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Connection Design

Tips, Tricks, and Lessons Learned

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Course Description

19.7 Seismic Connections
March 25, 2019

From mechanistic design forces to critical welds to protected zones to prequalified connections to exceptions, the SEoR and delegated connection designer need to be in phase with one another before connection design begins. Documented communication on all aspects of the project between the SEoR, fabricator, erector, and inspector is paramount in order to avoid unwelcome shaking of project schedule and budget.

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, fabricator, erector, and inspector to facilitate the review and quality processes.

AISC Night School

Learning Objectives

- Define the term demand critical welds. Identify the sources that give testing requirements and welder qualifications for demand critical welds.
- Explain the concept of expected strength, and its role in the safe detailing of connections in seismic force-resisting structural steel systems.
- Define the term protected zones. Explain where they are found and what special treatment they receive.
- List the seismic-related requirements that should be included on structural design drawings to ensure clear communication.

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

Session 7: Seismic Connections
March 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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Today's Discussion

- All in the context of delegated connection design

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- To understand delegated design work, we need to understand
  - Responsibilities (Larry, Cliff, Carol – Sessions 1, 2, 3, & 5)
  - Work Flow (Larry and Pat (Sessions 1 and 3)
  - Effective Communication (Larry, Cliff, Pat, Carol – Sessions 1-5)

Today's Discussion

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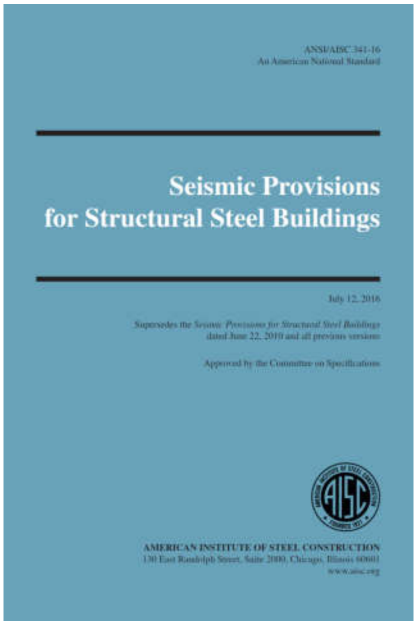
  - Responsibilities (Larry, Cliff, Carol – Sessions 1, 2, 3, & 5)
  - Work Flow (Larry and Pat (Sessions 1 and 3)
  - Effective Communication (Larry, Cliff, Pat, Carol – Sessions 1-5)
- We'll talk a little about
  - Effective Communication
  - General Detailing
  - Specific Issues Related to Seismic Connections

Today's Discussion

- AISC has offered many webinars and Night Schools on the design of connections
- Drucker, Dowswell, Fortney, Muir, Murray, Sabelli, Thornton, etc.
  - Specific examples
  - Number crunching
- The Seismic Design 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 seismic connections

First, A Few General Issues

Specification
for Structural Steel Buildings



ANSI/AISC 341-16
An American National Standard

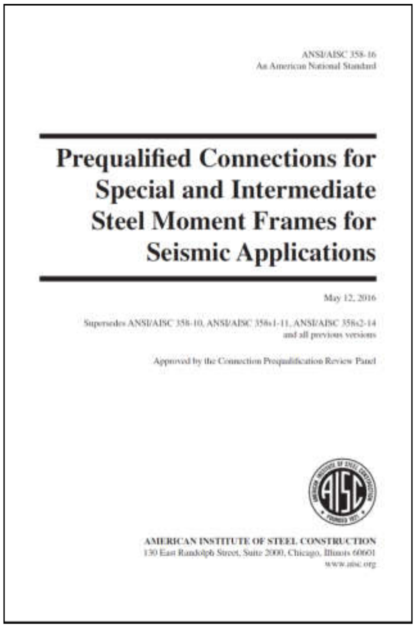
Seismic Provisions
for Structural Steel Buildings

July 12, 2016

Supersedes the *Seismic Provisions for Structural Steel Buildings*
dated June 22, 2010 and all previous versions

Approved by the Committee on Specifications

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
130 East Randolph Street, Suite 2000, Chicago, Illinois 60601
www.aisc.org



ANSI/AISC 358-16
An American National Standard

Prequalified Connections for
Special and Intermediate
Steel Moment Frames for
Seismic Applications

May 12, 2016

Supersedes ANSI/AISC 358-10, ANSI/AISC 358s1-11, ANSI/AISC 358s2-14
and all previous versions

Approved by the Connection Prequalification Review Panel

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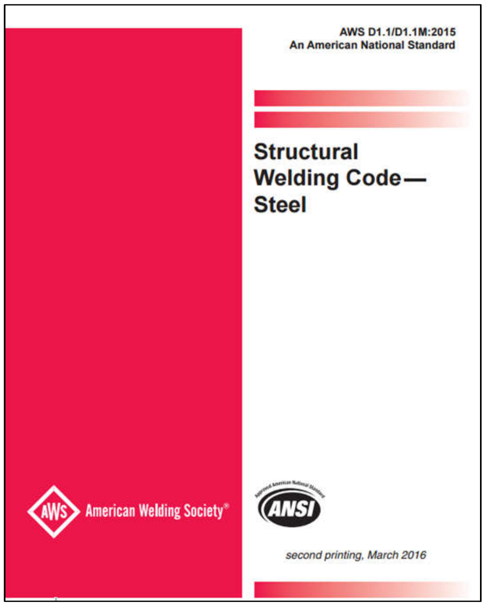
First, A Few General Issues



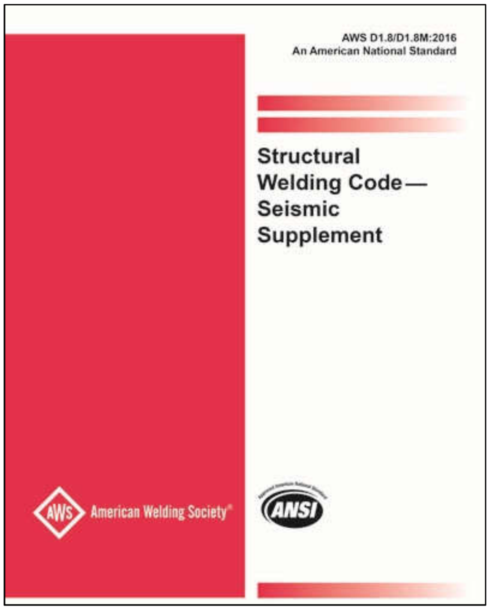
AWS D1.1/D1.1M:2015
An American National Standard

Structural
Welding Code—
Steel

American Welding Society®

ANSI

second printing, March 2016



AWS D1.8/D1.8M:2016
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First, A Few General Issues

- The requirements in the *Seismic Provisions* (AISC 341-16) are in addition to the requirements in the *Specification* (AISC 360-16)
    - The *Provisions* are clear when certain *Specification* requirements are excluded or substituted
        - Otherwise, the *Specification* requirements must still be satisfied

First, A Few General Issues

- Seismic Systems
- Demand Critical Welds
- Special Weld Access Holes and C.P. Corner Snips and Welding
- Expected Material and Available Strengths
- Protected Zones
- Heavy Sections
- Bolted Joints

First, A Few General Issues

Seismic Systems

- Ordinary Moment Frame (OMF)
  - R=3.5
  - Minimal Inelastic Deformation
    - Members and Connections
- Intermediate Moment Frame (IMF)
  - R=4.5
  - Limited Inelastic Deformation
    - Flexural Yielding of Beams and Columns, Panel Zone shear
  - Beam-to-Column Connections must be Qualified or Prequalified
- Special Moment Frame (SMF)
  - R=8
  - Significant Inelastic Deformation
    - Flexural Yielding of Beams
  - Beam-to-Column Connections must be Qualified or Prequalified

First, A Few General Issues

Seismic Systems

- Ordinary Concentrically Braced Frame (OCBF)
  - R=3.25
  - Minimal Inelastic Deformation
    - Members and Connections
- Special Concentrically Braced Frame (SCBF)
  - R=6
  - Significant Inelastic Deformation
    - Brace buckling and Tension Yield
- Eccentrically Braced Frame (EBF)
  - R=8
  - Significant Inelastic Deformation
    - Shear or Flexural Yielding of Link Beams

First, A Few General Issues

Seismic Systems

AISC 341-16 addresses other systems such as:

- Composite Moment Frames
- Composite Braced Frames
- Steel Plate Shear Walls
- Composite Shear Walls (coupled core walls)
- etc.

First, A Few General Issues

DEMAND CRITICAL WELDS

First, A Few General Issues

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There are basically three types of welds that are going to be considered

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First, A Few General Issues

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There are basically three types of welds that are going to be considered

1. AWS D1.1 – welding on members/elements not part of the SFRS (seismic forces resisting system),
2. AWS D1.8 – welding on members/elements part of the SFRS but not designated as demand critical, and

First, A Few General Issues

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3. AWS D1.8 – welding on members/elements part of the SFRS and designated as demand critical

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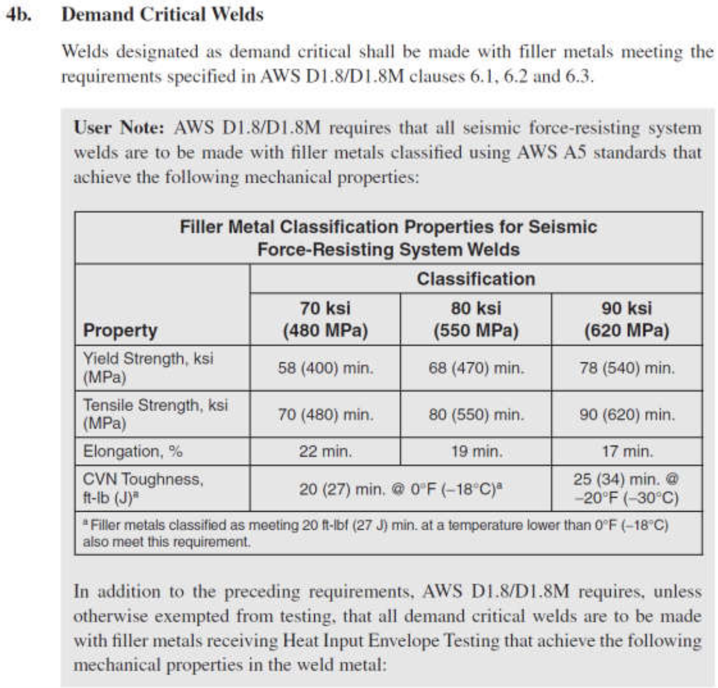
3. AWS D1.8 – welding on members/elements part of the SFRS and designated as demand critical

A demand critical weld is a weld used to join members or elements that are expected to experience significant inelastic demands

Consumables and Demand Critical Welds (341-16, A3.4)



4b. Demand Critical Welds

Welds designated as demand critical shall be made with filler metals meeting the requirements specified in AWS D1.8/D1.8M clauses 6.1, 6.2 and 6.3.

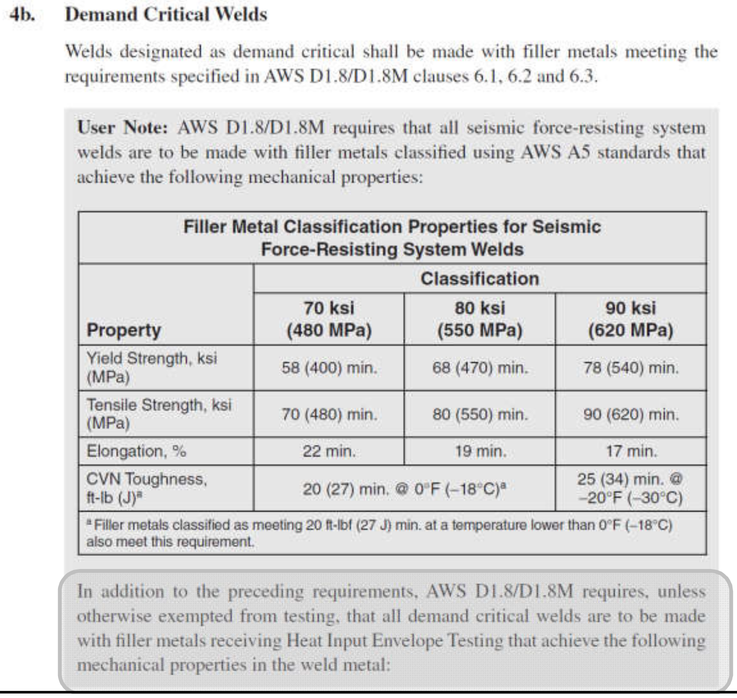
User Note: AWS D1.8/D1.8M requires that all seismic force-resisting system welds are to be made with filler metals classified using AWS A5 standards that achieve the following mechanical properties:

Filler Metal Classification Properties for Seismic Force-Resisting System Welds

Property	Classification		
	70 ksi (480 MPa)	80 ksi (550 MPa)	90 ksi (620 MPa)
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.	78 (540) min.
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.	90 (620) min.
Elongation, %	22 min.	19 min.	17 min.
CVN Toughness, ft-lb (J)[a]	20 (27) min. @ 0°F (−18°C)[a]		25 (34) min. @ −20°F (−30°C)

[a] Filler metals classified as meeting 20 ft-lbf (27 J) min. at a temperature lower than 0°F (−18°C) also meet this requirement.

In addition to the preceding requirements, AWS D1.8/D1.8M requires, unless otherwise exempted from testing, that all demand critical welds are to be made with filler metals receiving Heat Input Envelope Testing that achieve the following mechanical properties in the weld metal:



Consumables and Demand Critical Welds (341-16, A3.4)

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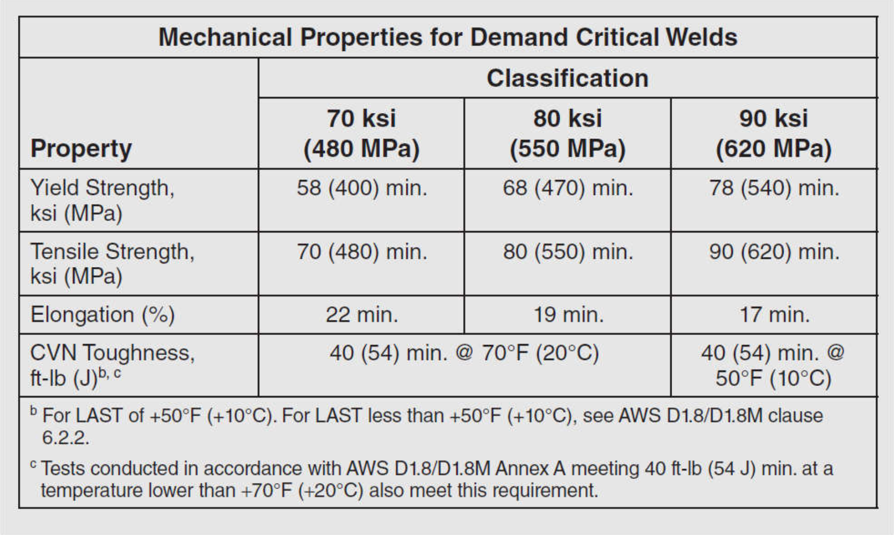

Mechanical Properties for Demand Critical Welds

Property	Classification 70 ksi (480 MPa)	80 ksi (550 MPa)	90 ksi (620 MPa)
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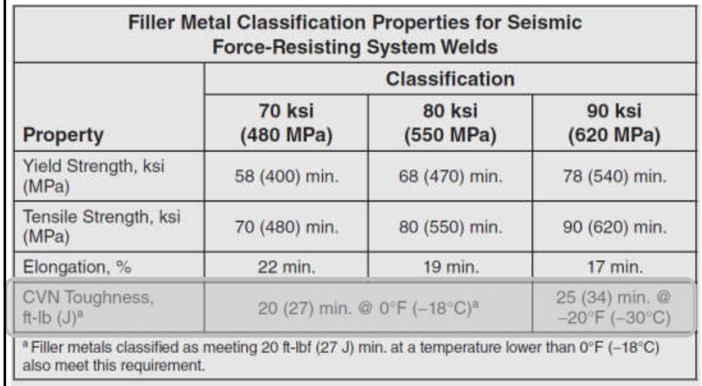
CVN Toughness, ft-lb (J)[b, c]	40 (54) min. @ 70°F (20°C)		40 (54) min. @ 50°F (10°C)

[b] For LAST of +50°F (+10°C). For LAST less than +50°F (+10°C), see AWS D1.8/D1.8M clause 6.2.2.

[c] Tests conducted in accordance with AWS D1.8/D1.8M Annex A meeting 40 ft-lb (54 J) min. at a temperature lower than +70°F (+20°C) also meet this requirement.

Consumables and Demand Critical Welds (341-16, A3.4)



Filler Metal Classification Properties for Seismic Force-Resisting System Welds

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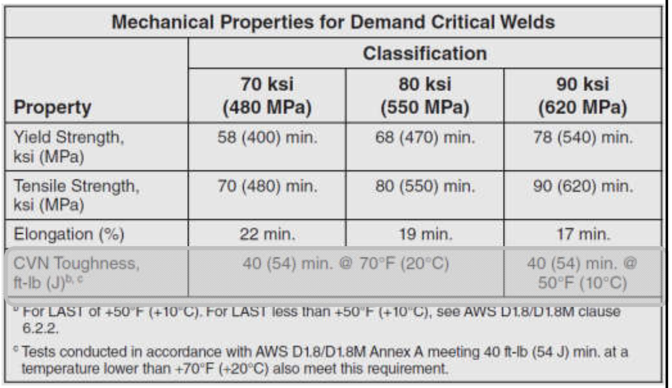

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These tests required for 40 @ 70 for special procedures are additional tests relative to "typical" Conformance Demonstration, and add cost to the filler material

Consumables and Demand Critical Welds (341-16, A3.4)

Filler Metal Classification Properties for Seismic Force-Resisting System Welds

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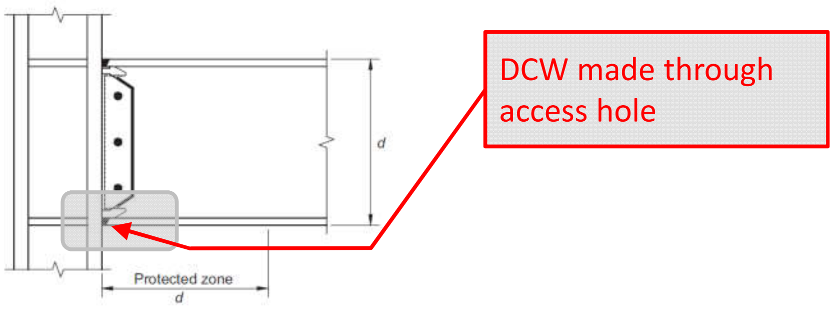
Consumables and Demand Critical Welds (341-16, A3.4)

Demand critical welds made through weld access holes (restricted access)



DCW made through access hole

For such welds, Supplemental Welder Qualification is required in accordance with AWS D1.8, C-Annex D.

Refer to AWS D1.8 C-D3 and C-D4 for requirements of test plate and specimen testing requirements, respectively

Consumables and Demand Critical Welds (341-16, A3.4)

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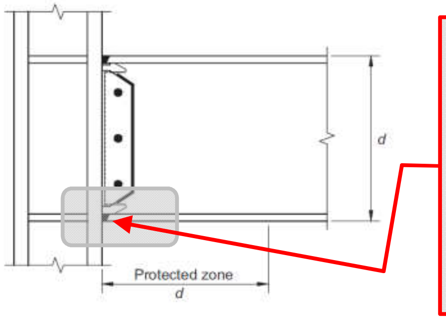

- Each pass has to be completed before adding another pass…
  - Half from one side of the web…
  - The other half from the other side of the web…
  - The welder has to demonstrate that he/she can properly tie those two half-passes together properly

For such welds, Supplemental Welded Qualification is required in accordance with AWS D1.8, C-Annex D.

Refer to AWS D1.8 C-D3 and C-D4 for requirements of test plate and specimen testing requirements, respectively

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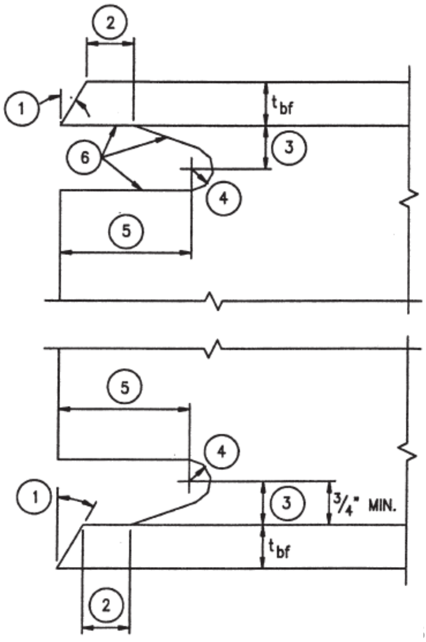
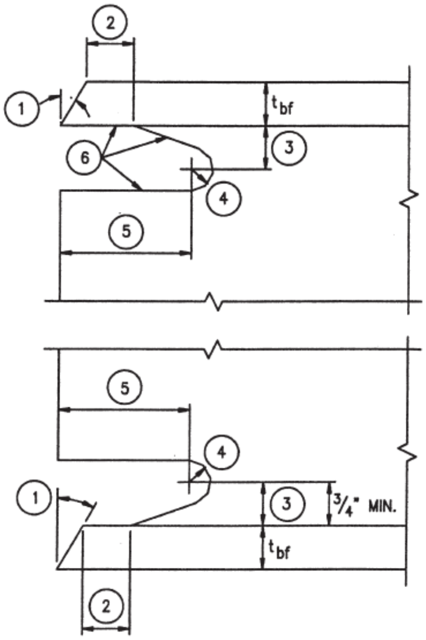
First, A Few General Issues

SPECIAL WELD ACCESS HOLES (AWS D1.8, 6.11.1.2)



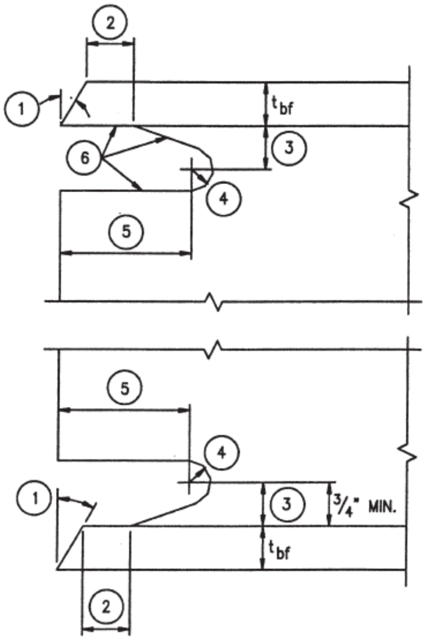
Special Seismic Weld Access Holes (AWS D1.8, 6.11.1.2)



Notes:
1. Bevel as required for selected groove weld.
2. Larger of t_{bf} or ½ in. (13 mm) (plus ½ t_{bf}, or minus ¼ t_{bf})
3. ¾ t_{bf} to t_{bf}, ¾ in. (19 mm) minimum (± ¼ in.) (± 6 mm)
4. ⅜ in. (10 mm) minimum radius (plus not limited, minus 0)
5. 3 t_{bf} (± ½ in.) (±13 mm)
6. See FEMA-353, "Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications," for fabrication details including cutting methods and smoothness requirements.

Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25°.

Special Seismic Weld Access Holes (AWS D1.8, 6.11.1.2)

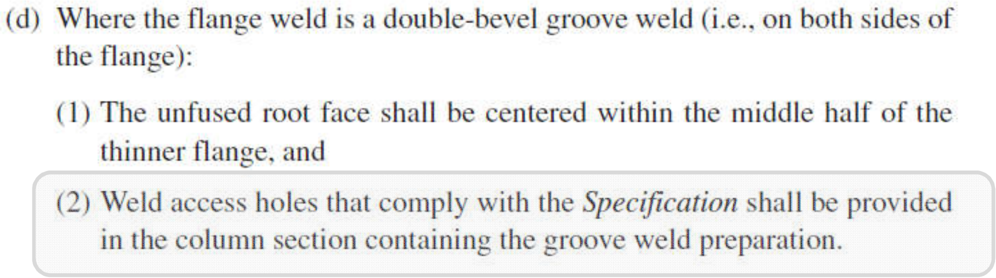


- FR moment connection in OMF IMF and SMF
  - Wide flange beam-to-flange of wide flange column
- Where required in AISC 358-16

Special Seismic Weld Access Holes (AWS D1.8, 6.11.1.2)

Welded Column Splices…
Not Required…

e.g., AISC 341-16 – E3.6g.2(d)(2)
-SMF-

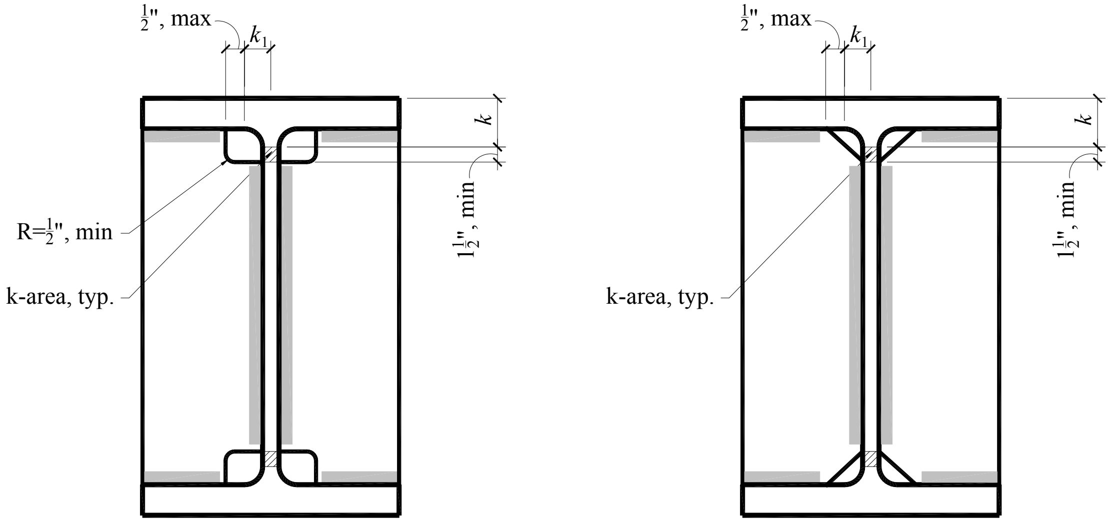


(d) Where the flange weld is a double-bevel groove weld (i.e., on both sides of the flange):

(1) The unfused root face shall be centered within the middle half of the thinner flange, and

(2) Weld access holes that comply with the *Specification* shall be provided in the column section containing the groove weld preparation.

