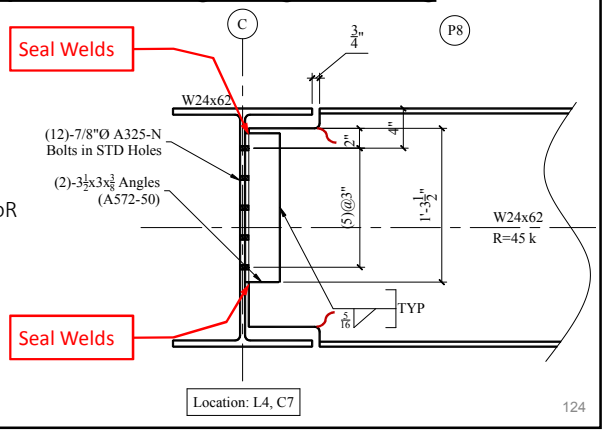
Connections Susceptible to Cracking During Galvanizing

I recommend that it is still good practice for the fabricator to document correspondence to the SEoR of the inherent dangers of these types of seal welds.



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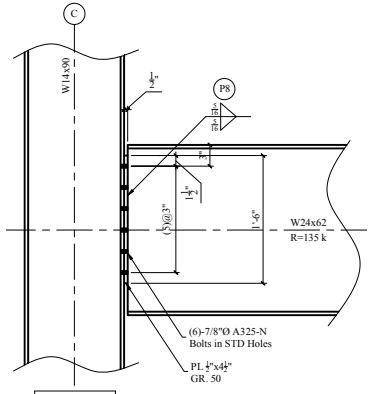
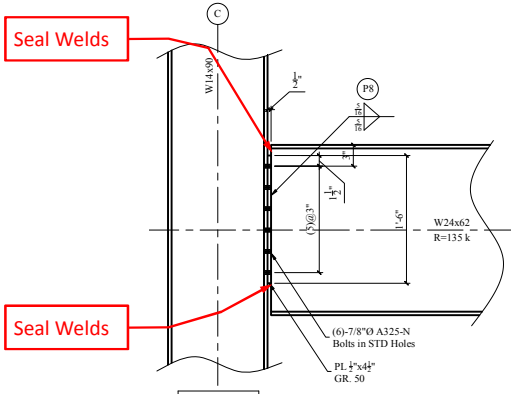
Connections Susceptible to Cracking During Galvanizing



125

Connections Susceptible to Cracking During Galvanizing



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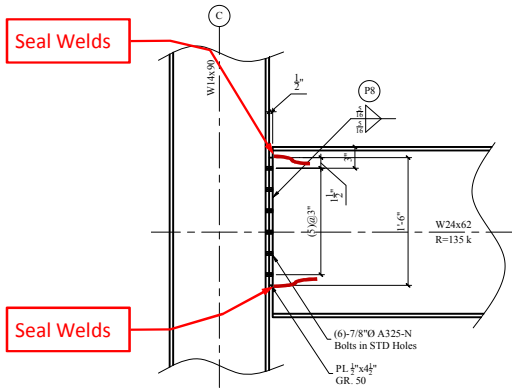
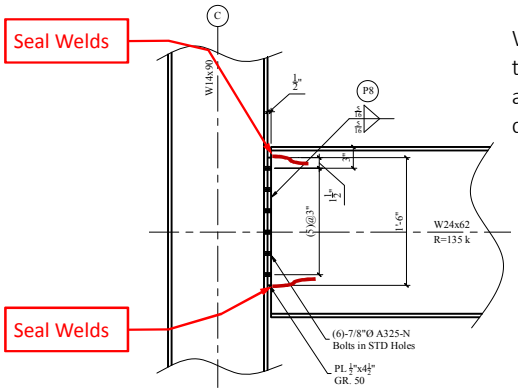
Connections Susceptible to Cracking During Galvanizing



127

Connections Susceptible to Cracking During Galvanizing



We have experienced this in both half-depth and three-quarter depth connections

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Connections Susceptible to Cracking During Galvanizing

In general,

- Make seal welds as small as possible
- Make sure your welders are aware of the dangers of gouging
- Grind cuts and radii smooth prior to galvanizing
- Make radii at least 2 times the plate thickness
- Increase, ramp-up, visual inspections of the parts/assemblies after galvanization and prior to hanging

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Shear 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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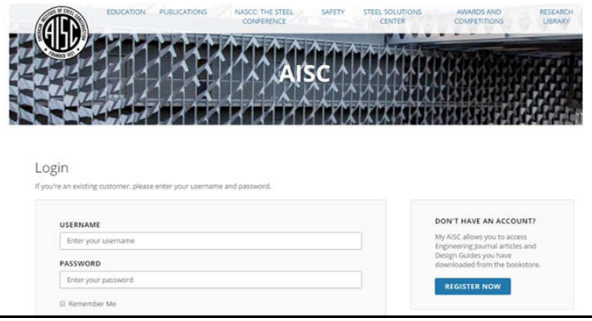
8-Session Registrants

Night School Resources

Go to www.aisc.org and sign in.



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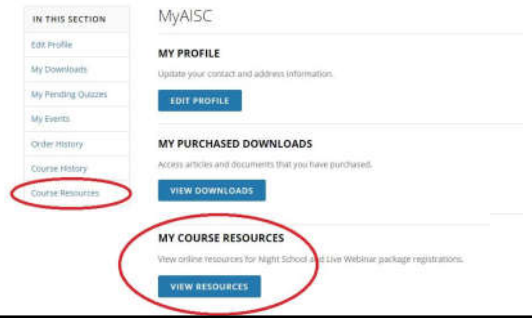
8-Session Registrants

Night School Resources

Go to www.aisc.org and sign in.



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Night School Resources

Course Resources

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Night School Resources

Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

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8-Session Registrants

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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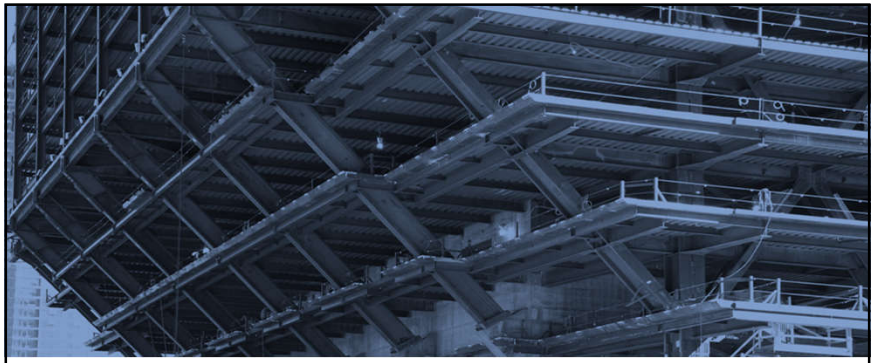

AISC | Thank you

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