Special Continuity Plate (C.P.) Corner Snips (AWS D1.8, 4.1)



Where required in AISC 358

C.P. Geometry and Welding (AISC 341 E3.6f.2, IMF & SMF)

(a) Continuity-Plate Width

The width of the continuity plate shall be determined as follows:

(1) For W-shape columns, continuity plates shall, at a minimum, extend from the column web to a point opposite the tips of the wider beam flanges.

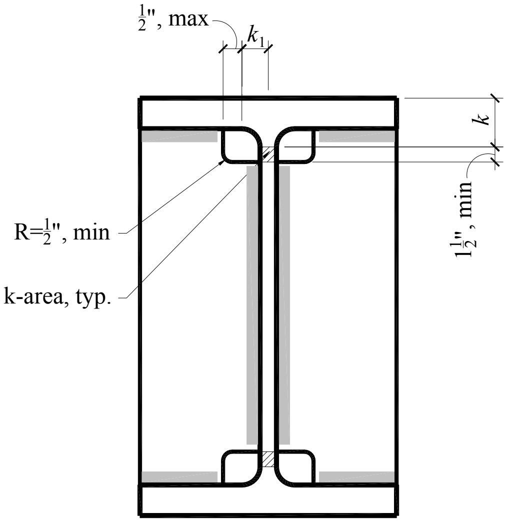
(2) For boxed wide-flange columns, continuity plates shall extend the full width from column web to side plate of the column.

(b) Continuity-Plate Thickness

The minimum thickness of the plates shall be determined as follows:

(1) For one-sided connections, the continuity plate thickness shall be at least 50% of the thickness of the beam flange.

(2) For two-sided connections, the continuity plate thickness shall be at least equal to 75% of the thickness of the thicker beam flange on either side of the column.



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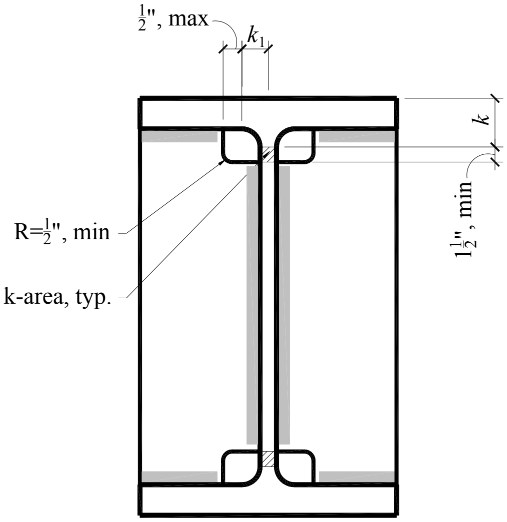


This is a relaxation from previous cycles.
The thinner the continuity plate, the smaller the welds and cost of welding!

C.P. Geometry and Welding (AISC 341 E3.6f.2, IMF & SMF)

(c) Continuity-Plate Welding

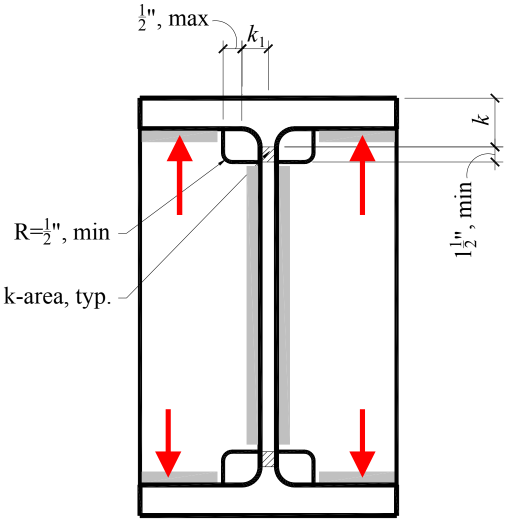
Continuity plates shall be welded to column flanges using CJP groove welds.

Continuity plates shall be welded to column webs or extended doubler plates using groove welds or fillet welds. The required strength of the welded joints of continuity plates to the column web or extended doubler plate shall be the lesser of the following:

(1) The sum of the available tensile strengths of the contact areas of the continuity plates to the column flanges that have attached beam flanges

(2) The available shear strength of the contact area of the plate with the column web or extended doubler plate

(3) The available shear strength of the column web, when the continuity plate is welded to the column web, or the available shear strength of the doubler plate, when the continuity plate is welded to an extended doubler plate



C.P. Geometry and Welding (AISC 341 E3.6f.2, IMF & SMF)

(c) Continuity-Plate Welding

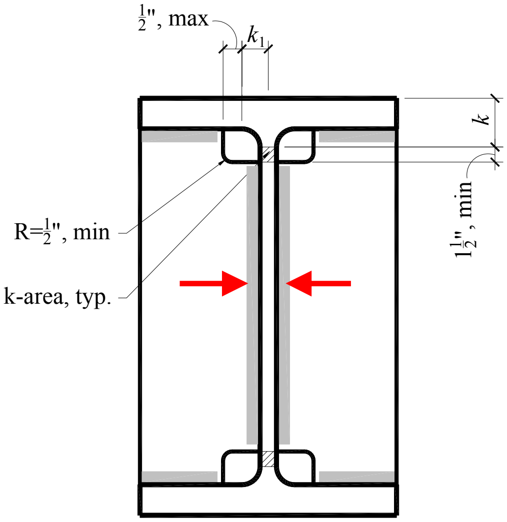
Continuity plates shall be welded to column flanges using CJP groove welds.

Continuity plates shall be welded to column webs or extended doubler plates using groove welds or fillet welds. The required strength of the welded joints of continuity plates to the column web or extended doubler plate shall be the lesser of the following:

(1) The sum of the available tensile strengths of the contact areas of the continuity plates to the column flanges that have attached beam flanges

(2) The available shear strength of the contact area of the plate with the column web or extended doubler plate

(3) The available shear strength of the column web, when the continuity plate is welded to the column web, or the available shear strength of the doubler plate, when the continuity plate is welded to an extended doubler plate



C.P. Geometry and Welding (AISC 341 E3.6f.2, IMF & SMF)

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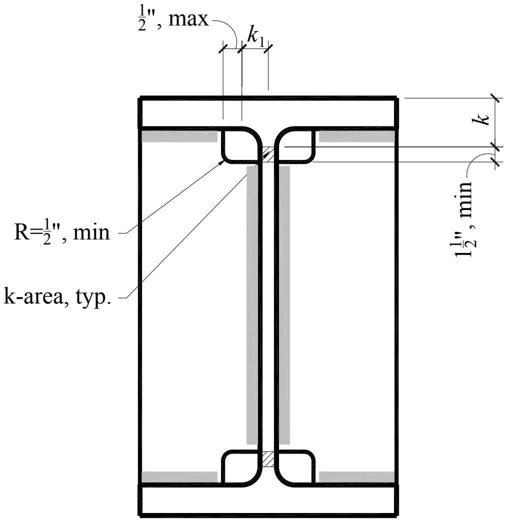
(1) The sum of the available tensile strengths of the contact areas of the continuity plates to the column flanges that have attached beam flanges

(2) The available shear strength of the contact area of the plate with the column web or extended doubler plate

(3) The available shear strength of the column web, when the continuity plate is welded to the column web, or the available shear strength of the doubler plate, when the continuity plate is welded to an extended doubler plate



More likely, all interfaces will need CJP welds; COSTLY.

Strive for clean columns!

C.P. Geometry and Welding (AISC 341 E3.6f.1(b), IMF & SMF)

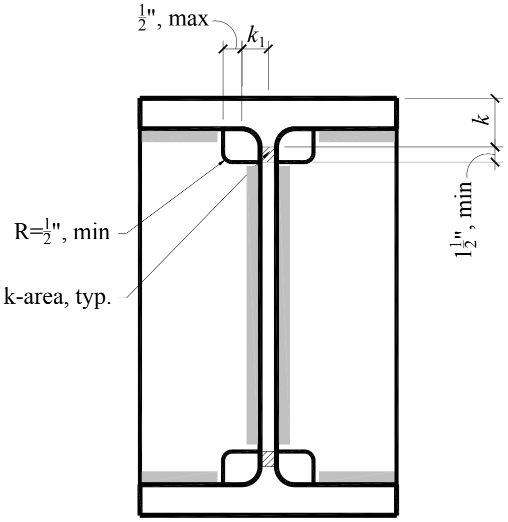
(b) Where the column flange thickness is less than the limiting thickness, t_{lim}, determined in accordance with this provision.

(1) Where the beam flange is welded to the flange of a W-shape or built-up I-shaped column, the limiting column-flange thickness is:

$$t_{\text{lim}} = \frac{b_{bf}}{6} \qquad \text{(E3-8)}$$

(2) Where the beam flange is welded to the flange of the I-shape in a boxed wide-flange column, the limiting column-flange thickness is:

$$t_{\text{lim}} = \frac{b_{bf}}{12} \qquad \text{(E3-9)}$$

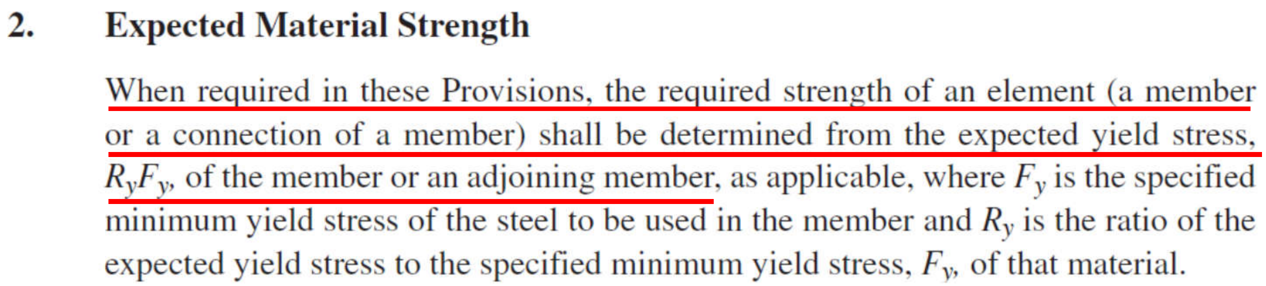


Additional checks; limit states required in AISC 360 must still be checked

First, A Few General Issues

MATERIAL & AVAILABLE EXPECTED STRENGTH (AISC 341-16 Clause A3.2)

Expected Strengths AISC 341-16 Clause A3.2



2. Expected Material Strength

When required in these Provisions, the required strength of an element (a member or a connection of a member) shall be determined from the expected yield stress, R_yF_y, of the member or an adjoining member, as applicable, where F_y is the specified minimum yield stress of the steel to be used in the member and R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y, of that material.

Expected Strengths AISC 341-16 Clause A3.2

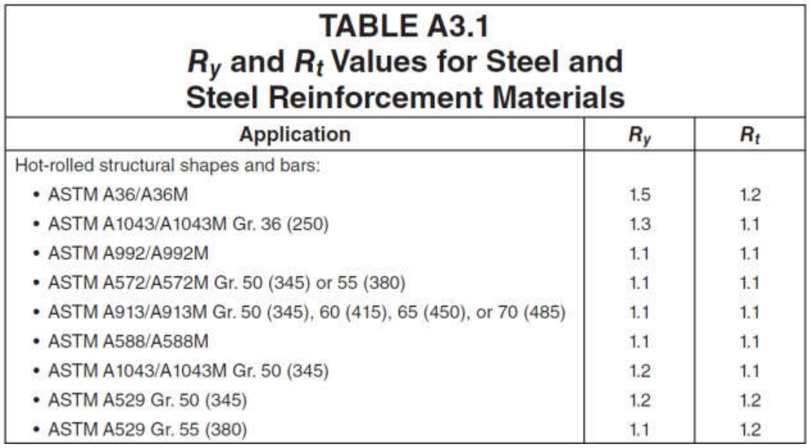
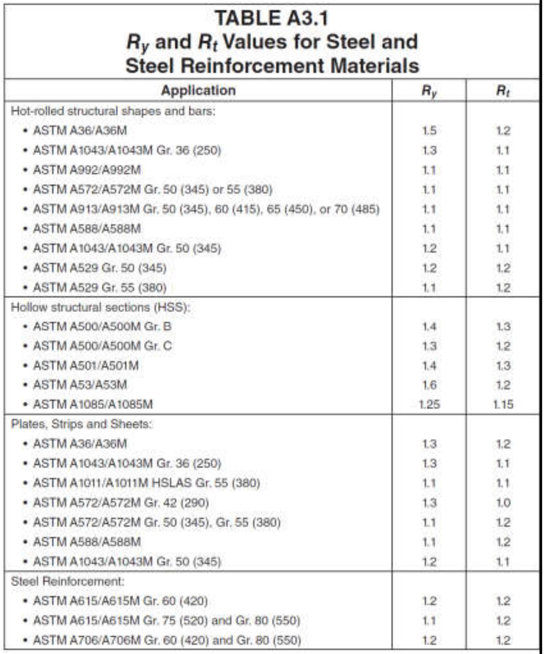


TABLE A3.1
R_y and R_t Values for Steel and Steel Reinforcement Materials

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



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• 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
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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

Expected Strengths AISC 341-16 Clause A3.2

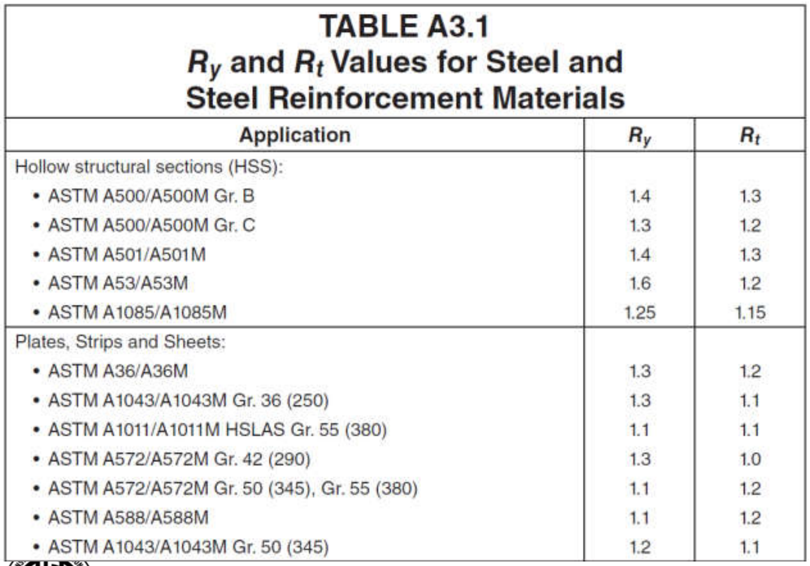

TABLE A3.1
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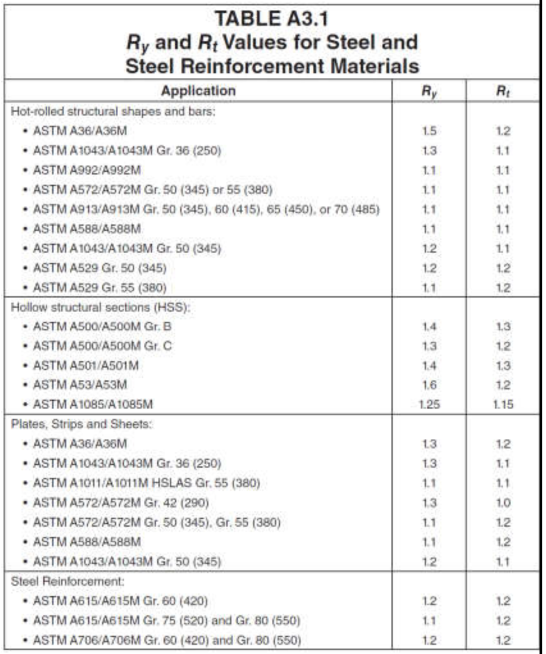
• ASTM A588/A588M	1.1	1.2
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1



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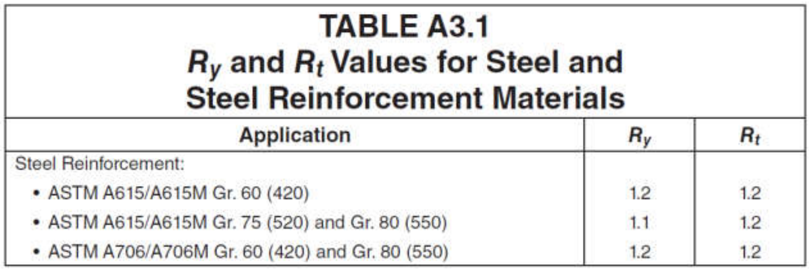
• ASTM A706/A706M Gr. 60 (420) and Gr. 80 (550)	1.2	1.2

Expected Strengths AISC 341-16 Clause A3.2



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Steel Reinforcement:		
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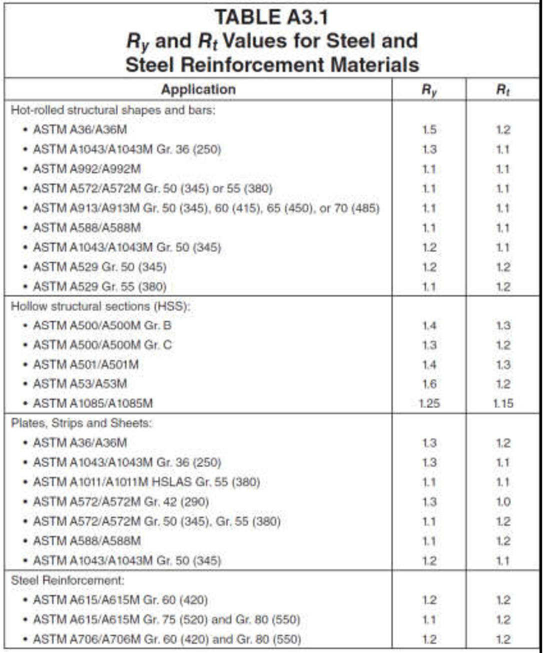
• ASTM A615/A615M Gr. 75 (520) and Gr. 80 (550)	1.1	1.2
• ASTM A706/A706M Gr. 60 (420) and Gr. 80 (550)	1.2	1.2



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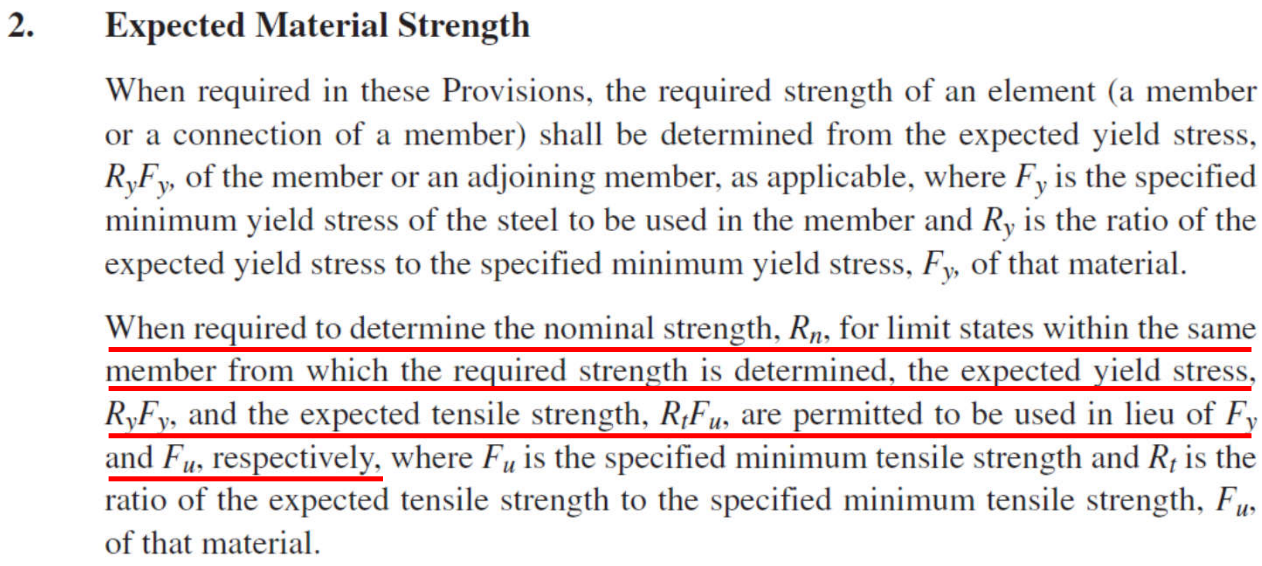
Expected Strengths AISC 341-16 Clause A3.2



2. Expected Material Strength

When required in these Provisions, the required strength of an element (a member or a connection of a member) shall be determined from the expected yield stress, R_yF_y, of the member or an adjoining member, as applicable, where F_y is the specified minimum yield stress of the steel to be used in the member and R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y, of that material.

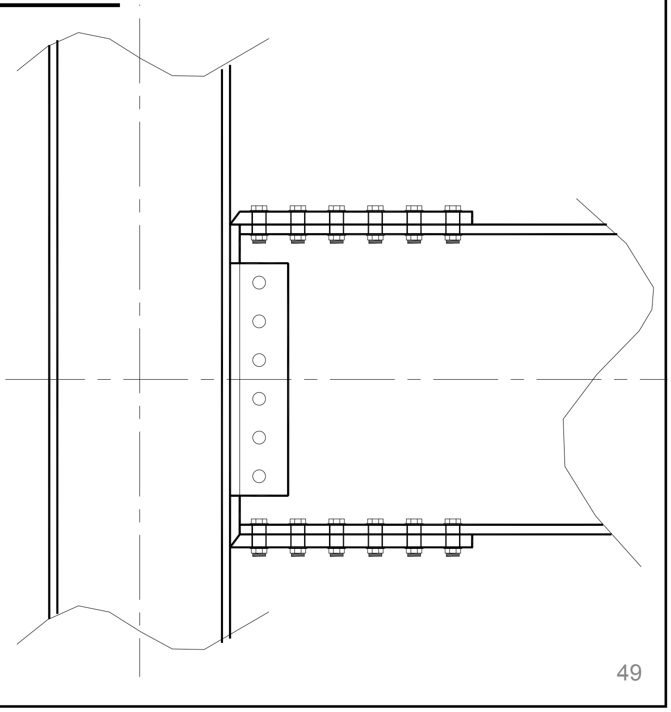
When required to determine the nominal strength, R_n, for limit states within the same member from which the required strength is determined, the expected yield stress, R_yF_y, and the expected tensile strength, R_tF_u, are permitted to be used in lieu of F_y and F_u, respectively, where F_u is the specified minimum tensile strength and R_t is the ratio of the expected tensile strength to the specified minimum tensile strength, F_u, of that material.

Expected Strengths AISC 341-16 Clause A3.2

Consider a BFP (bolted Flange Plate connection)

The required strength of the connection is based on the expected plastic flexural strength of the beam.

‘…the beam causes the load…’

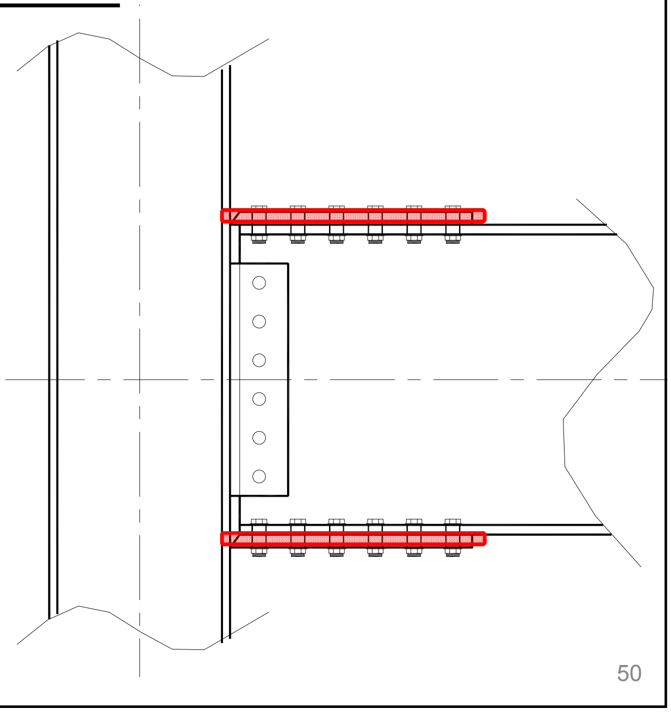


Expected Strengths AISC 341-16 Clause A3.2

‘…the beam causes the load…’

Limit state checks on the flange plate…

Use F_y in limit state checks for available strength



Expected Strengths AISC 341-16 Clause A3.2

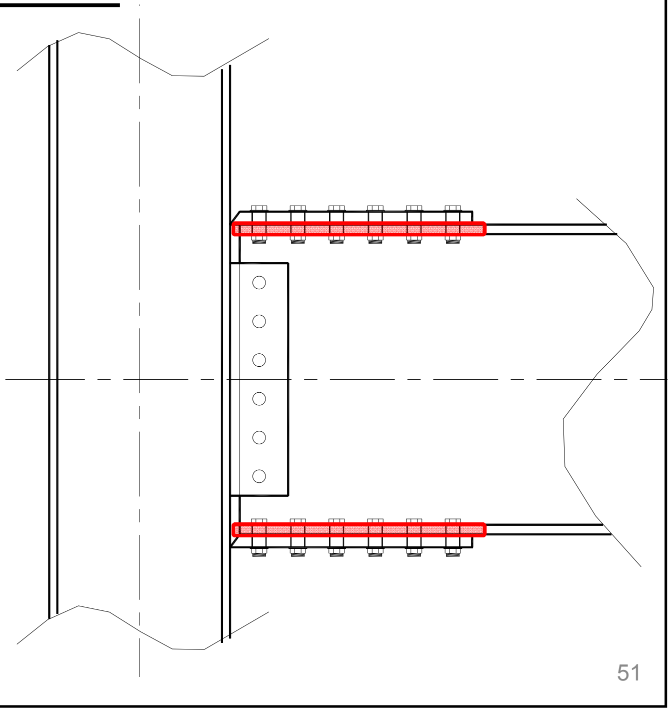
'…the beam causes the load…'

Limit state checks on the flange plate…

Use F_y in limit state checks for available strength

Limit state checks on the beam flange…

Use R_yF_y in limit state checks for available strength

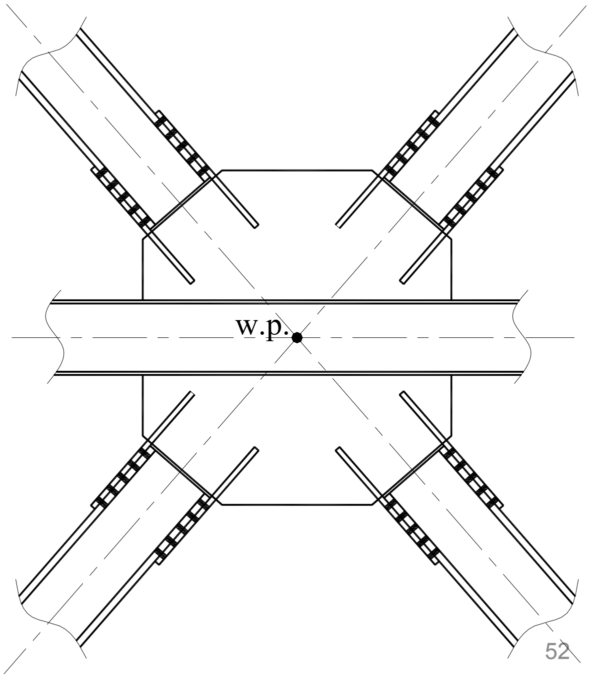


Expected Strengths AISC 341-16 Clause A3.2

Consider a vertical brace connection

The required strength of the connection is based on the expected plastic tensile/compression strength of the brace.

'…the brace causes the load…'

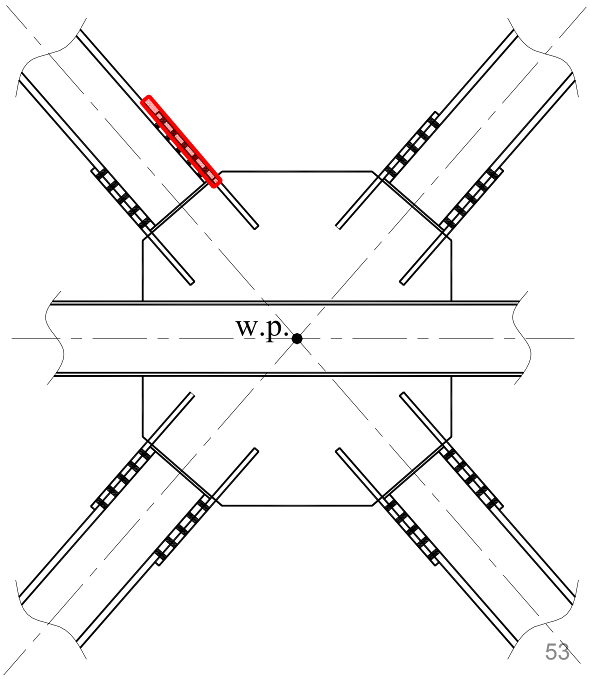


Expected Strengths AISC 341-16 Clause A3.2

'…the brace causes the load…'

Limit state checks on the flange plates…

Use F_y in limit state checks for available strength



Expected Strengths AISC 341-16 Clause A3.2

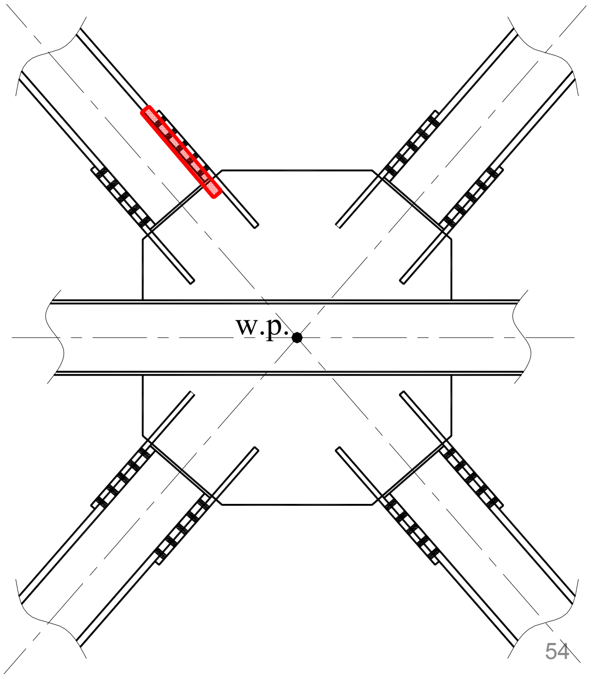
'…the brace causes the load…'

Limit state checks on the flange plates…

Use F_y in limit state checks for available strength

Limit state checks on the brace flanges…

Use R_yF_y in limit state checks for available strength



Expected Strengths AISC 341-16 Clause A3.2

'…the brace causes the load…'

Limit state checks on the flange plates…

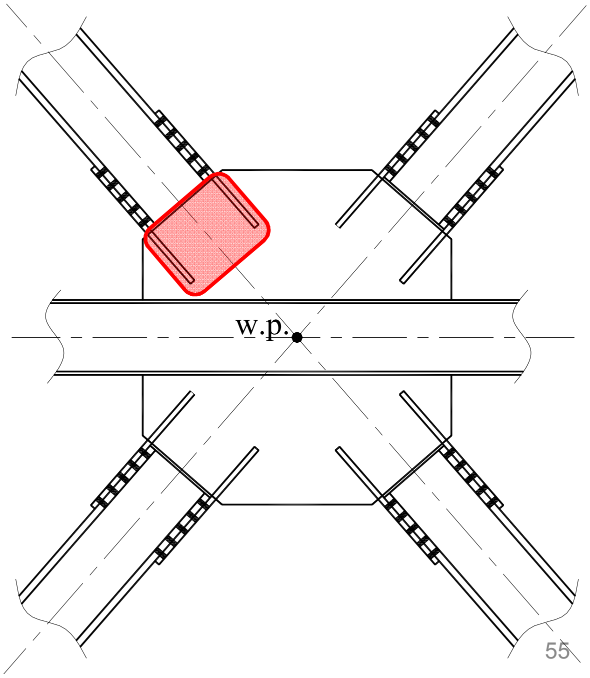
Use F_y in limit state checks for available strength

Limit state checks on the brace flanges…

Use R_yF_y in limit state checks for available strength

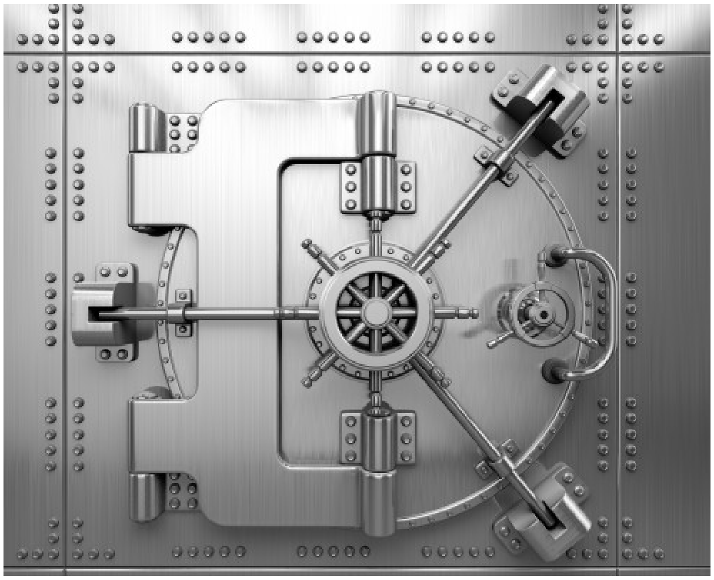
Limit state checks on the gussets…

Use F_y in limit state checks for available strength



First, A Few General Issues

PROTECTED ZONES (AISC 341, D1.3 & System Specific)



Protected Zones

Basically…

… a protected zone is a region of a structural component/element expected to experience significant inelastic deformation…

… any weldments or attachments that would detrimentally effect the behavior or performance of the element is prohibited…

Protected Zones

Basically…

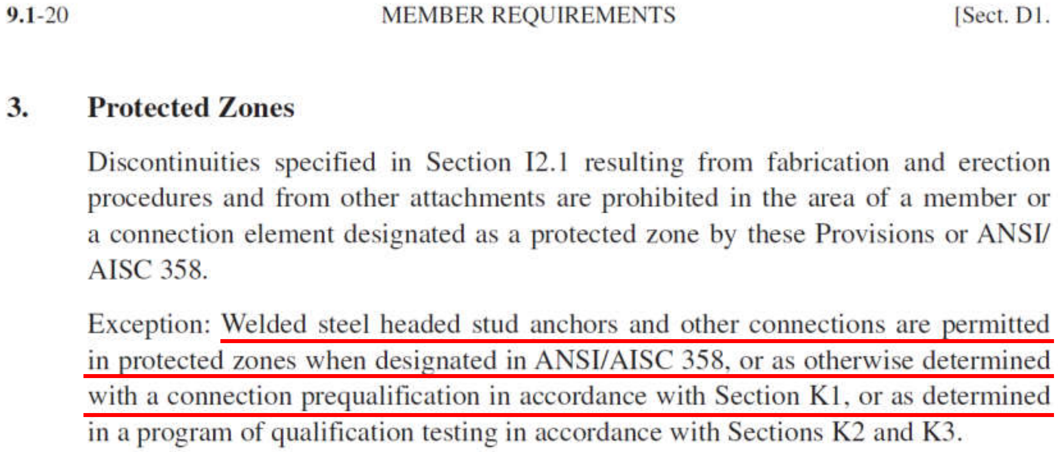
… a protected zone is a region of a structural component/element expected to experience significant inelastic deformation…

… any weldments or attachments that would detrimentally effect the behavior or performance of the element is prohibited…

… any weldments or attachments that are “approved work,” for example

- welding an HSS brace in the protected zone of a gusset plate in SCBF,
- Erection holes at brace ends in SCBFs,
- Connections of stability bracing at plastic hinge locations,
- etc.

Protected Zones



9.1-20 MEMBER REQUIREMENTS [Sect. D1.

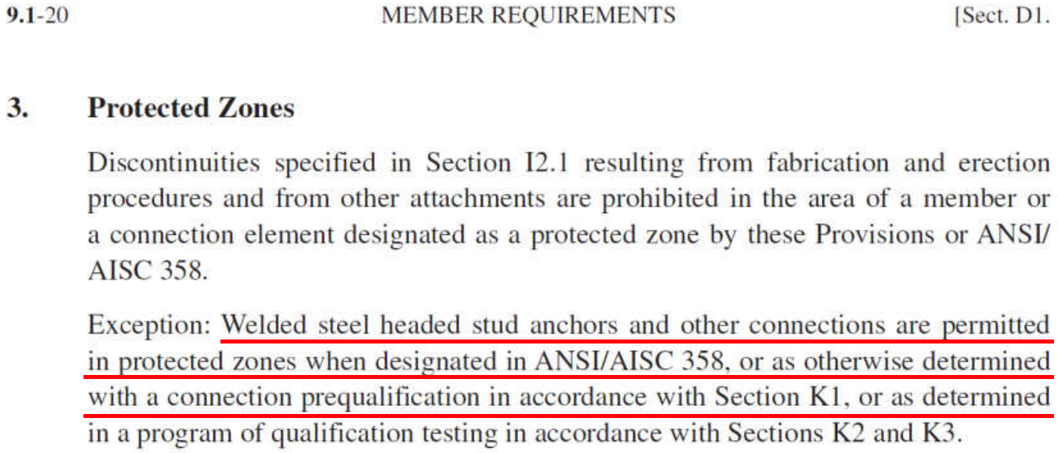
3. Protected Zones

Discontinuities specified in Section I2.1 resulting from fabrication and erection procedures and from other attachments are prohibited in the area of a member or a connection element designated as a protected zone by these Provisions or ANSI/AISC 358.

Exception: Welded steel headed stud anchors and other connections are permitted in protected zones when designated in ANSI/AISC 358, or as otherwise determined with a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Sections K2 and K3.

Very helpful. Be aware

Protected Zones



9.1-20 MEMBER REQUIREMENTS [Sect. D1.

3. Protected Zones

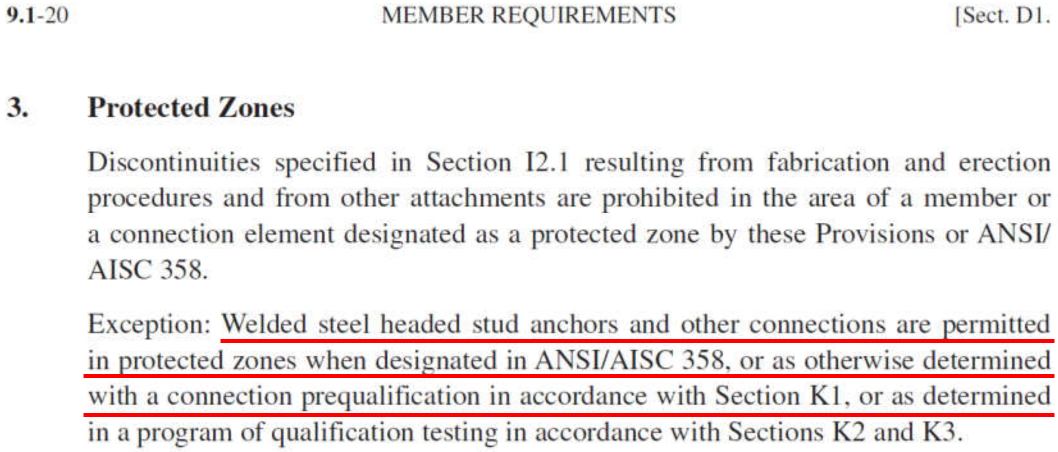
Discontinuities specified in Section I2.1 resulting from fabrication and erection procedures and from other attachments are prohibited in the area of a member or a connection element designated as a protected zone by these Provisions or ANSI/AISC 358.

Exception: Welded steel headed stud anchors and other connections are permitted in protected zones when designated in ANSI/AISC 358, or as otherwise determined with a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Sections K2 and K3.

Designations of protected zones are provided in AISC 341 for each system type…

… and in AISC 358 for connection type…

Protected Zones



9.1-20 MEMBER REQUIREMENTS [Sect. D1.

3. Protected Zones

Discontinuities specified in Section I2.1 resulting from fabrication and erection procedures and from other attachments are prohibited in the area of a member or a connection element designated as a protected zone by these Provisions or ANSI/AISC 358.

Exception: Welded steel headed stud anchors and other connections are permitted in protected zones when designated in ANSI/AISC 358, or as otherwise determined with a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Sections K2 and K3.

Protected zones must be identified on design, fabrication, and erection documents

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Protected Zones

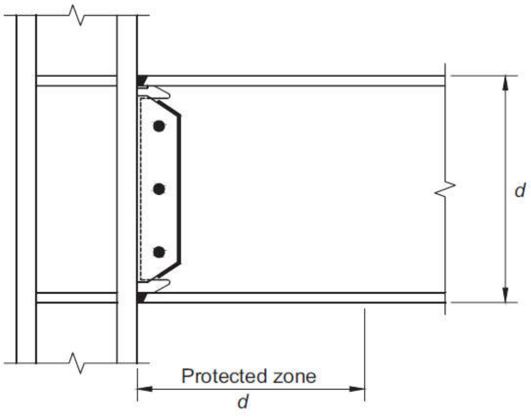


Fig. 8.1. WUF-W moment connection.

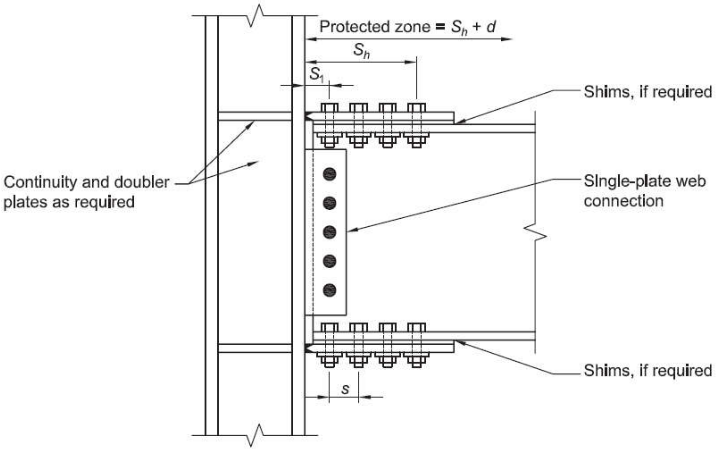


Fig. 7.1. Bolted flange plate moment connection.

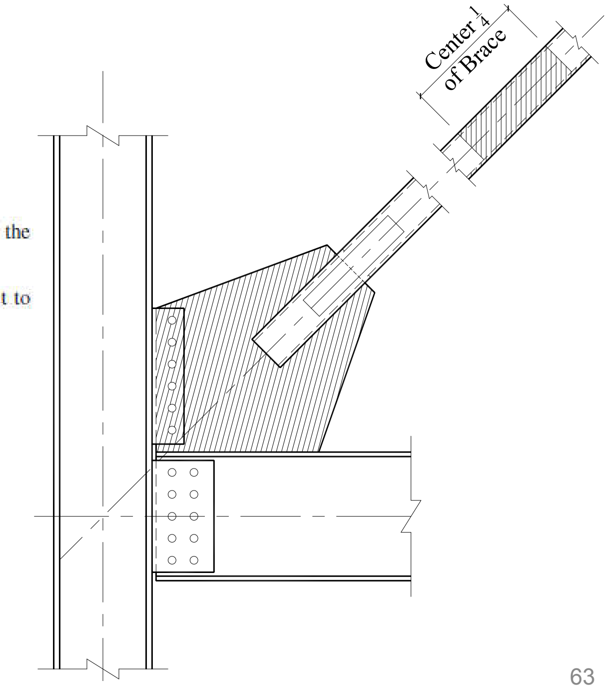
Protected Zones (AISC 341-16 F2.5c)

5c. Protected Zones

The protected zone of SCBF shall satisfy Section D1.3, and shall include the following:

(a) For braces, the center one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling

(b) Elements that connect braces to beams and columns



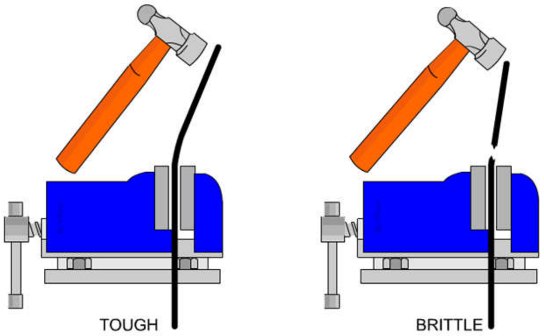
Protected Zones

Note that if an error does occur, AWS D1.8, Section 6.18, provides some guidance on…

- Erection aids,
- Removal of welds,
- Correction of errors,
- Repair of gouges and notches,
- Repair of mislocated holes,
- Repair of mislocated stud welds, and
- Repair of mislocated screws and shot pins

First, A Few General Issues

HEAVY SECTIONS (AISC 341, A3.3)



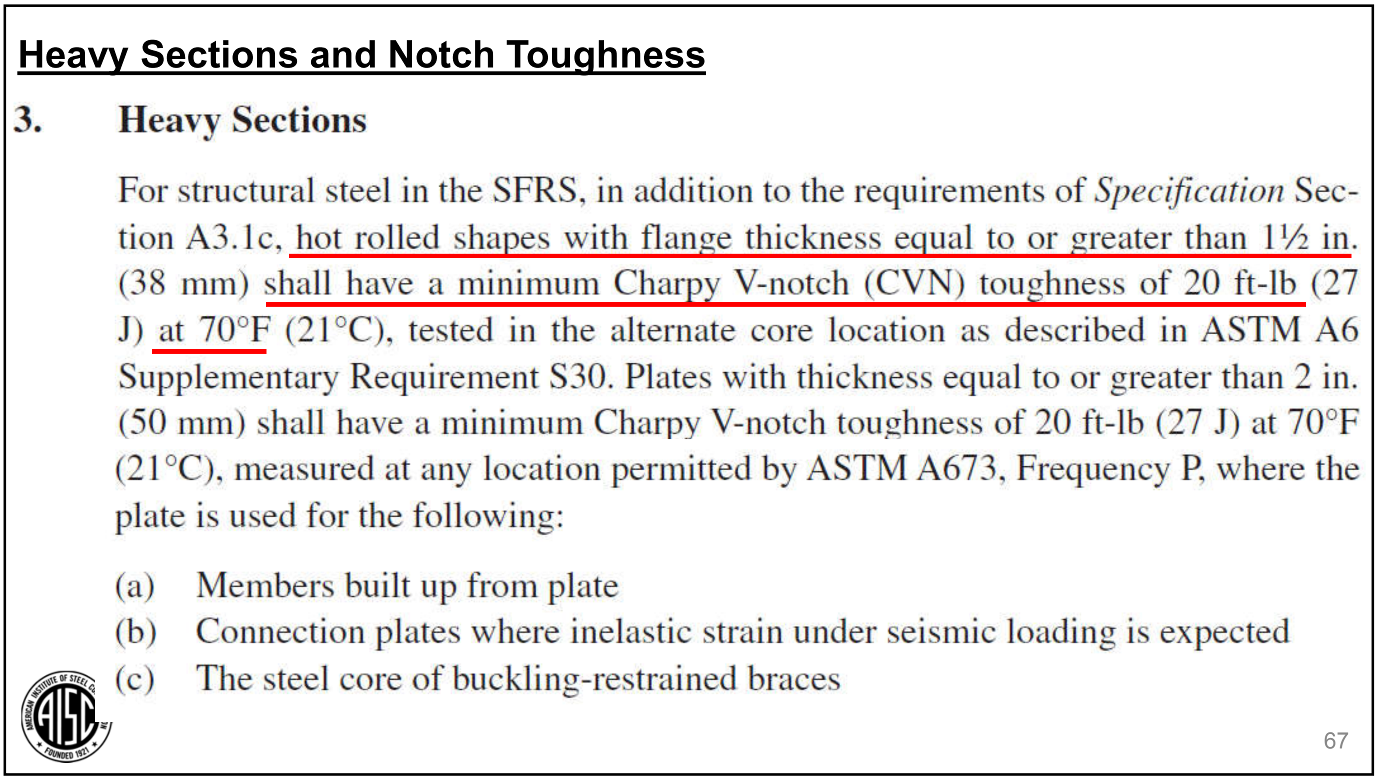
First, A Few General Issues

HEAVY SECTIONS (AISC 341, A3.3)

Notch Toughness

Basically is a measure of a material, such as a column flange or connection plate, to have sufficient through thickness strength…

…sufficient ductility of “thicker” elements subjected to seismic loading (inelastic demands)

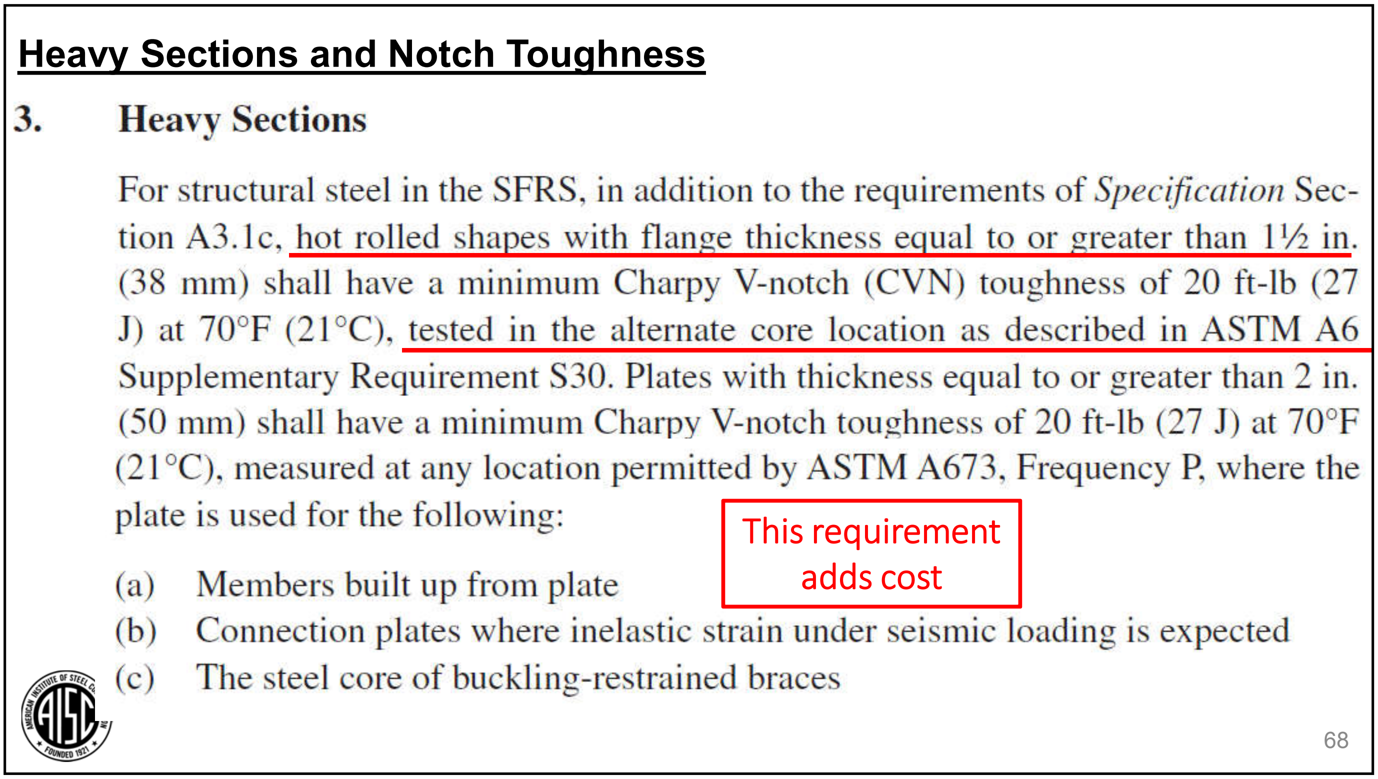


Heavy Sections and Notch Toughness

3. Heavy Sections

For structural steel in the SFRS, in addition to the requirements of *Specification* Section A3.1c, hot rolled shapes with flange thickness equal to or greater than 1½ in. (38 mm) shall have a minimum Charpy V-notch (CVN) toughness of 20 ft-lb (27 J) at 70°F (21°C), tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates with thickness equal to or greater than 2 in. (50 mm) shall have a minimum Charpy V-notch toughness of 20 ft-lb (27 J) at 70°F (21°C), measured at any location permitted by ASTM A673, Frequency P, where the plate is used for the following:

(a) Members built up from plate
(b) Connection plates where inelastic strain under seismic loading is expected
(c) The steel core of buckling-restrained braces



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This requirement adds cost

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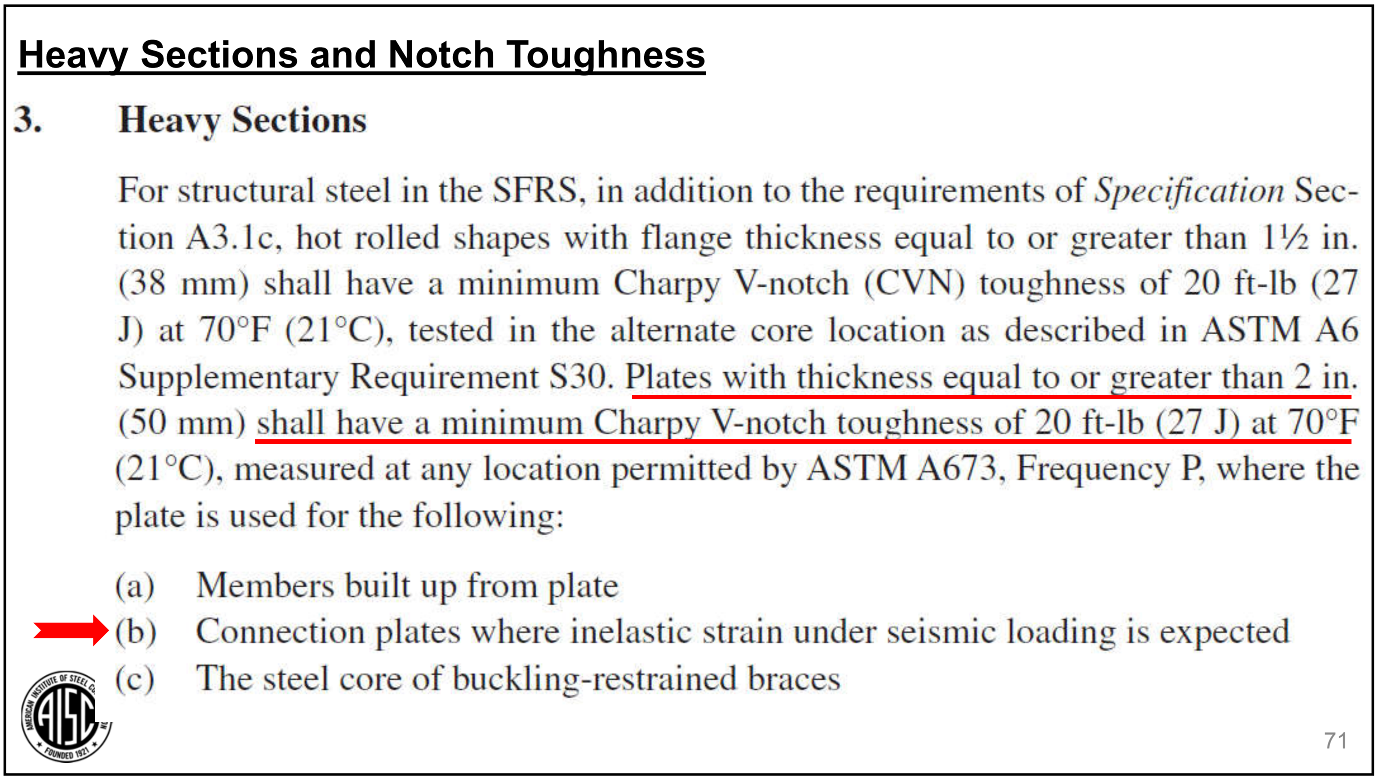

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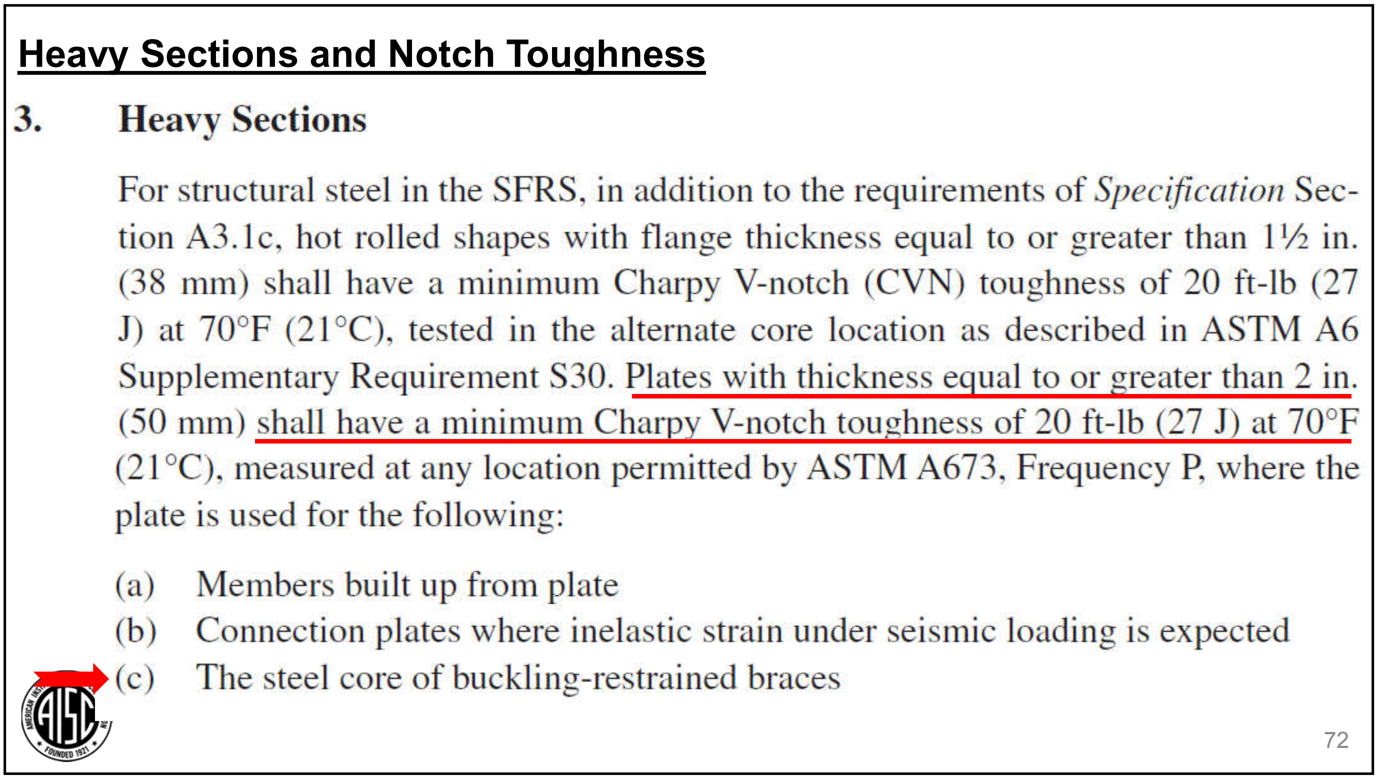

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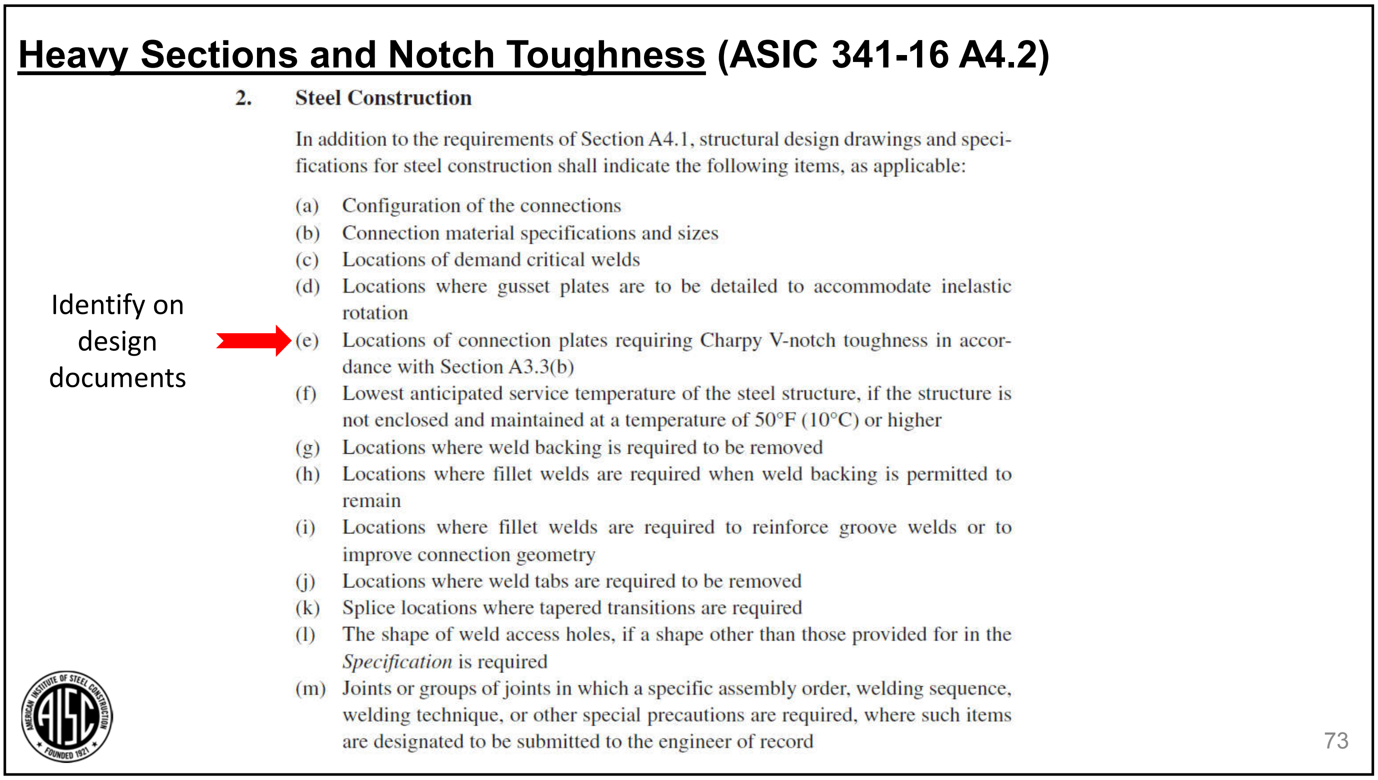

Heavy Sections and Notch Toughness (ASIC 341-16 A4.2)

2. Steel Construction

In addition to the requirements of Section A4.1, structural design drawings and specifications for steel construction shall indicate the following items, as applicable:

(a) Configuration of the connections
(b) Connection material specifications and sizes
(c) Locations of demand critical welds
(d) Locations where gusset plates are to be detailed to accommodate inelastic rotation
(e) Locations of connection plates requiring Charpy V-notch toughness in accordance with Section A3.3(b)
(f) Lowest anticipated service temperature of the steel structure, if the structure is not enclosed and maintained at a temperature of 50°F (10°C) or higher
(g) Locations where weld backing is required to be removed
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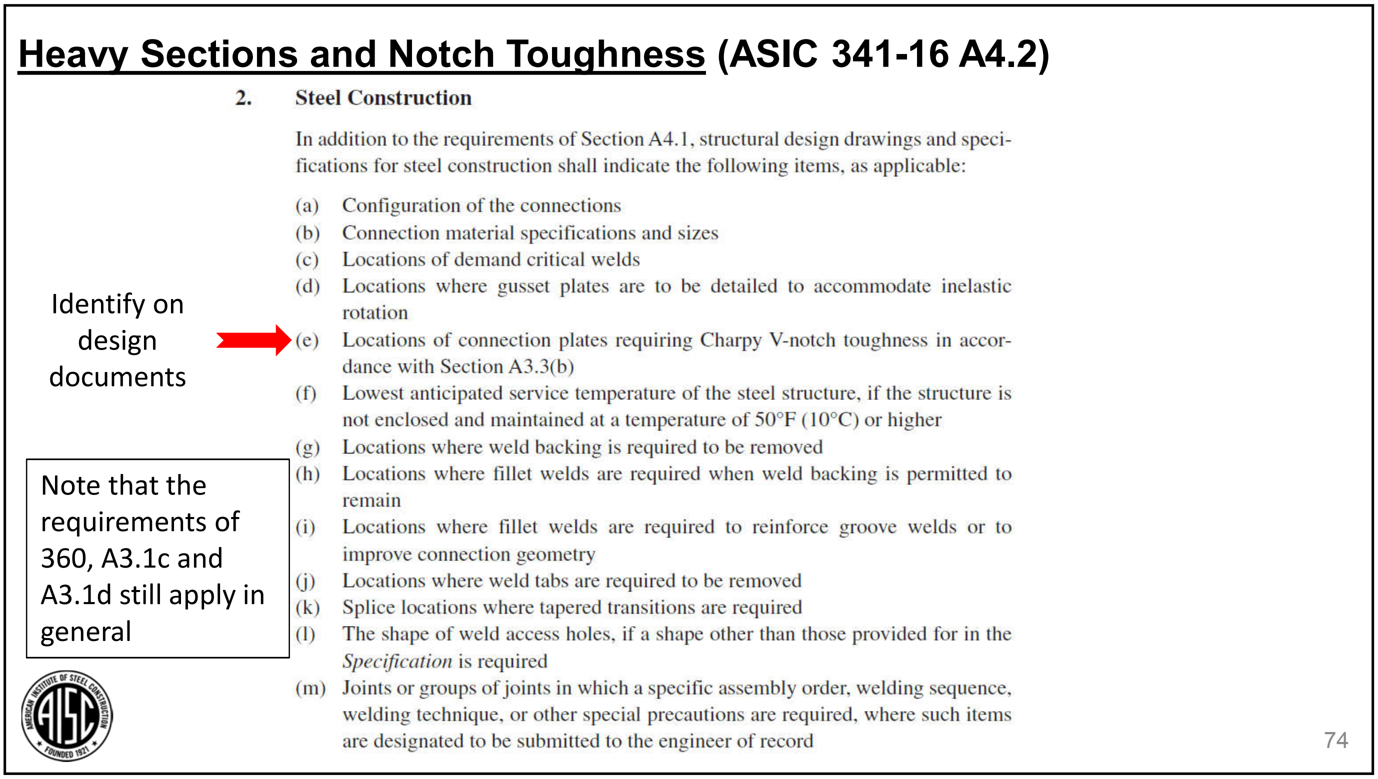
(k) Splice locations where tapered transitions are required
(l) The shape of weld access holes, if a shape other than those provided for in the *Specification* is required
(m) Joints or groups of joints in which a specific assembly order, welding sequence, welding technique, or other special precautions are required, where such items are designated to be submitted to the engineer of record

Identify on design documents



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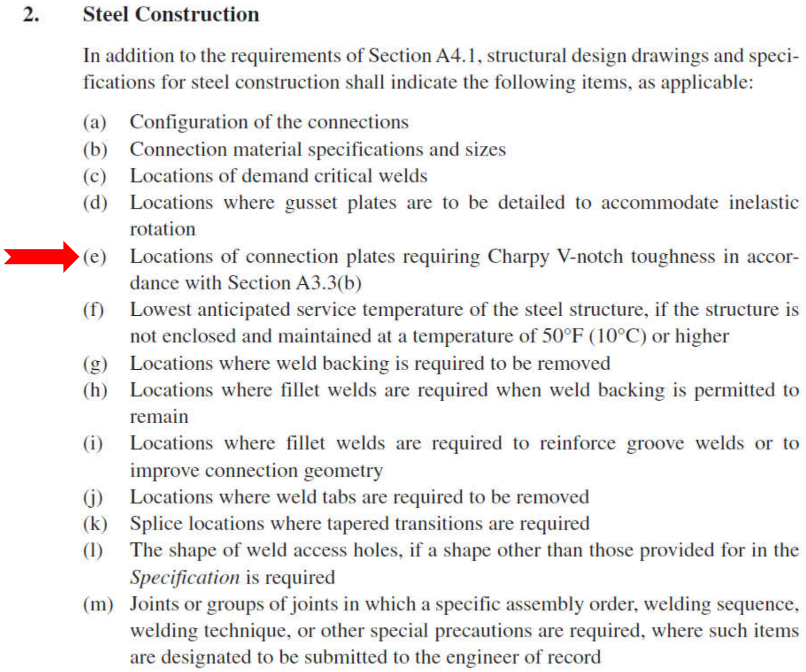
Note that the requirements of 360, A3.1c and A3.1d still apply in general

Heavy Sections and Notch Toughness (ASIC 341-16 A4.2)



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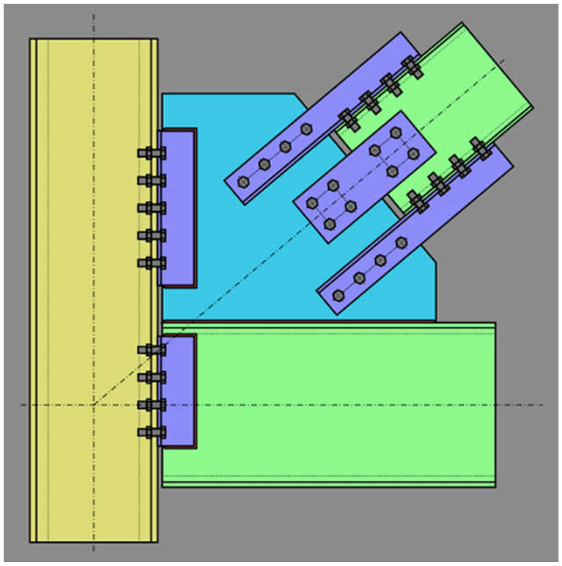
Identify on design documents

Fabricators may have to order these sections in lengths to ensure enough fall-off material to use for the tests if purchased through a service center

First, A Few General Issues

Bolted Joints (AISC 341, D2.2)



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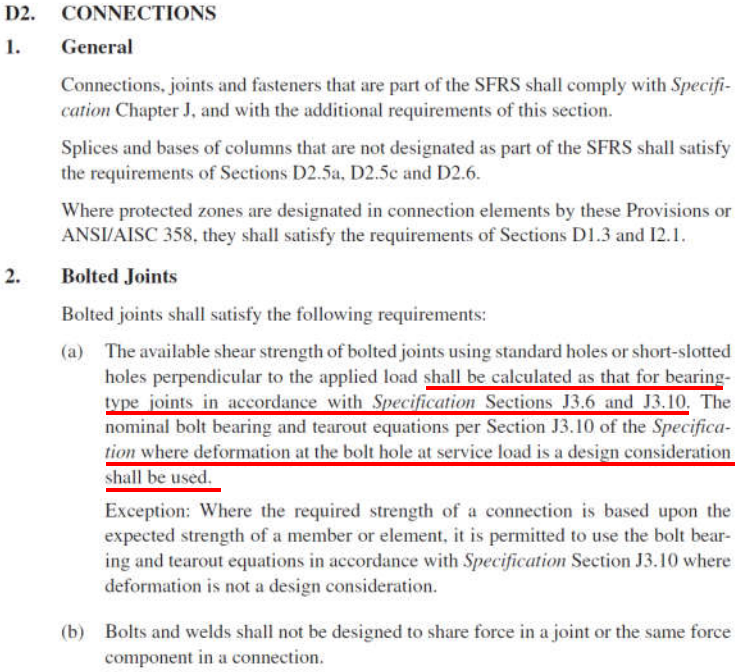


D2. CONNECTIONS

1. General

Connections, joints and fasteners that are part of the SFRS shall comply with *Specification* Chapter J, and with the additional requirements of this section.

Splices and bases of columns that are not designated as part of the SFRS shall satisfy the requirements of Sections D2.5a, D2.5c and D2.6.

Where protected zones are designated in connection elements by these Provisions or ANSI/AISC 358, they shall satisfy the requirements of Sections D1.3 and I2.1.

2. Bolted Joints

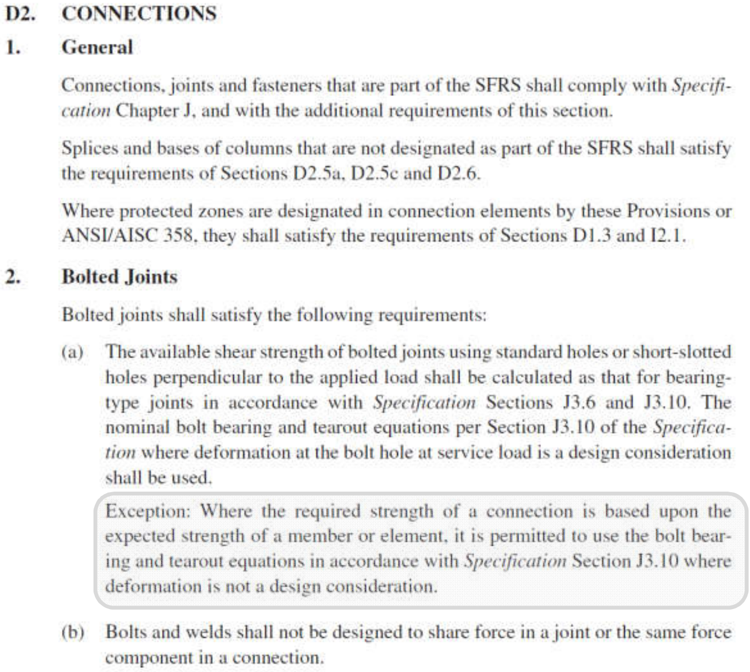
Bolted joints shall satisfy the following requirements:

(a) The available shear strength of bolted joints using standard holes or short-slotted holes perpendicular to the applied load shall be calculated as that for bearing-type joints in accordance with *Specification* Sections J3.6 and J3.10. The nominal bolt bearing and tearout equations per Section J3.10 of the *Specification* where deformation at the bolt hole at service load is a design consideration shall be used.

Exception: Where the required strength of a connection is based upon the expected strength of a member or element, it is permitted to use the bolt bearing and tearout equations in accordance with *Specification* Section J3.10 where deformation is not a design consideration.

(b) Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

Bolted Joints (AISC 341, D2.2)



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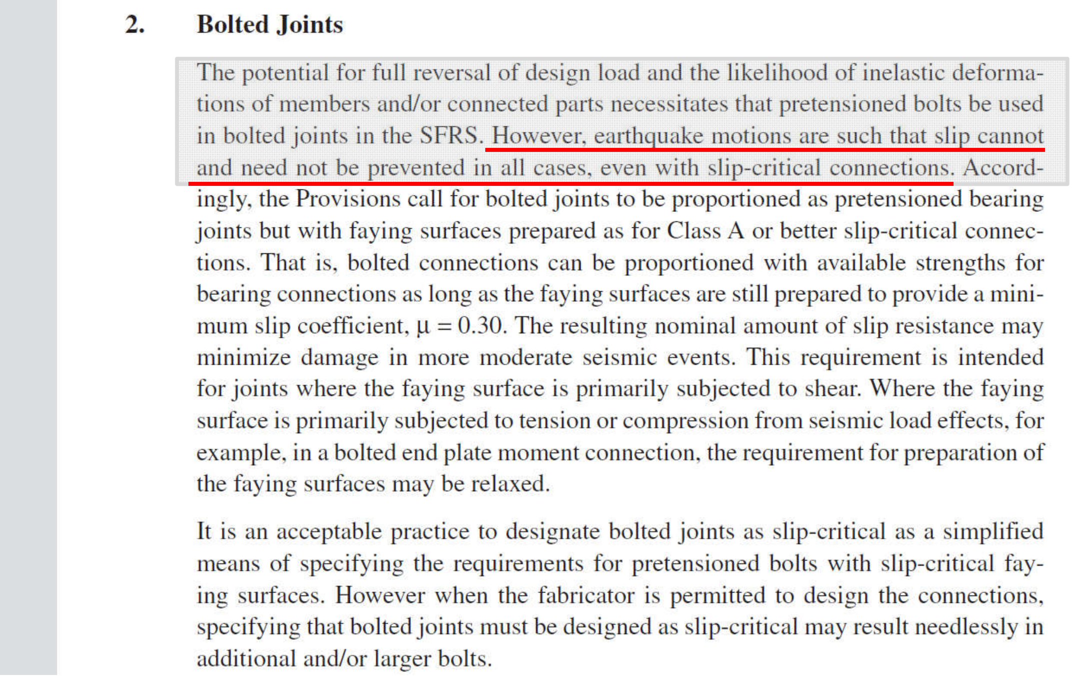

2. Bolted Joints

The potential for full reversal of design load and the likelihood of inelastic deformations of members and/or connected parts necessitates that pretensioned bolts be used in bolted joints in the SFRS. However, earthquake motions are such that slip cannot and need not be prevented in all cases, even with slip-critical connections. Accordingly, the Provisions call for bolted joints to be proportioned as pretensioned bearing joints but with faying surfaces prepared as for Class A or better slip-critical connections. That is, bolted connections can be proportioned with available strengths for bearing connections as long as the faying surfaces are still prepared to provide a minimum slip coefficient, μ = 0.30. The resulting nominal amount of slip resistance may minimize damage in more moderate seismic events. This requirement is intended for joints where the faying surface is primarily subjected to shear. Where the faying surface is primarily subjected to tension or compression from seismic load effects, for example, in a bolted end plate moment connection, the requirement for preparation of the faying surfaces may be relaxed.

It is an acceptable practice to designate bolted joints as slip-critical as a simplified means of specifying the requirements for pretensioned bolts with slip-critical faying surfaces. However when the fabricator is permitted to design the connections, specifying that bolted joints must be designed as slip-critical may result needlessly in additional and/or larger bolts.

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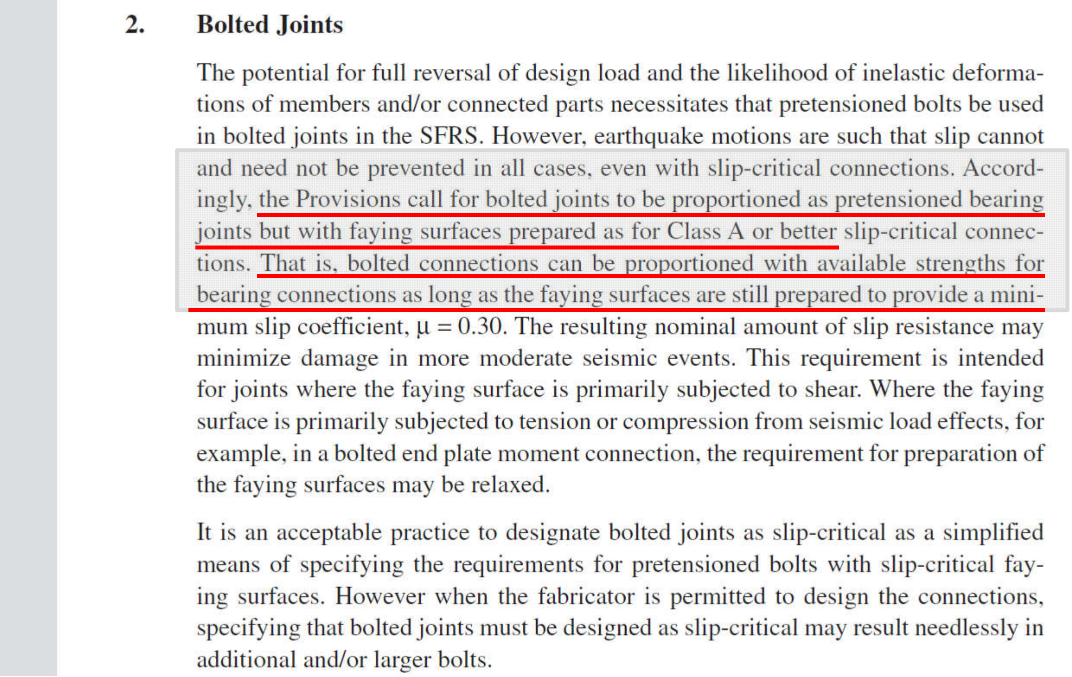

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Poll Question 1

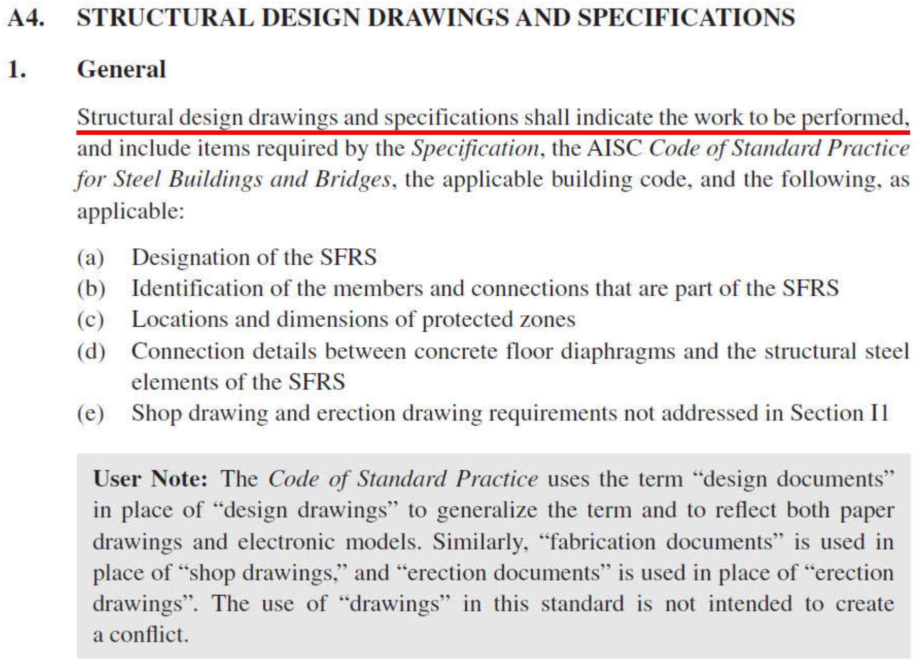
Q: Supplemental welder certification in conformance with AWS D1.8 Annex D is required…

a) For all welds connecting members part of the seismic lateral force resisting system,
b) When placing demand critical welds with restricted access,
c) When placing welds required to meet demand critical criteria, or
d) For all CJP welds in beam-to-column joints in OMF, IMF, & SMF systems

Select your answer!

Effective Communication and Structural Drawings

Effective Communication and Structural Drawings (AISC 341-16)



A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

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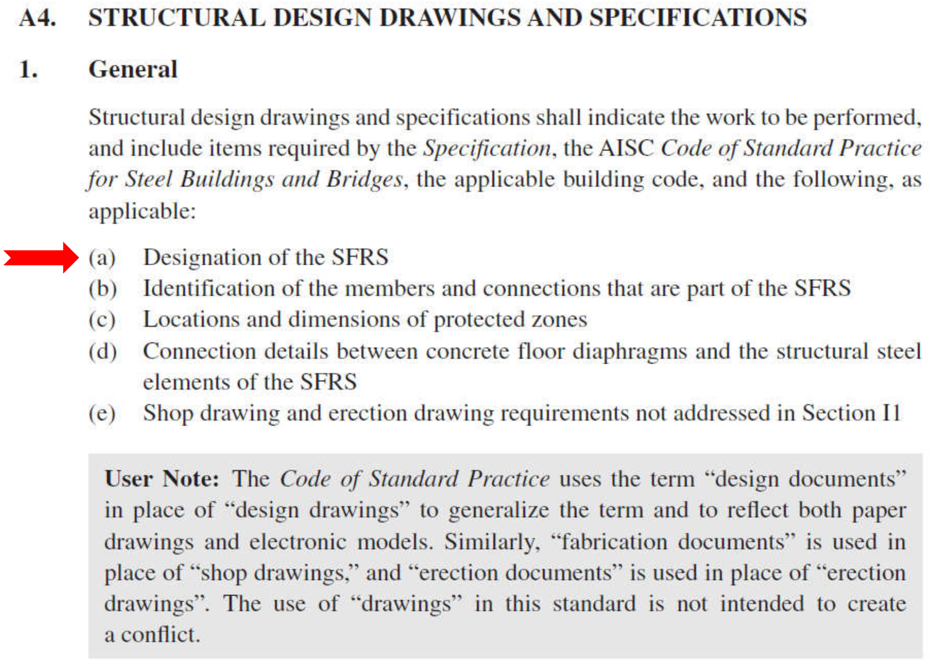
Effective Communication and Structural Drawings (AISC 341-16)



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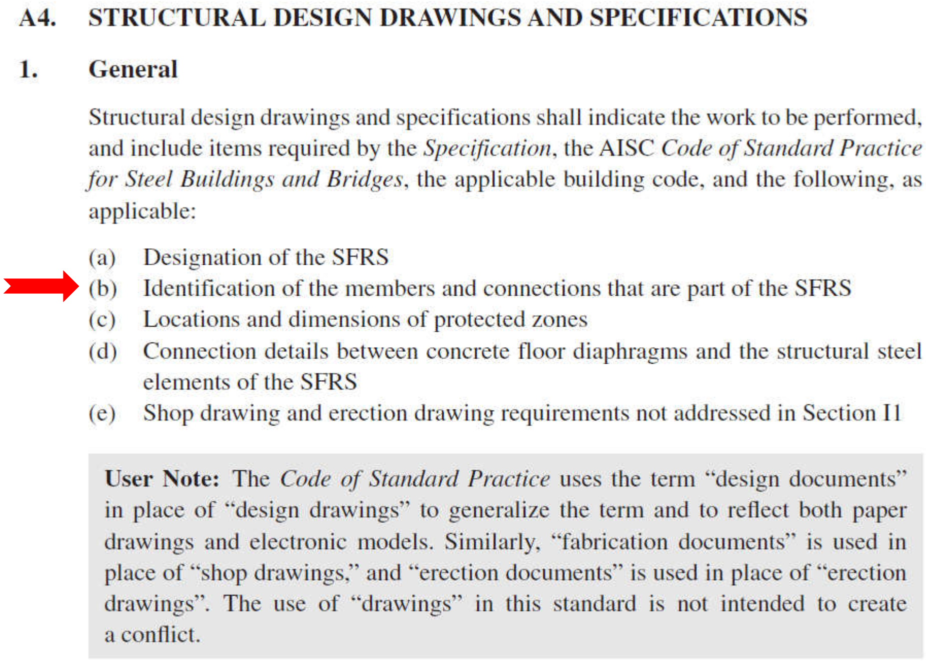

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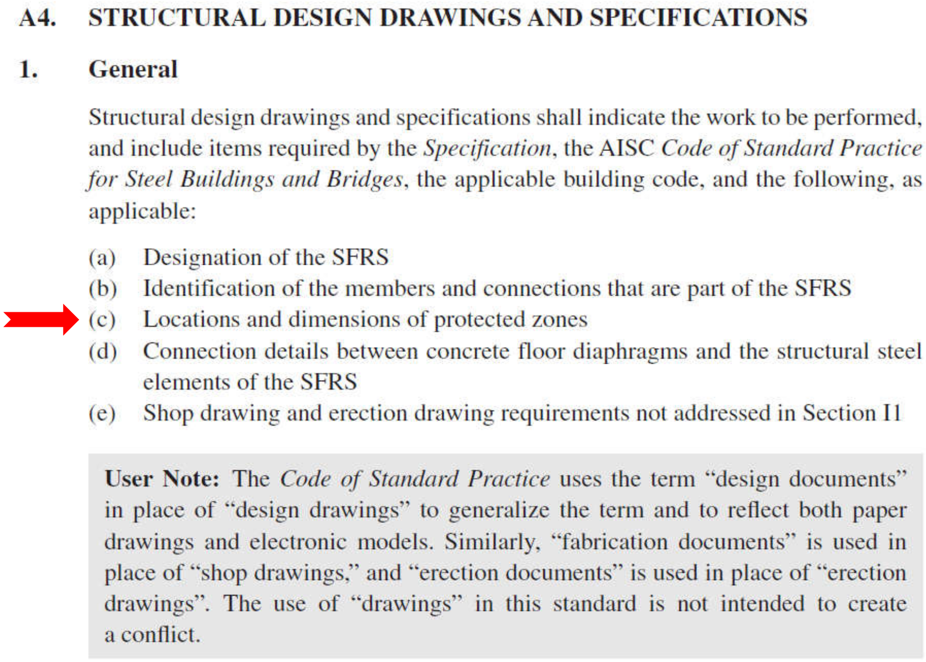

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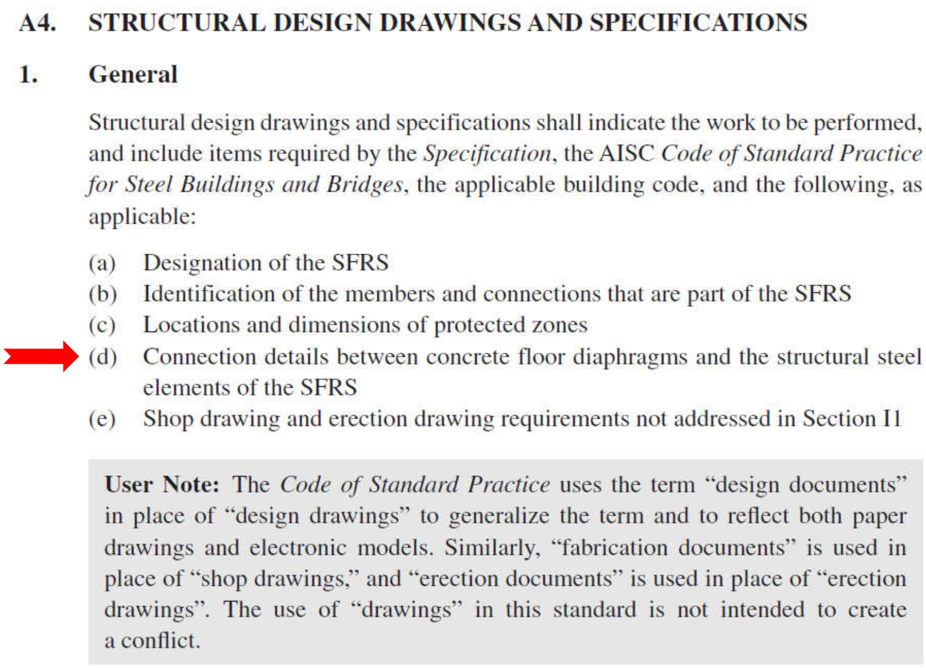

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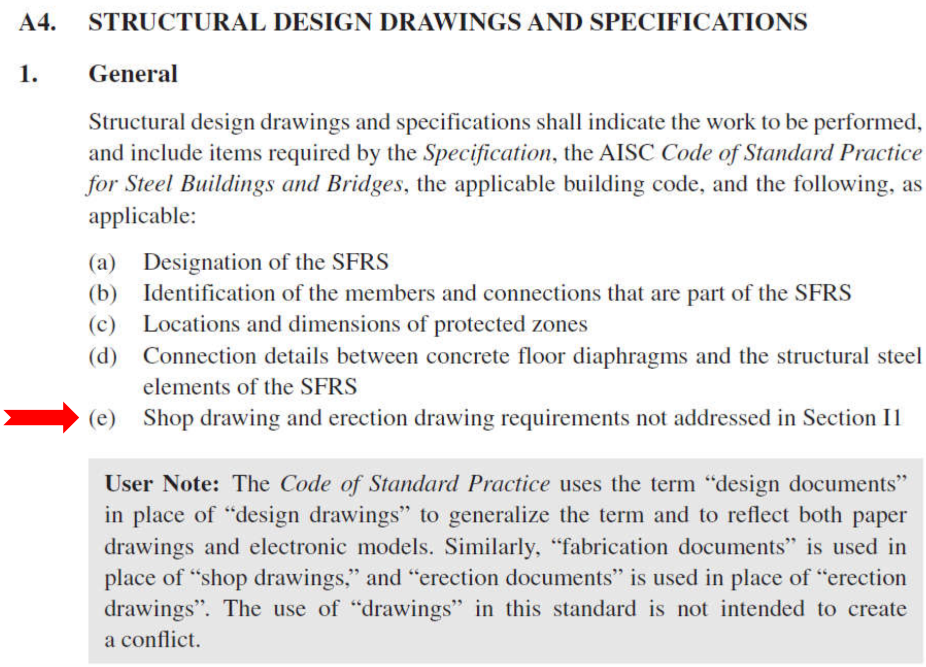

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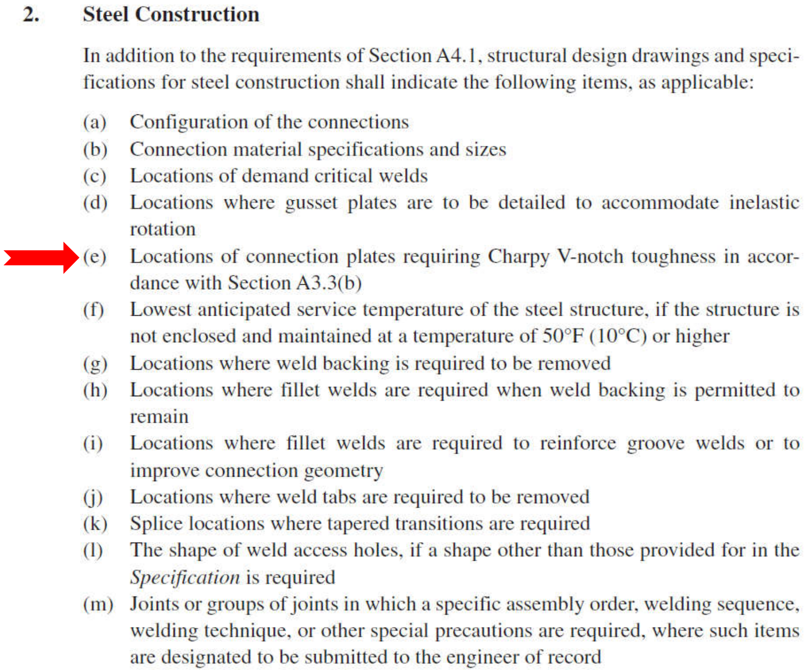
Effective Communication and Structural Drawings (ASIC 341-16 A4)



2. Steel Construction

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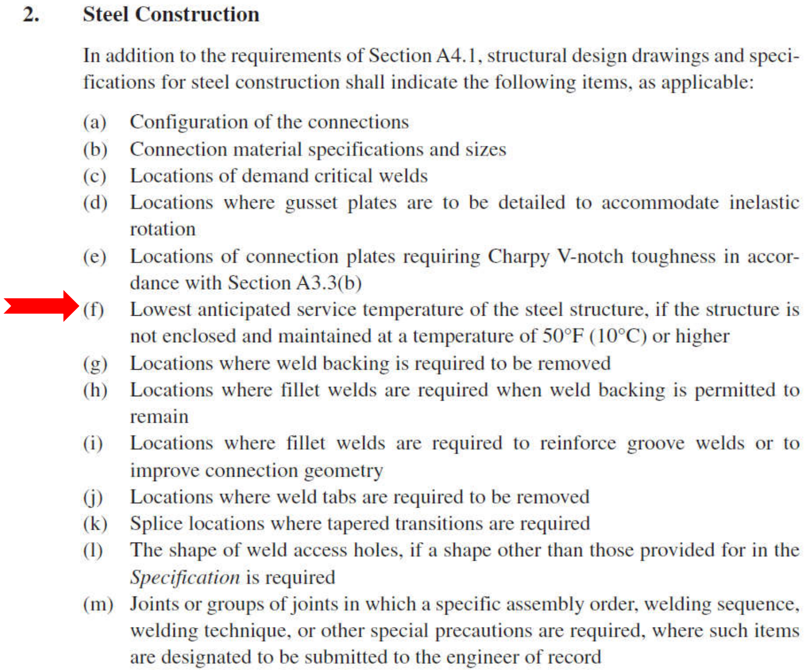

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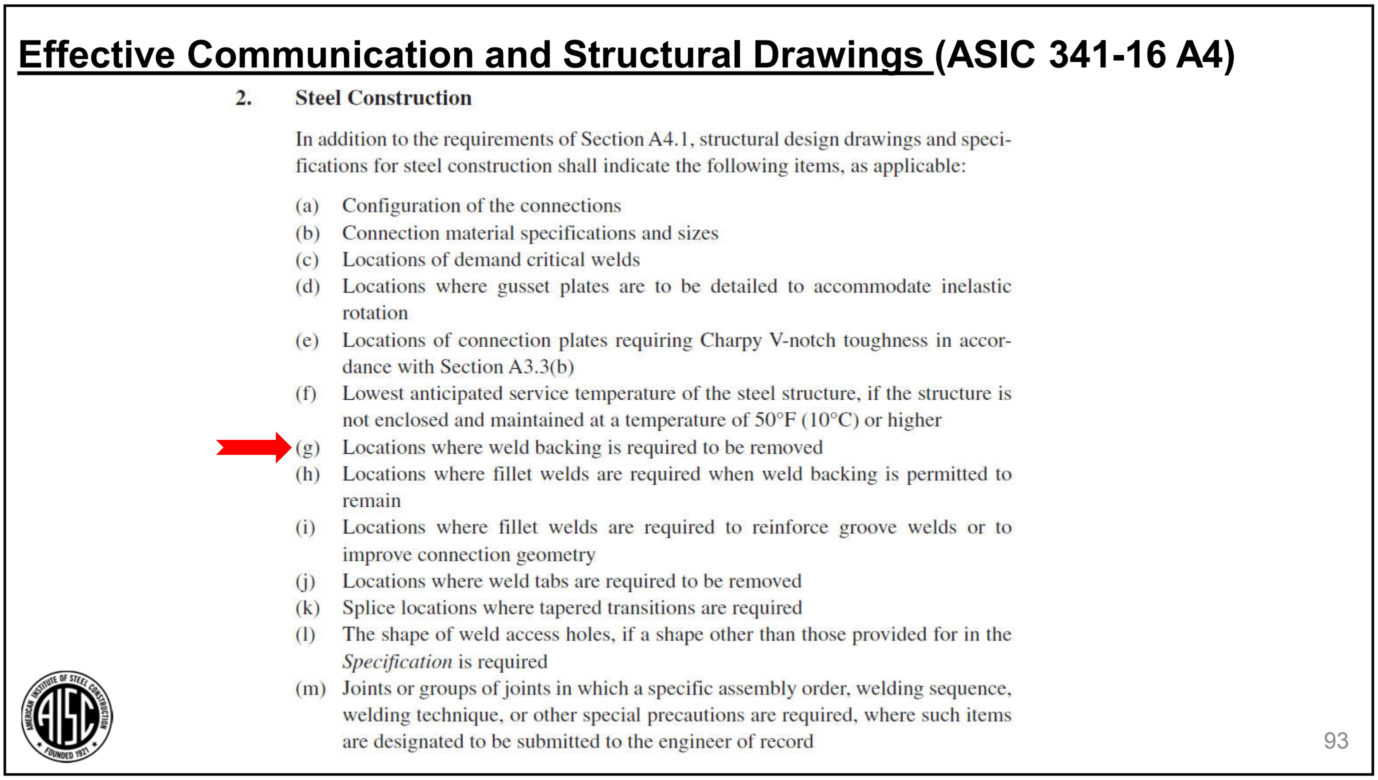

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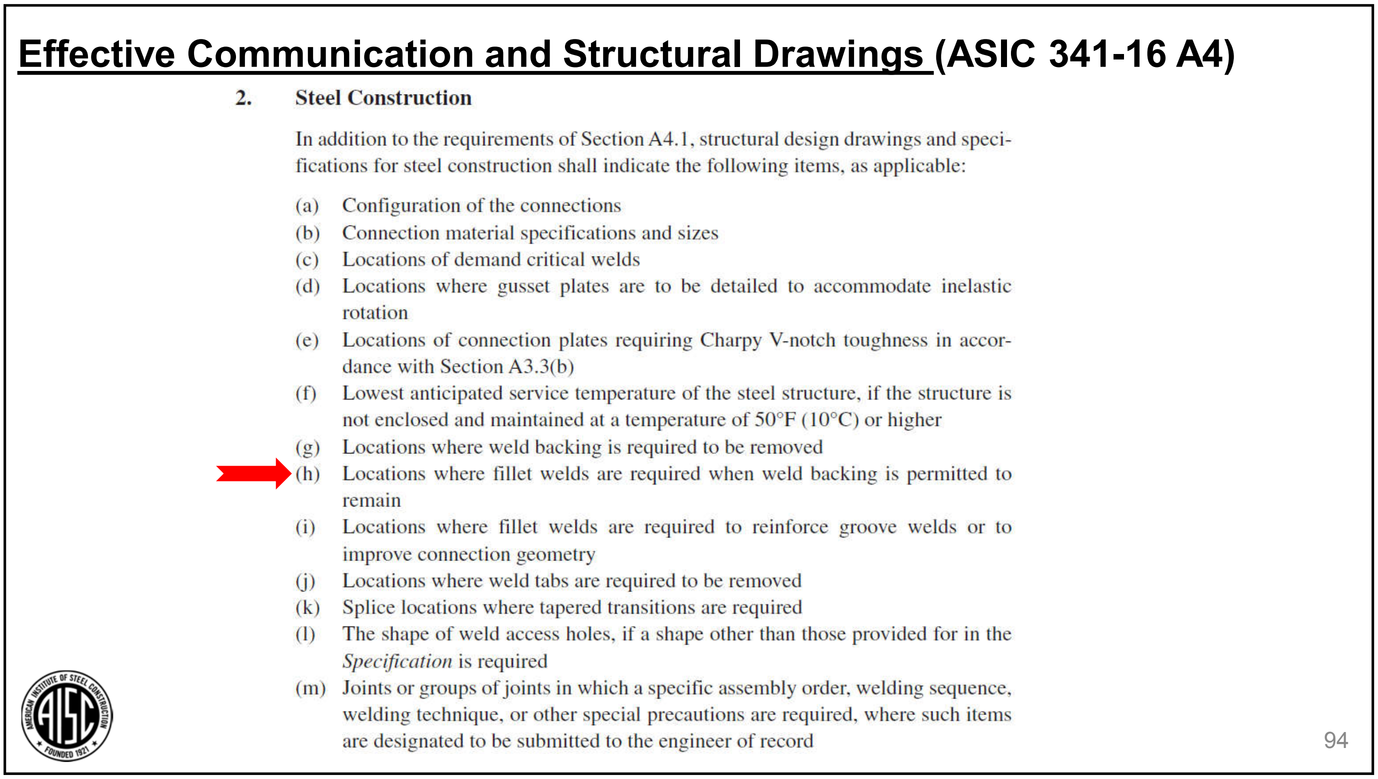

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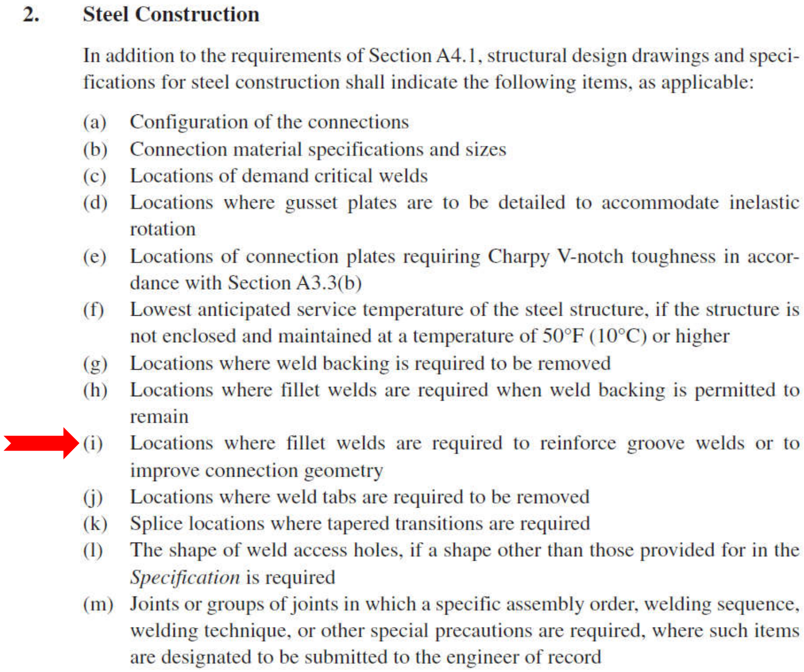

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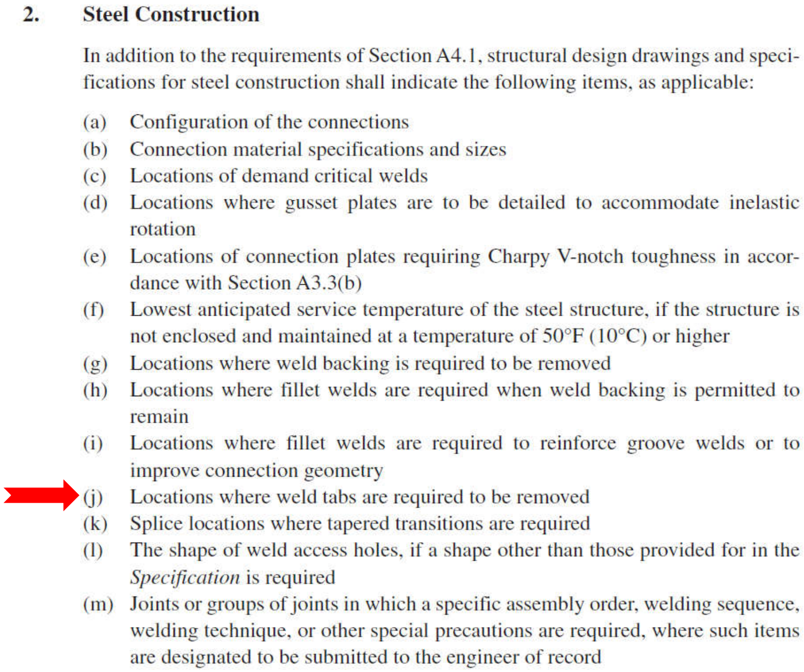

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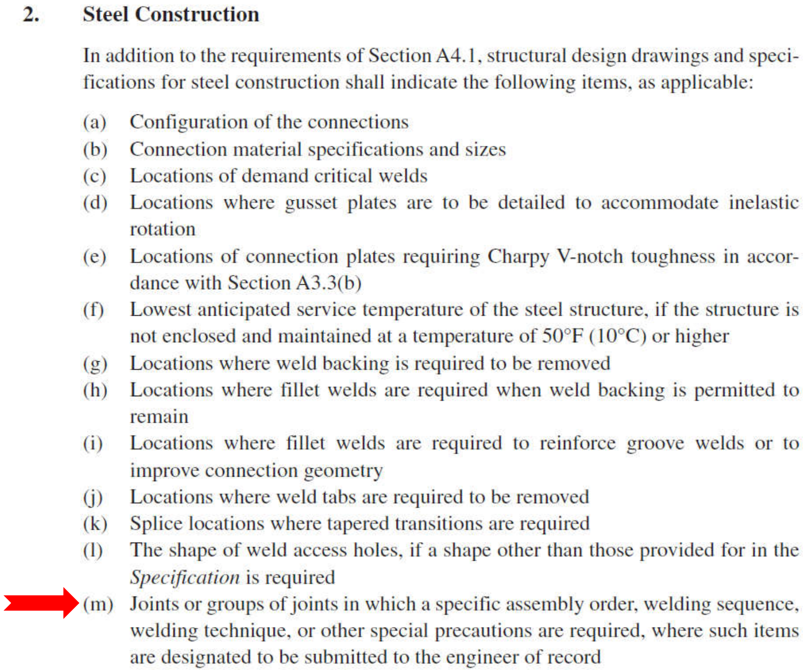

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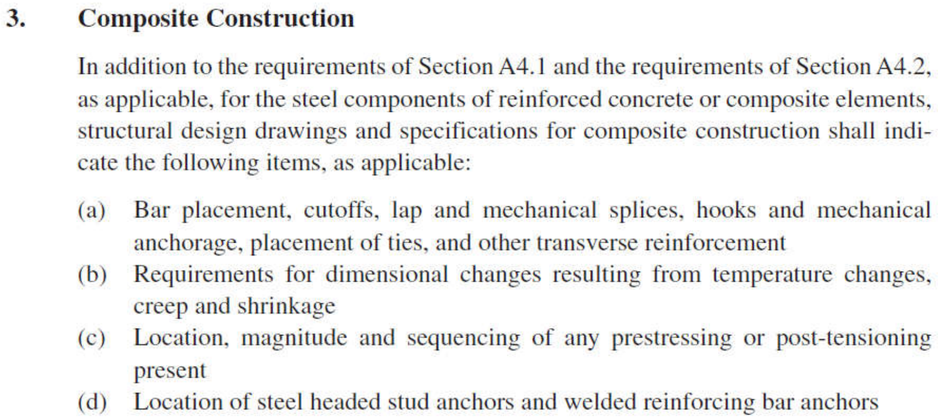
Effective Communication and Structural Drawings (ASIC 341-16 A4)



3. Composite Construction

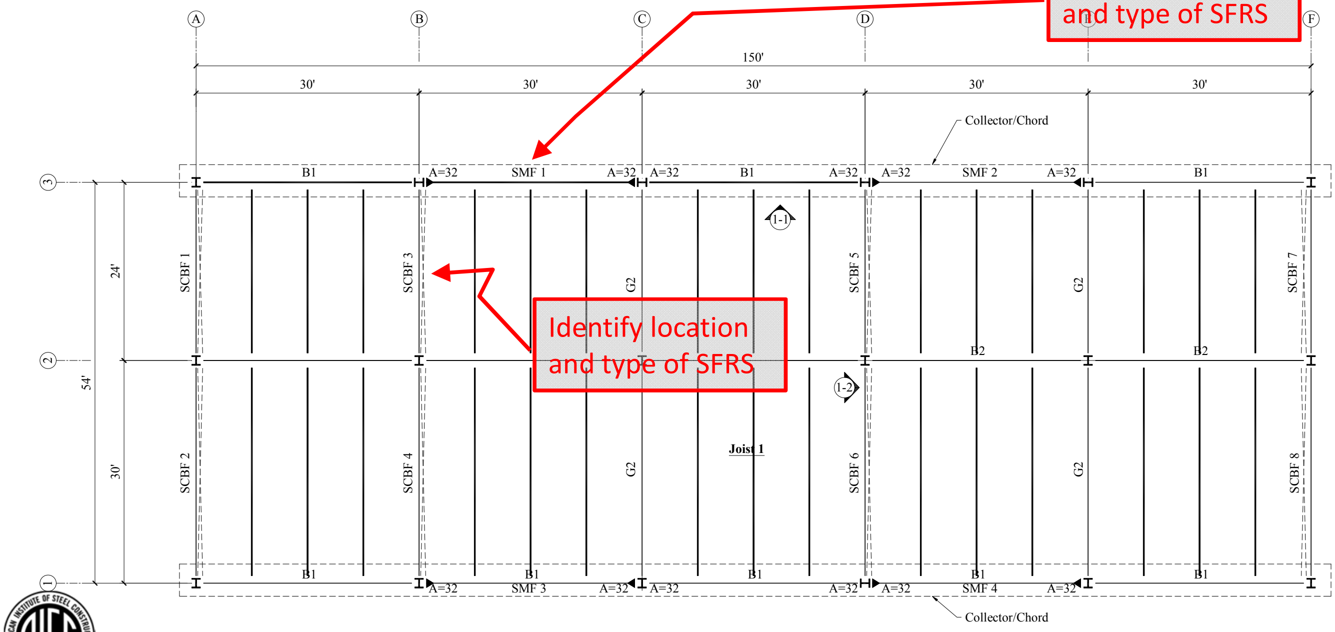
In addition to the requirements of Section A4.1 and the requirements of Section A4.2, as applicable, for the steel components of reinforced concrete or composite elements, structural design drawings and specifications for composite construction shall indicate the following items, as applicable:

(a) Bar placement, cutoffs, lap and mechanical splices, hooks and mechanical anchorage, placement of ties, and other transverse reinforcement
(b) Requirements for dimensional changes resulting from temperature changes, creep and shrinkage
(c) Location, magnitude and sequencing of any prestressing or post-tensioning present
(d) Location of steel headed stud anchors and welded reinforcing bar anchors

There are similar requirements for composite construction used in seismic applications

Effective Communication and Structural Drawings

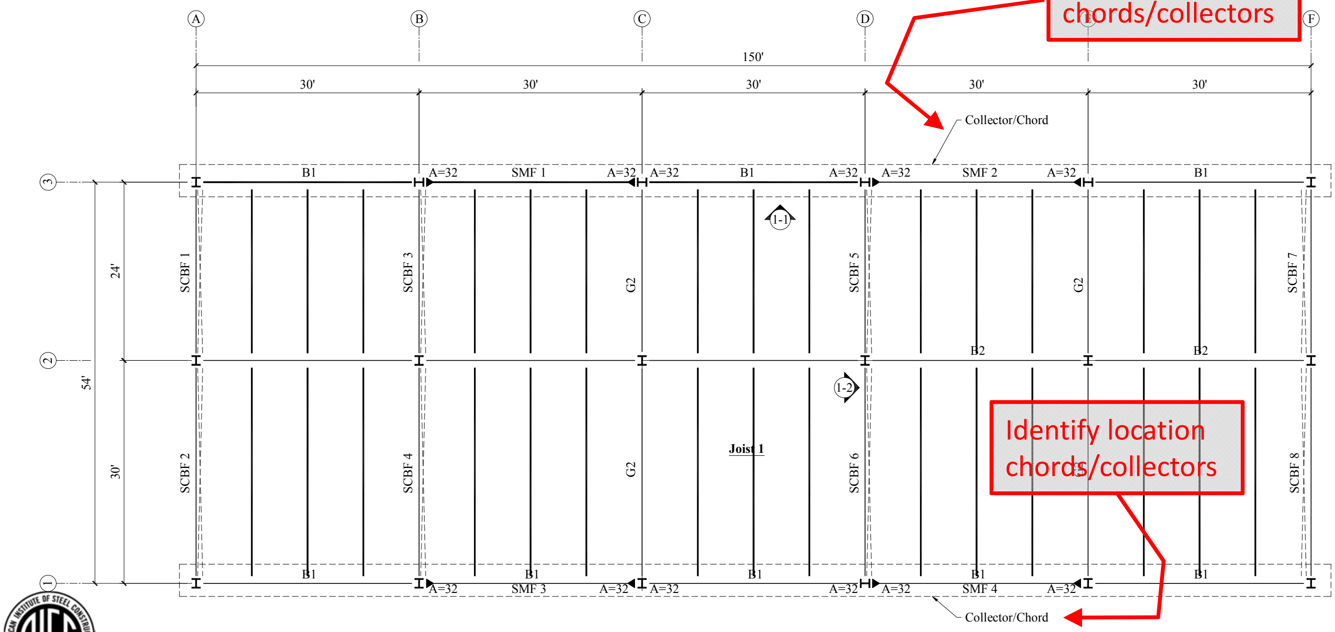
Identify location and type of SFRS



Identify location and type of SFRS

Effective Communication and Structural Drawings

Identify location chords/collectors



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Effective Communication and Structural Drawings

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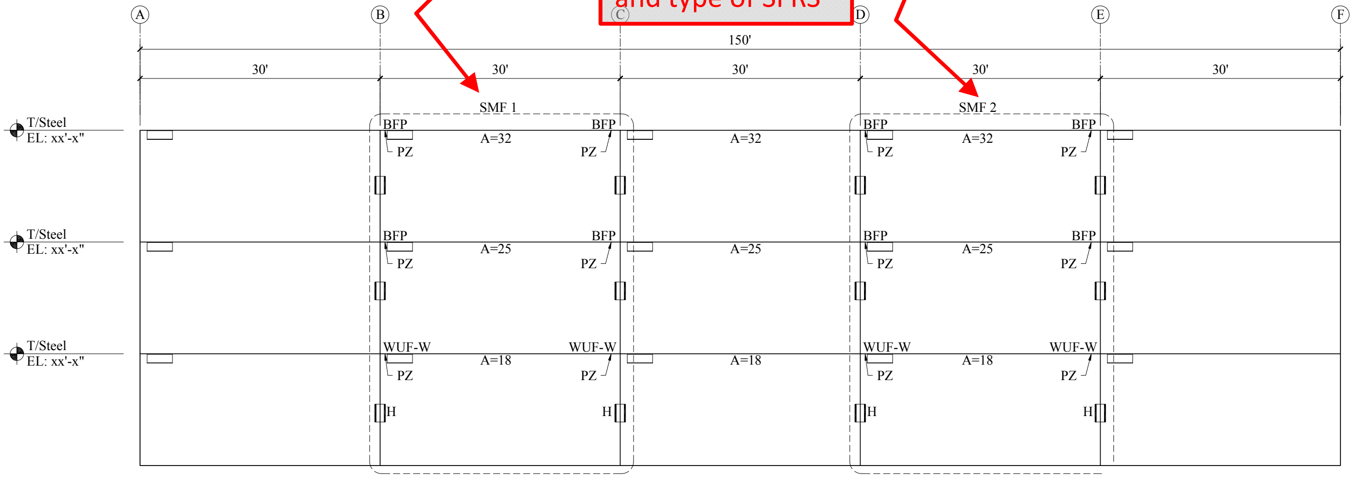

Effective Communication and Structural Drawings

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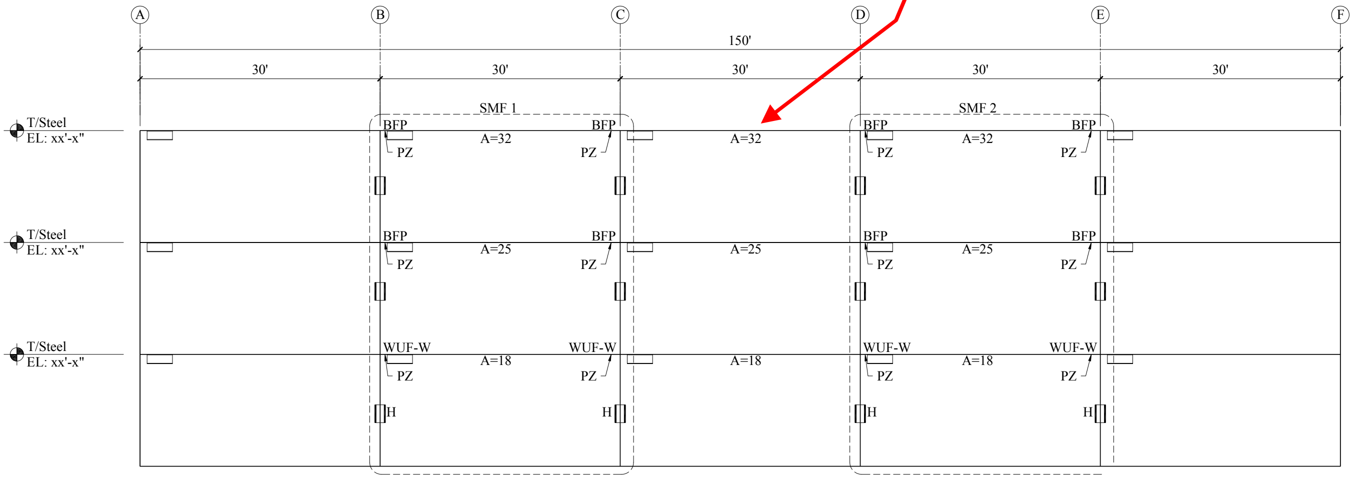
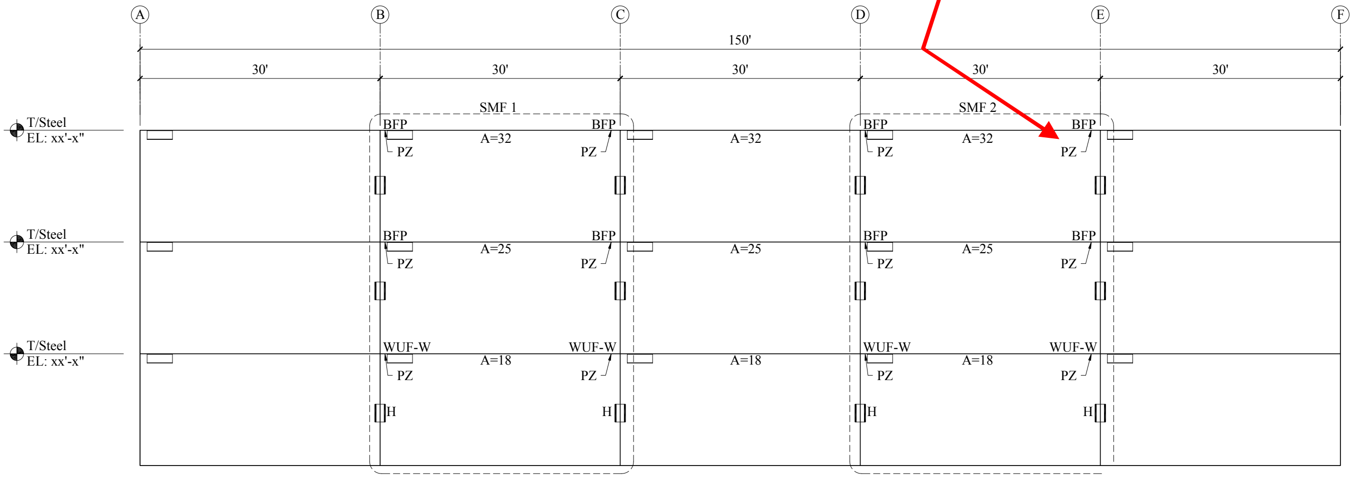


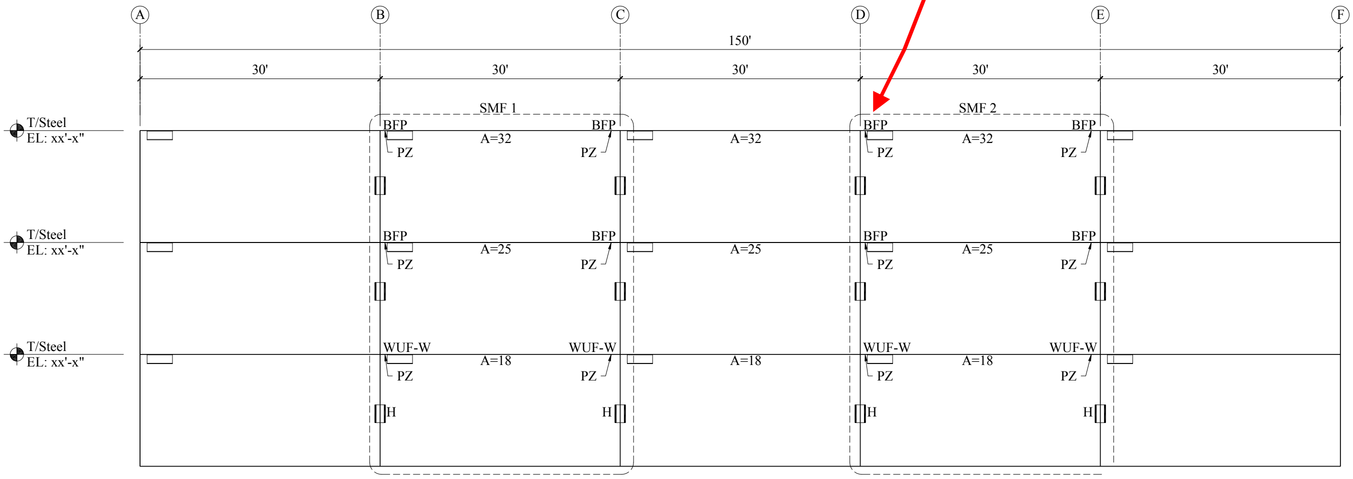
Effective Communication and Structural Drawings

Identify location of protected zones

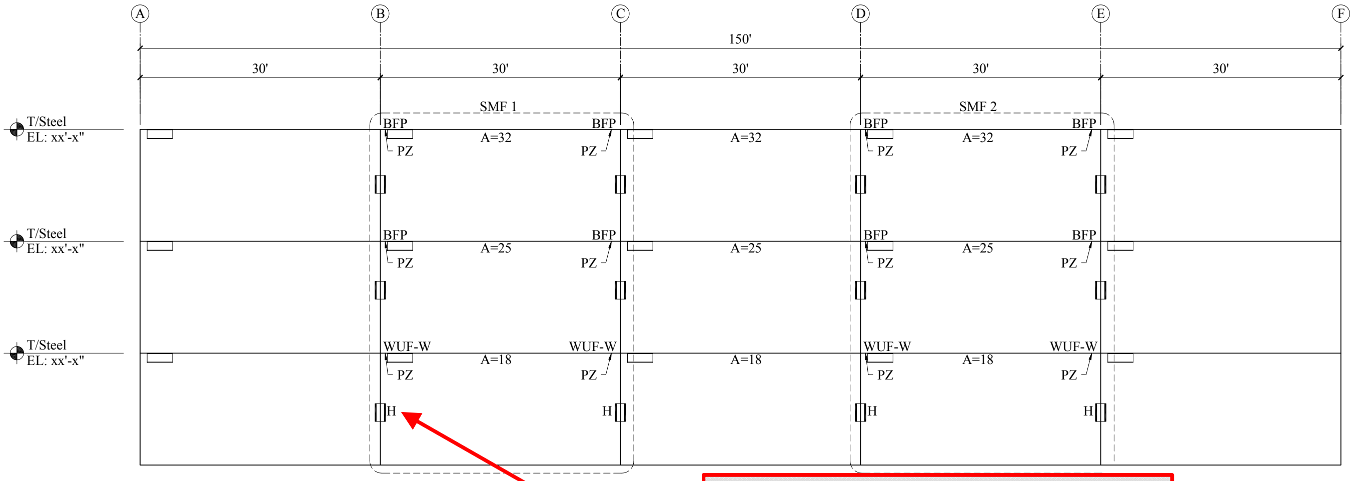


Effective Communication and Structural Drawings

Identify type of prequalified connection

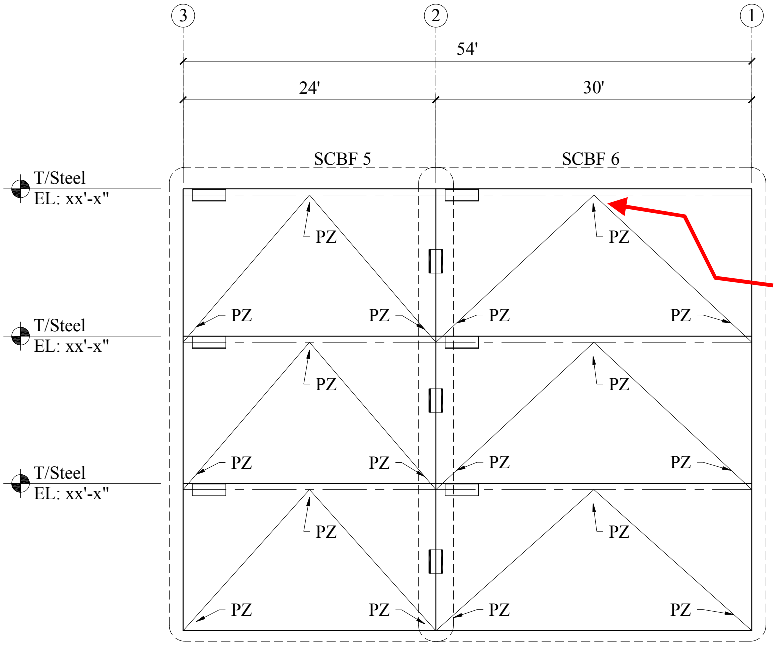


Effective Communication and Structural Drawings



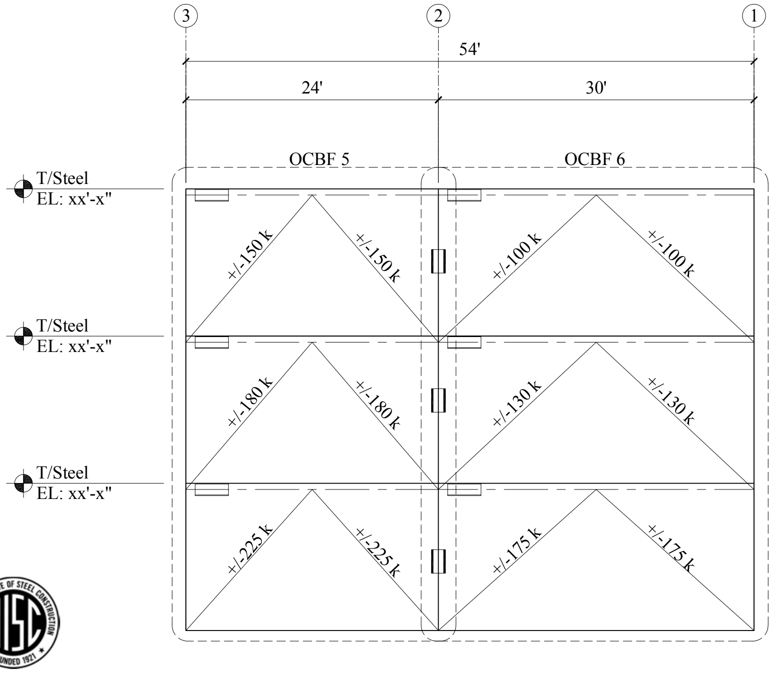
Identify members/hardware where notch toughness is required (heavy sections/thick plates)

Effective Communication and Structural Drawings



Identify:
- Work points,
- Protected zones, and
- Accommodation of brace buckling

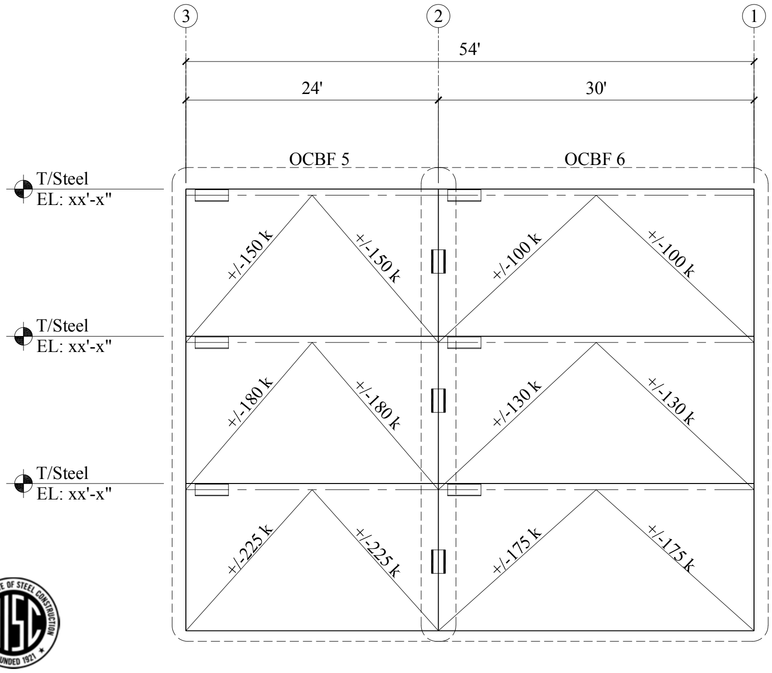
Effective Communication and Structural Drawings



If OCBF, clarify the required connection design loads…

If $R_yF_yA_g$, don't post loads on the braces

Effective Communication and Structural Drawings



If OCBF, clarify the required connection design loads…

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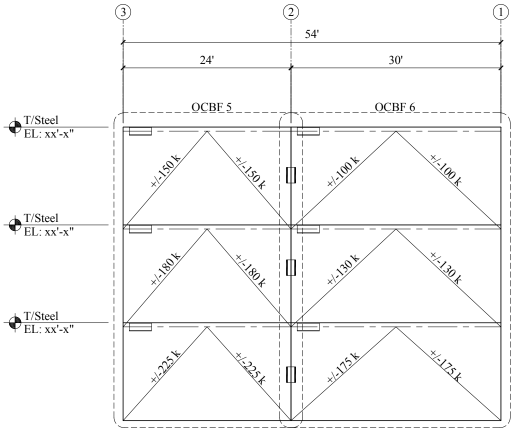
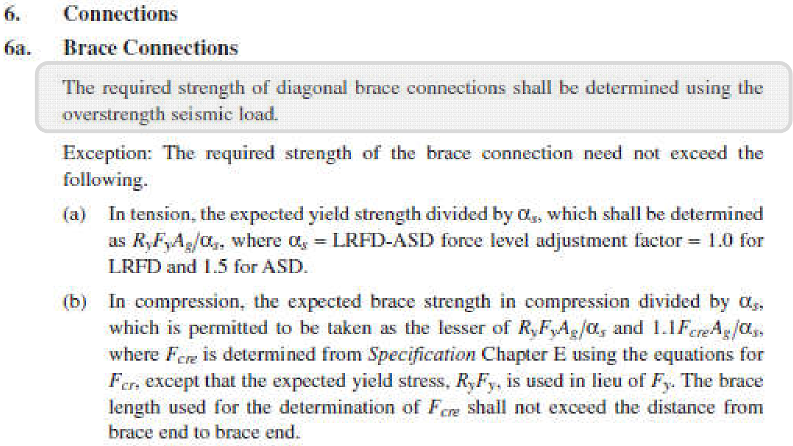
If exception is not taken, post the total required connection design forces; not something that needs to be modified (e.g., overstrength)

Effective Communication and Structural Drawings

AISC 341-16 F2.6a





6. Connections

6a. Brace Connections

The required strength of diagonal brace connections shall be determined using the overstrength seismic load.

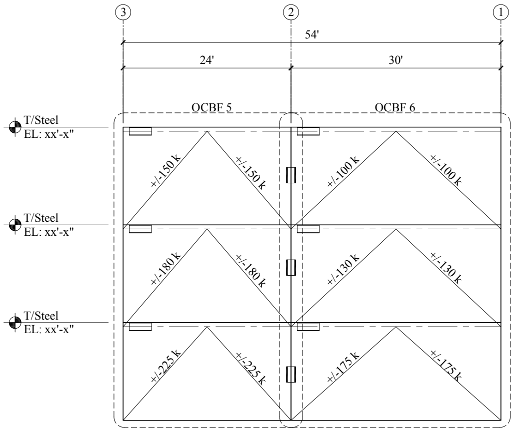
Exception: The required strength of the brace connection need not exceed the following.

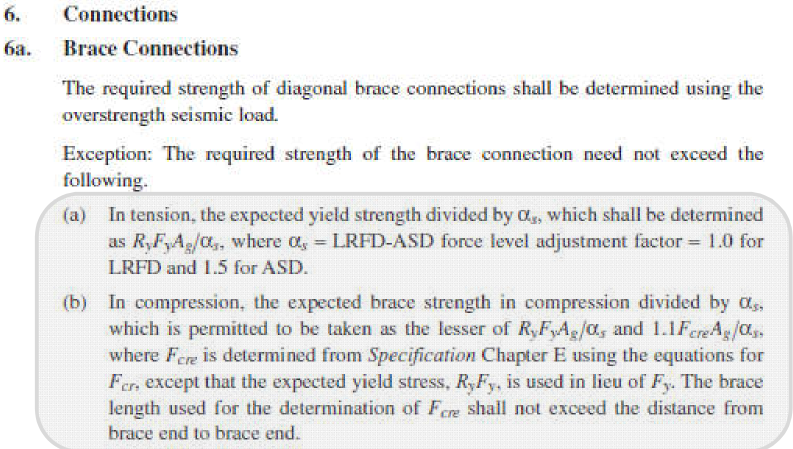
(a) In tension, the expected yield strength divided by α_s, which shall be determined as $R_yF_yA_g/\alpha_s$, where α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.

(b) In compression, the expected brace strength in compression divided by α_s, which is permitted to be taken as the lesser of $R_yF_yA_g/\alpha_s$ and $1.1F_{cre}A_g/\alpha_s$, where F_{cre} is determined from *Specification* Chapter E using the equations for F_{cr}, except that the expected yield stress, R_yF_y, is used in lieu of F_y. The brace length used for the determination of F_{cre} shall not exceed the distance from brace end to brace end.

Effective Communication and Structural Drawings

AISC 341-16 F2.6a





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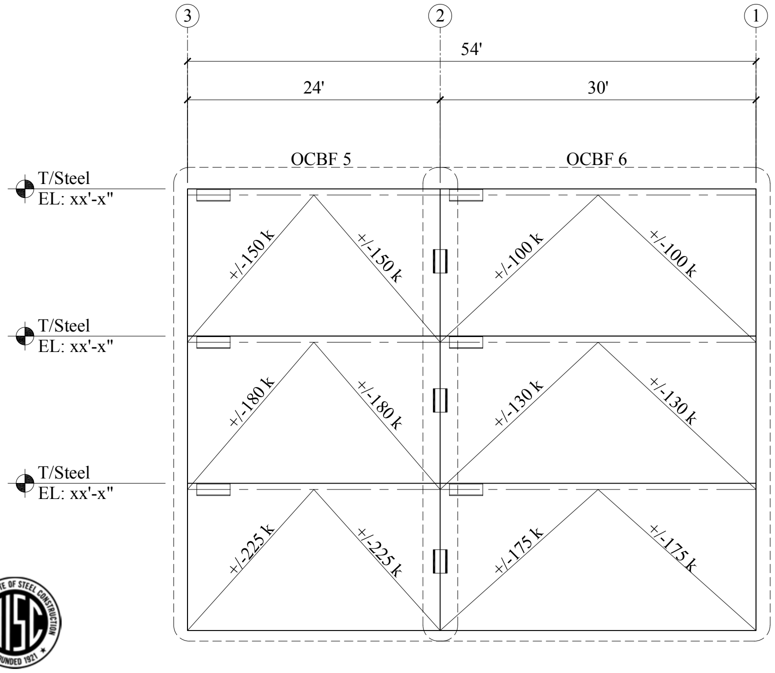
Effective Communication and Structural Drawings



The design documents should clearly provide the design forces for the brace connection

Poll Question 2

Q: Where SCBF systems are identified on construction documents, AISC 341, Section F2, clearly defines where brace buckling accommodation is required. Therefore, it is up to the fabricator and connection designer to know this with no further identification on the design documents.

a) True

b) False

Select your answer!

Stability Bracing (AISC 341 Section D1.2 and specific systems)

Many systems addressed in AISC 341 require stability bracing for the beams in the seismic force resisting system

- Ensure beam can reach its expected plastic strength in IMF and SMF systems
- Ensure no LTB in beams of braced frame systems with inverted V- and V-type configurations
- etc.

- The identification of locations and requirements for such bracing is rarely provided on design documents

- Framing should be configured to facilitate such bracing, and locations need to be identified in the design documents

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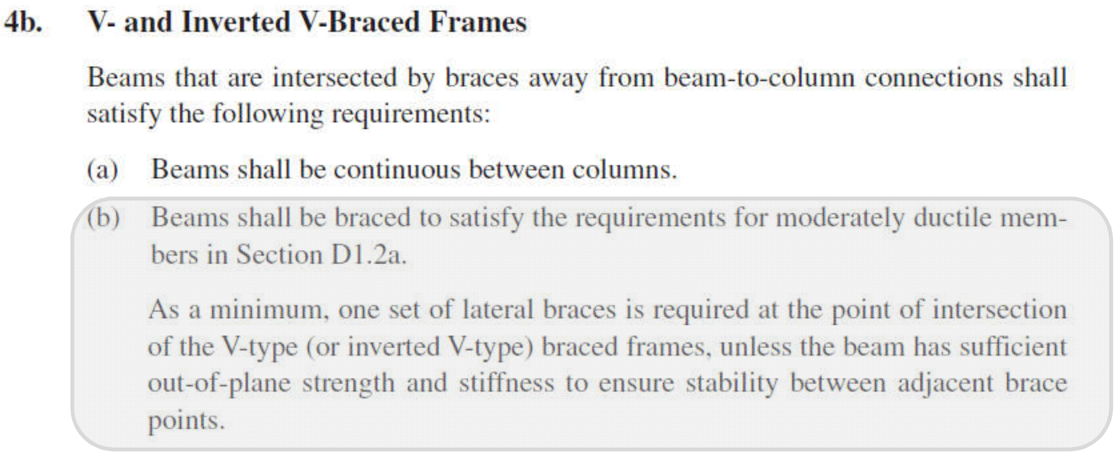
Stability Bracing (AISC 341 Section D1.2 and specific systems)

For example, AISC 341 Section F2 provides the requirements for the special concentrically braced frame system



4b. V- and Inverted V-Braced Frames

Beams that are intersected by braces away from beam-to-column connections shall satisfy the following requirements:

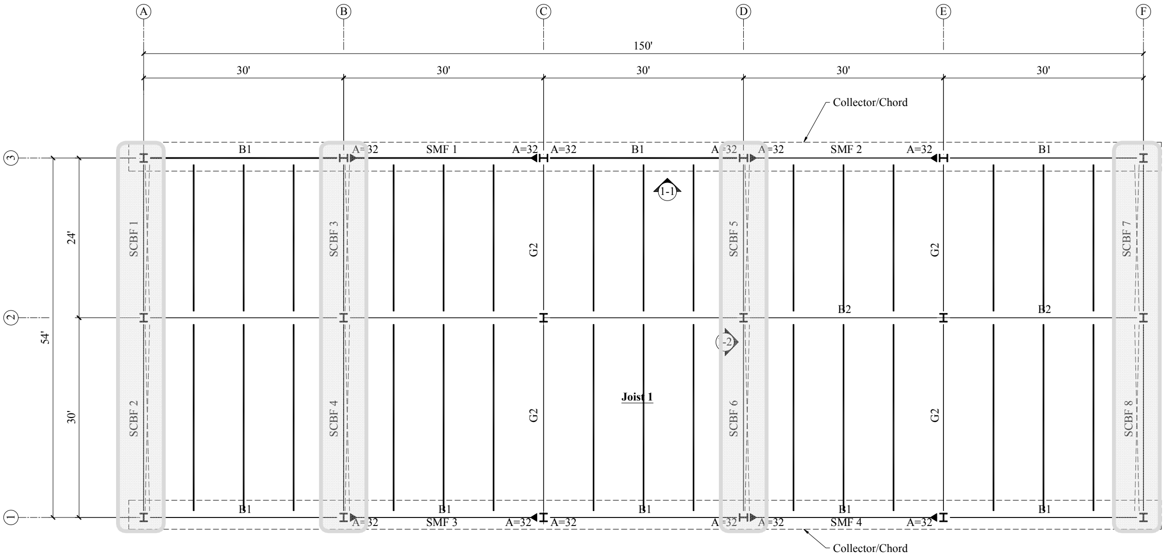
(a) Beams shall be continuous between columns.

(b) Beams shall be braced to satisfy the requirements for moderately ductile members in Section D1.2a.

As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) braced frames, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

Stability Bracing (AISC 341 Section D1.2 and specific systems)

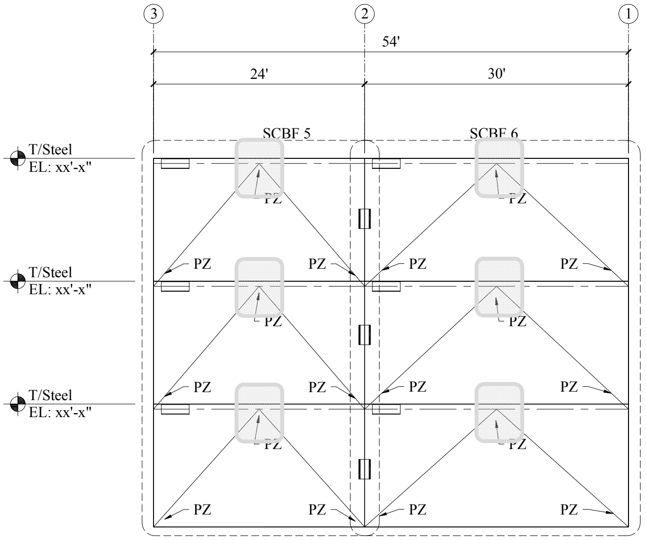
For example, AISC 341 Section F2 provides the requirements for the special concentrically braced frame system



Stability Bracing (AISC 341 Section D1.2 and specific systems)

For example, AISC 341 Section F2 provides the requirements for the special concentrically braced frame system

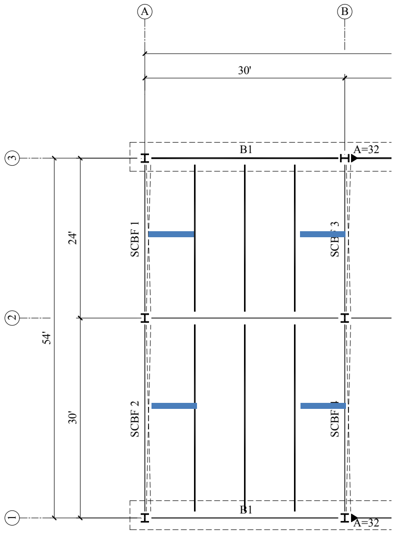
The framing plan provides no means of satisfying the lateral bracing requirements with an infill beam



Stability Bracing (AISC 341 Section D1.2 and specific systems)

For example, AISC 341 Section F2 provides the requirements for the special concentrically braced frame system

One option is to add an infill beam

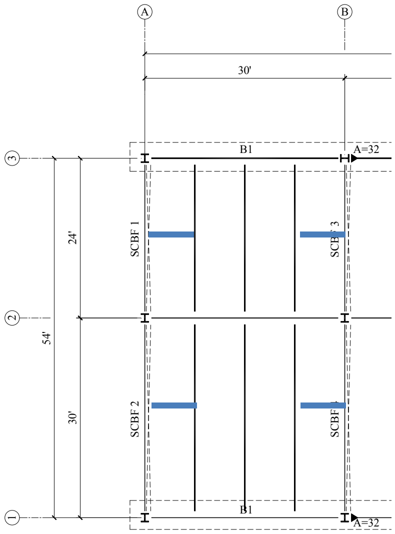


Stability Bracing (AISC 341 Section D1.2 and specific systems)

For example, AISC 341 Section F2 provides the requirements for the special concentrically braced frame system

Another option would be to verify that the slab/deck provides sufficient lateral bracing to the top flange, and then add a kicker from the bottom flange to the underside of the deck

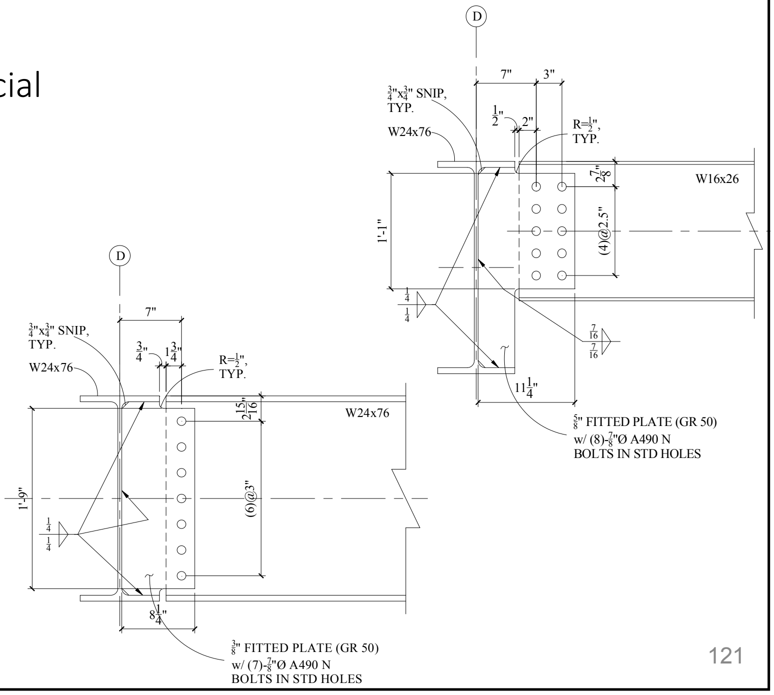
Either way, the SEoR needs to provide direction



Stability Bracing (AISC 341 Section D1.2 and specific systems)

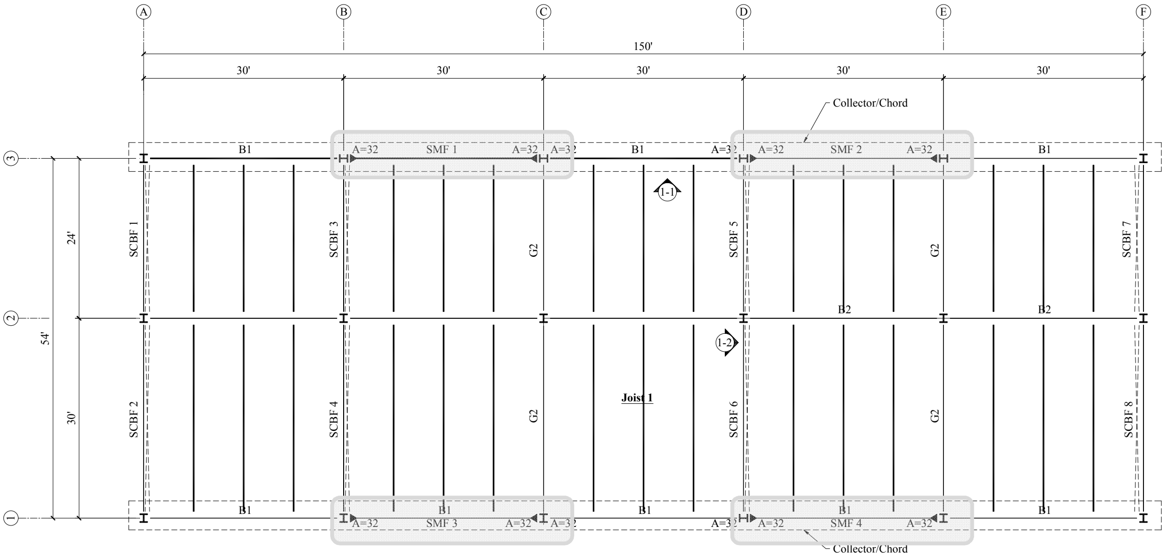
For example, AISC 341 Section F2 provides the requirements for the special concentrically braced frame system

Examples 4.3.4 and 4.3.5 in the SDM provide details for stability bracing connections for equal depth and unequal depth beams, respectively.

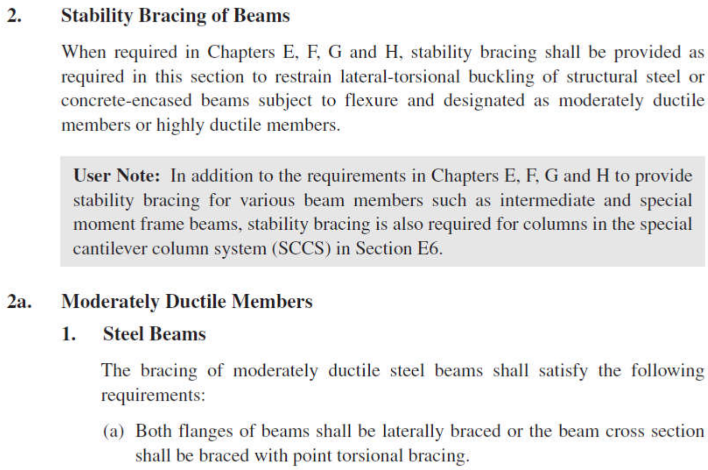


Stability Bracing (AISC 341 Section D1.2 and specific systems)

Moment Frames…



Stability Bracing (AISC 341 Section D1.2 and specific systems)



2. **Stability Bracing of Beams**

When required in Chapters E, F, G and H, stability bracing shall be provided as required in this section to restrain lateral-torsional buckling of structural steel or concrete-encased beams subject to flexure and designated as moderately ductile members or highly ductile members.

User Note: In addition to the requirements in Chapters E, F, G and H to provide stability bracing for various beam members such as intermediate and special moment frame beams, stability bracing is also required for columns in the special cantilever column system (SCCS) in Section E6.

2a. **Moderately Ductile Members**

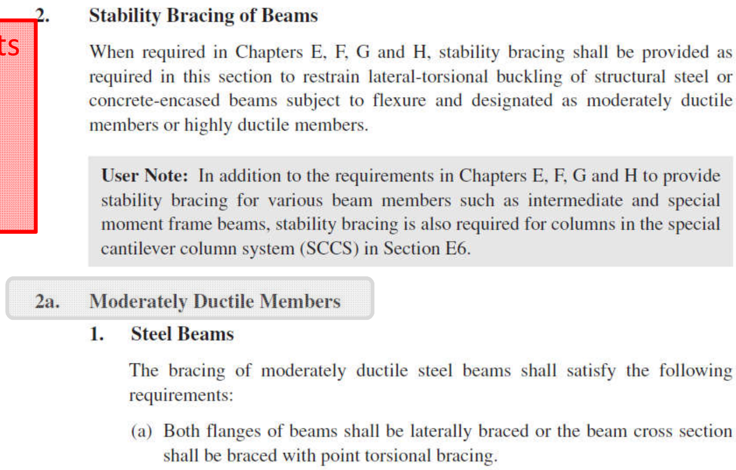
1. **Steel Beams**

The bracing of moderately ductile steel beams shall satisfy the following requirements:

(a) Both flanges of beams shall be laterally braced or the beam cross section shall be braced with point torsional bracing.

Stability Bracing (AISC 341 Section D1.2 and specific systems)

Design documents should identify moderately and highly ductile members



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User Note: In addition to the requirements in Chapters E, F, G and H to provide stability bracing for various beam members such as intermediate and special moment frame beams, stability bracing is also required for columns in the special cantilever column system (SCCS) in Section E6.

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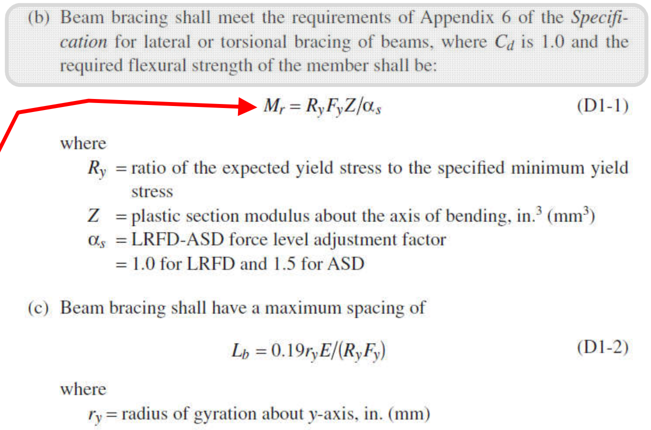
Stability Bracing (AISC 341 Section D1.2 and specific systems)



(b) Beam bracing shall meet the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams, where C_d is 1.0 and the required flexural strength of the member shall be:

$$M_r = R_y F_y Z / \alpha_s \qquad \text{(D1-1)}$$

where

R_y = ratio of the expected yield stress to the specified minimum yield stress

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD

(c) Beam bracing shall have a maximum spacing of

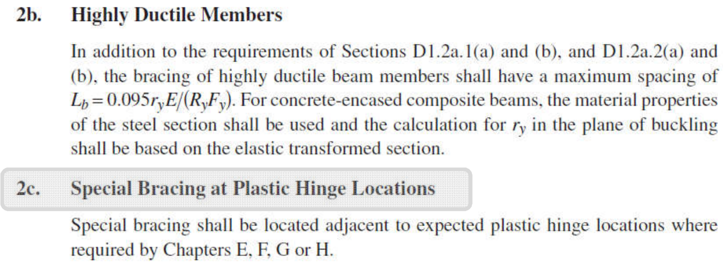
$$L_b = 0.19 r_y E/(R_y F_y) \qquad \text{(D1-2)}$$

where

r_y = radius of gyration about y-axis, in. (mm)

Required moment to be used with App. 6 equations

Stability Bracing (AISC 341 Section D1.2 and specific systems)



2b. Highly Ductile Members

In addition to the requirements of Sections D1.2a.1(a) and (b), and D1.2a.2(a) and (b), the bracing of highly ductile beam members shall have a maximum spacing of $L_b = 0.095 r_y E/(R_y F_y)$. For concrete-encased composite beams, the material properties of the steel section shall be used and the calculation for r_y in the plane of buckling shall be based on the elastic transformed section.

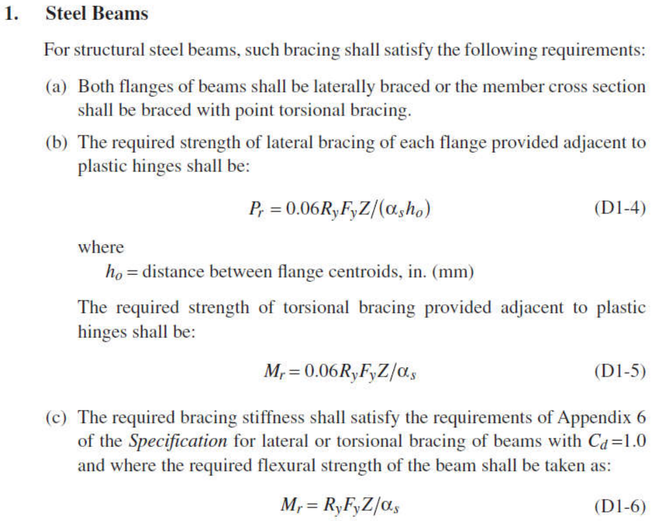
2c. Special Bracing at Plastic Hinge Locations

Special bracing shall be located adjacent to expected plastic hinge locations where required by Chapters E, F, G or H.

AISC 341 prescribes which systems require stability bracing and whether members are considered moderate or highly ductile members

Stability Bracing (AISC 341 Section D1.2 and specific systems)

Locations of required beam stability bracing as required in AISC 341 should be identified on design documents (AISC 341-16 D1.2c.1).



1. **Steel Beams**

For structural steel beams, such bracing shall satisfy the following requirements:

(a) Both flanges of beams shall be laterally braced or the member cross section shall be braced with point torsional bracing.

(b) The required strength of lateral bracing of each flange provided adjacent to plastic hinges shall be:

$$P_r = 0.06 R_y F_y Z / (\alpha_s h_o) \qquad \text{(D1-4)}$$

where

h_o = distance between flange centroids, in. (mm)

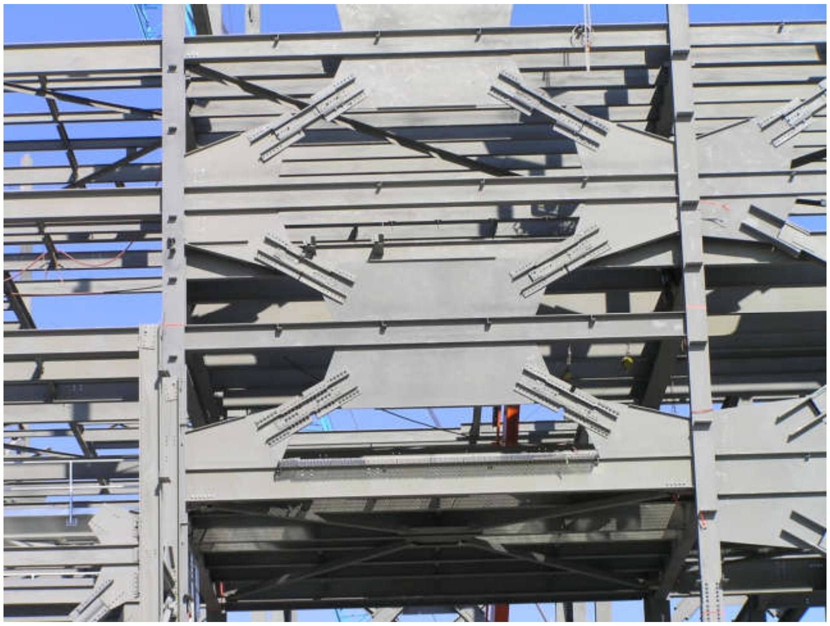
The required strength of torsional bracing provided adjacent to plastic hinges shall be:

$$M_r = 0.06 R_y F_y Z / \alpha_s \qquad \text{(D1-5)}$$

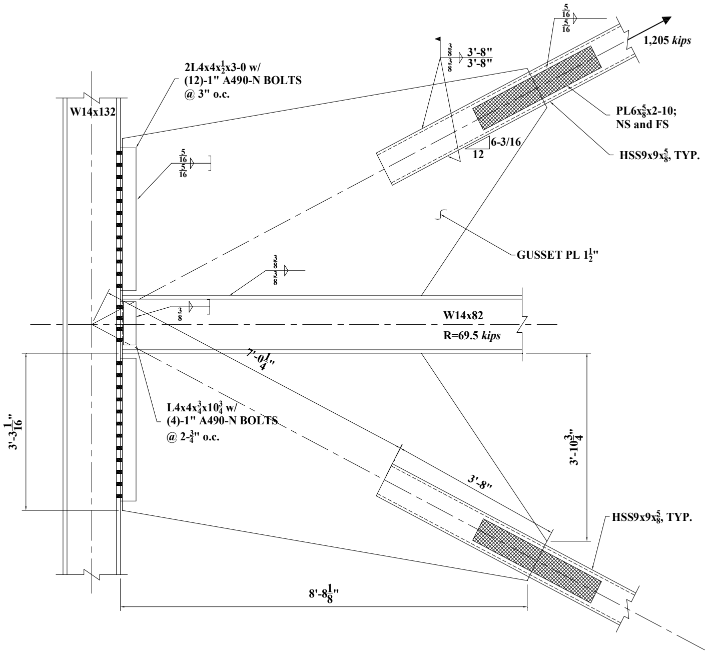
(c) The required bracing stiffness shall satisfy the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams with C_d =1.0 and where the required flexural strength of the beam shall be taken as:

$$M_r = R_y F_y Z / \alpha_s \qquad \text{(D1-6)}$$

SCBF – Accommodating Brace Buckling



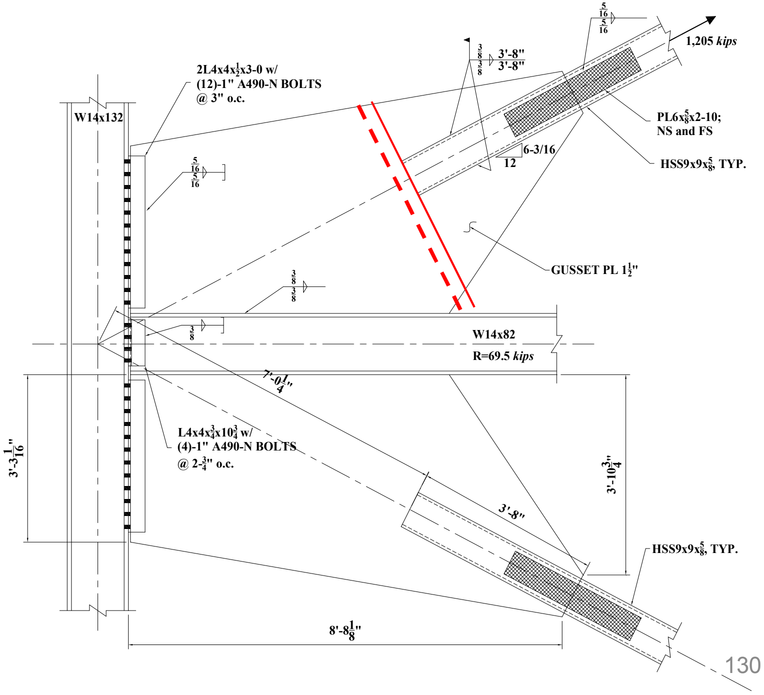
SCBF – Accommodating Brace Buckling

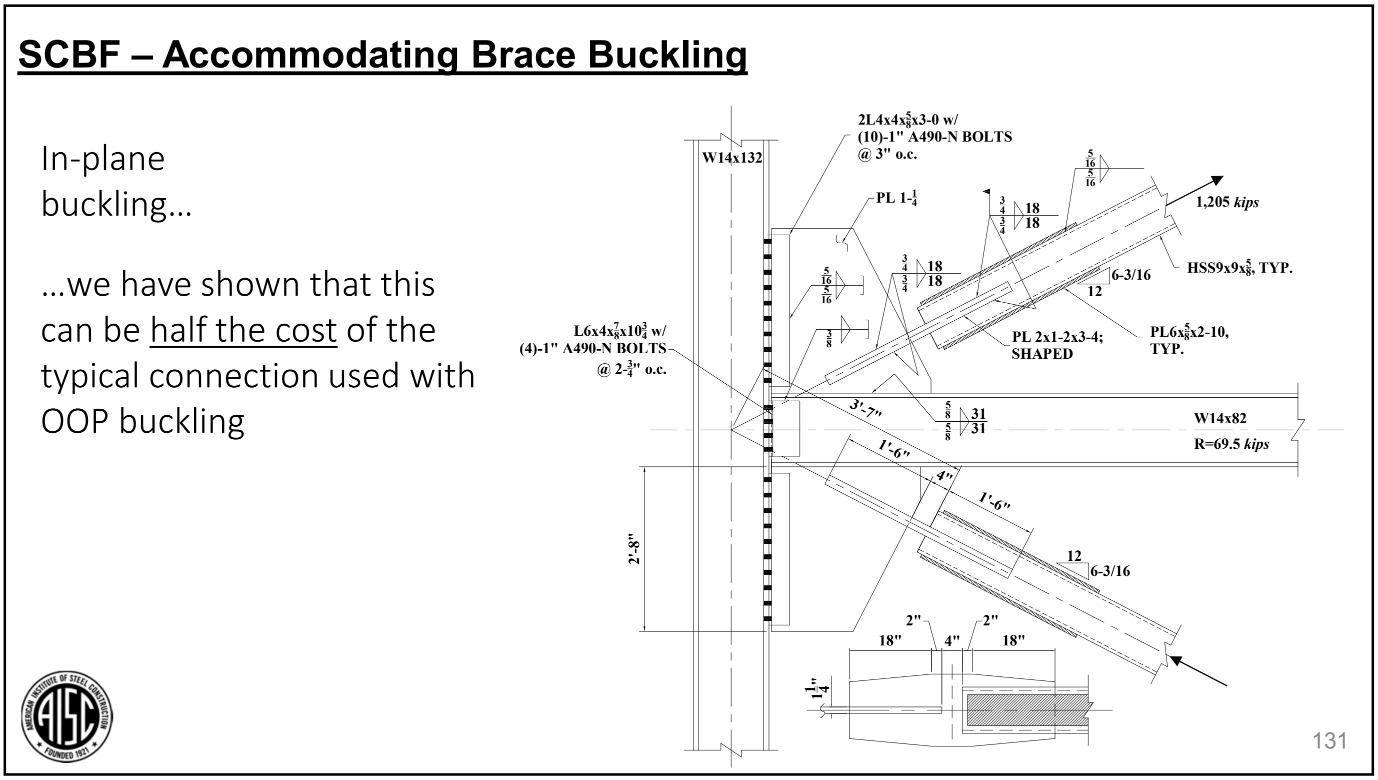


SCBF – Accommodating Brace Buckling

Lovingly known as the 2*t* method…

…Out-of-plane (OOP) buckling

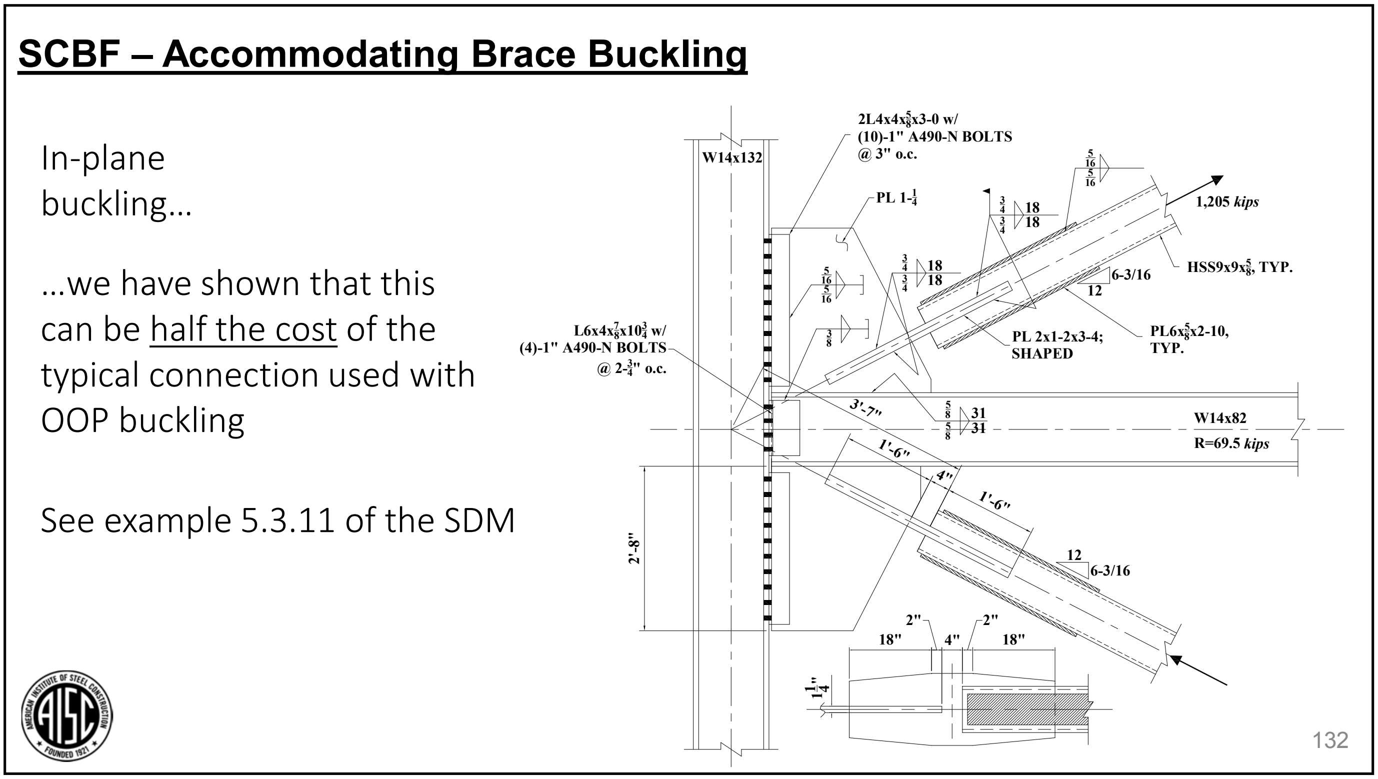




SCBF – Accommodating Brace Buckling

In-plane buckling…

…we have shown that this can be half the cost of the typical connection used with OOP buckling



SCBF – Accommodating Brace Buckling

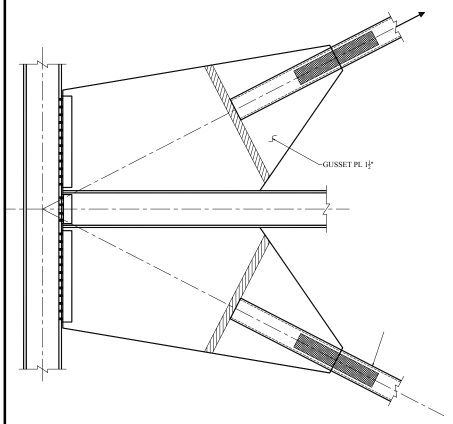
In-plane buckling…

…we have shown that this can be half the cost of the typical connection used with OOP buckling

See example 5.3.11 of the SDM

SCBF – Accommodating Brace Buckling

Other Methods for OOP

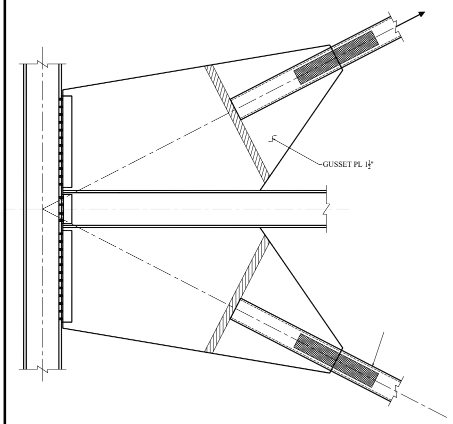


Linear – 2*t* – 30 deg.

Standard Way

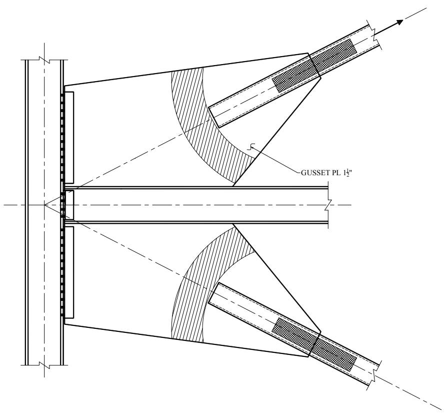
SCBF – Accommodating Brace Buckling

Other Methods for OOP



Linear – 2*t* – 30 deg.

Standard Way

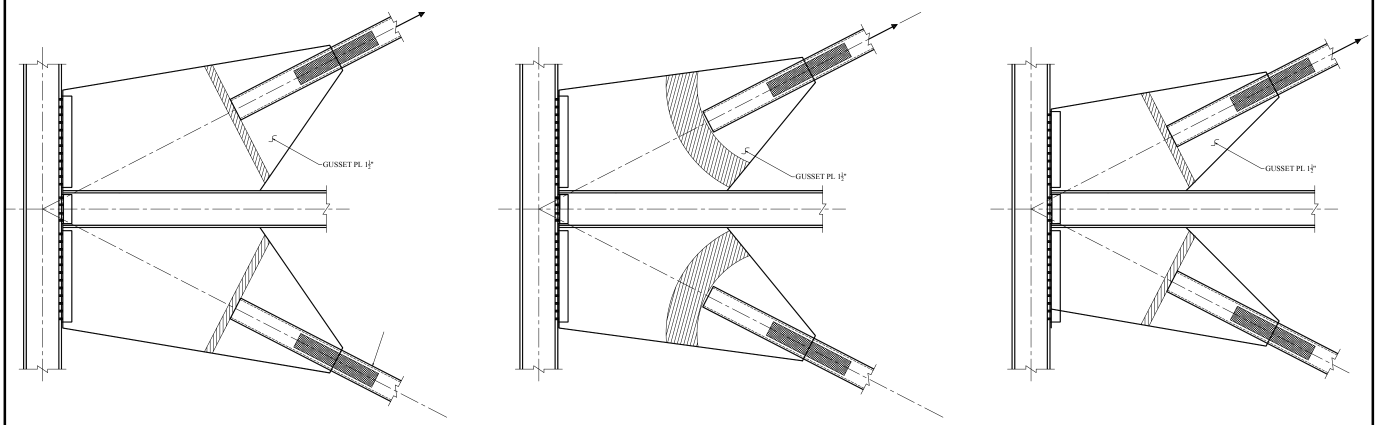


Elliptical – 8*t*

SDM
Example 5.3.10

SCBF – Accommodating Brace Buckling

Other Methods for OOP



Linear – 2*t* – 30 deg.

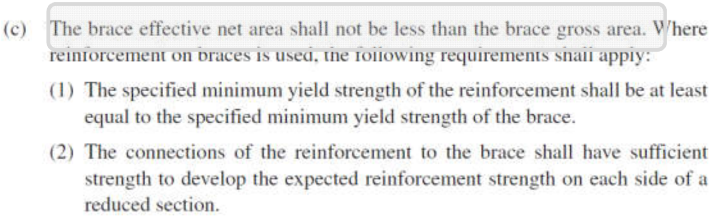
Elliptical – 8*t*

Linear – 2*t* - <30 deg.

Standard Way

SDM
Example 5.3.10

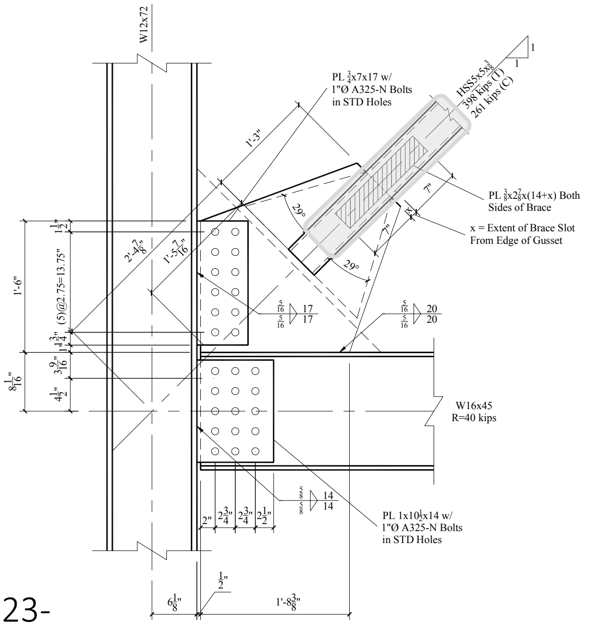
SCBF – Net Section Rupture (F2.5b(c))



(c) The brace effective net area shall not be less than the brace gross area. Where reinforcement on braces is used, the following requirements shall apply:

(1) The specified minimum yield strength of the reinforcement shall be at least equal to the specified minimum yield strength of the brace.

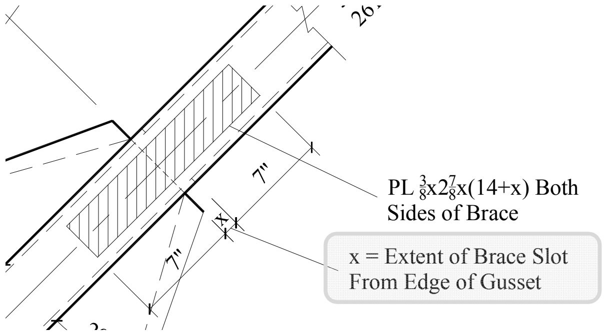
(2) The connections of the reinforcement to the brace shall have sufficient strength to develop the expected reinforcement strength on each side of a reduced section.



This effects brace types such as HSS slotted over gusset

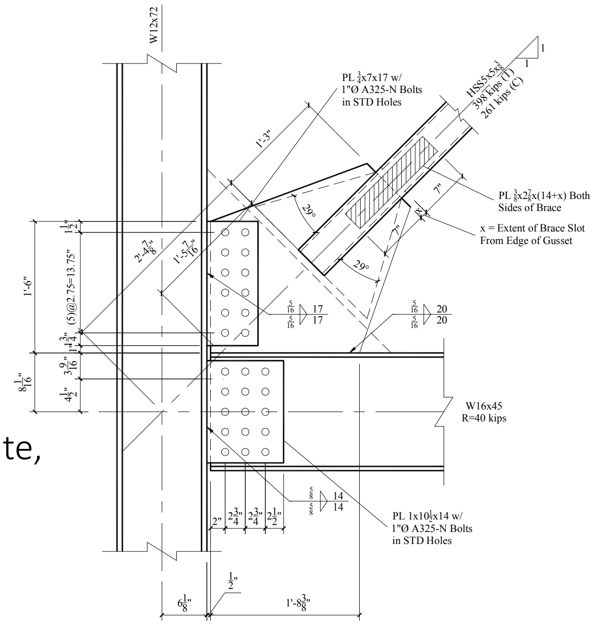
See example 5.3.7 of the SDM, pp. 5-223-5-225 for example calculations

SCBF – Net Section Rupture (F2.5b(c))



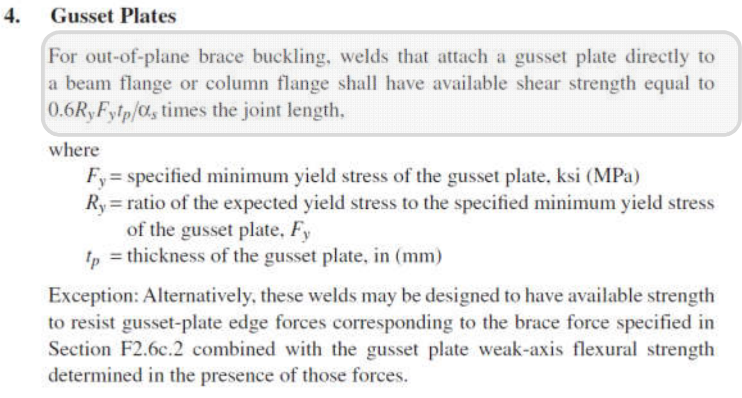
PL $\frac{3}{8}$x2$\frac{7}{8}$x(14+x) Both Sides of Brace

x = Extent of Brace Slot From Edge of Gusset



When specifying the extent of the reinforcing plate, be sure to allow for excessive slot in HSS usually provided to accommodate erection

SCBF – Gusset Plate Welds (F2.6c.4)



4. Gusset Plates

For out-of-plane brace buckling, welds that attach a gusset plate directly to a beam flange or column flange shall have available shear strength equal to $0.6R_yF_yt_p/\alpha_s$ times the joint length.

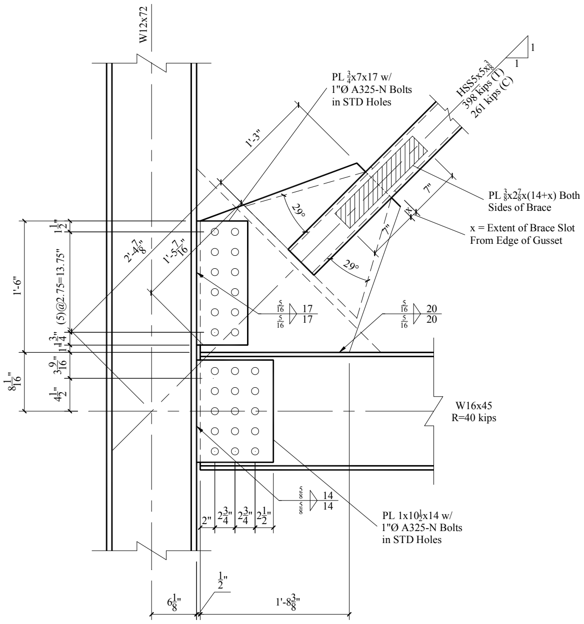
where

F_y = specified minimum yield stress of the gusset plate, ksi (MPa)

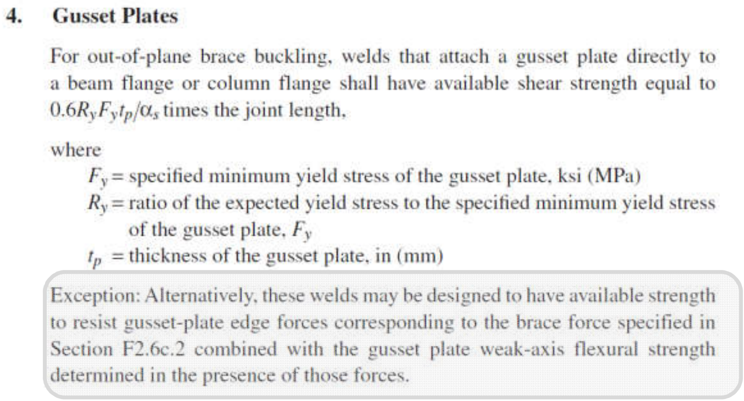
R_y = ratio of the expected yield stress to the specified minimum yield stress of the gusset plate, F_y

t_p = thickness of the gusset plate, in (mm)

Exception: Alternatively, these welds may be designed to have available strength to resist gusset-plate edge forces corresponding to the brace force specified in Section F2.6c.2 combined with the gusset plate weak-axis flexural strength determined in the presence of those forces.



SCBF – Gusset Plate Welds (F2.6c.4)



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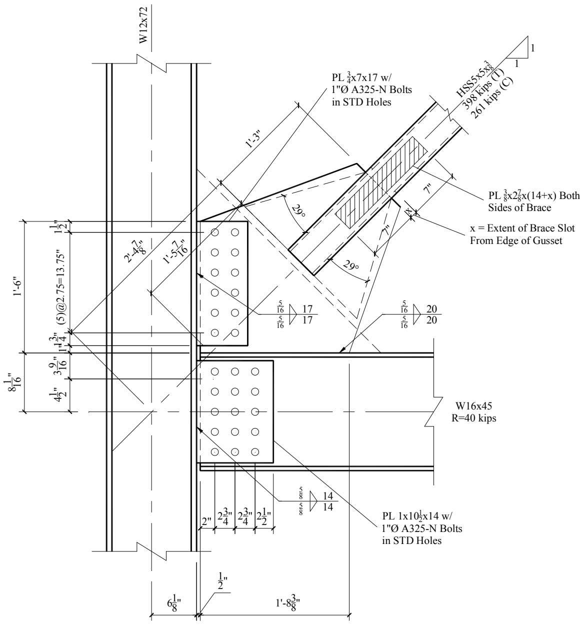

Carter, Charles J.; Muir, Larry S.; Dowswell, Bo (2016). "Establishing and Developing the Weak-Axis Strength of Plates Subjected to Applied Loads," *Engineering Journal*, American Institute of Steel Construction, Vol. 53, pp. 147-157.

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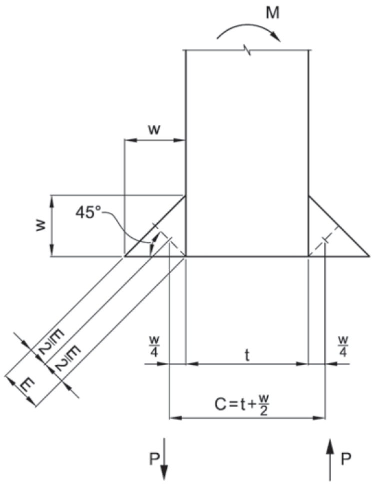

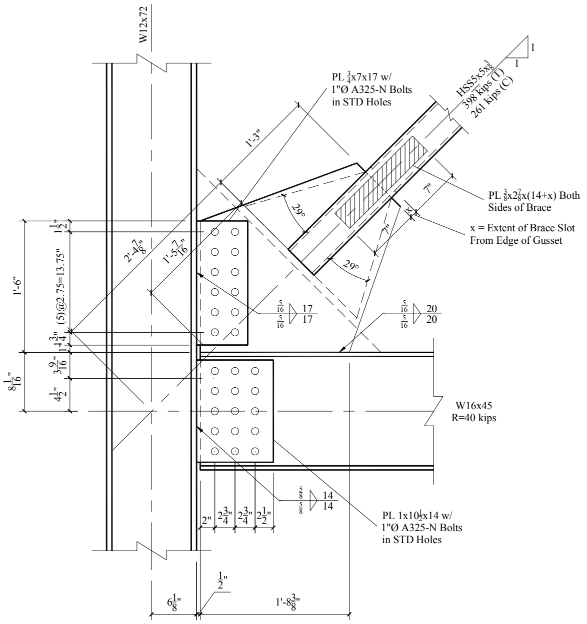

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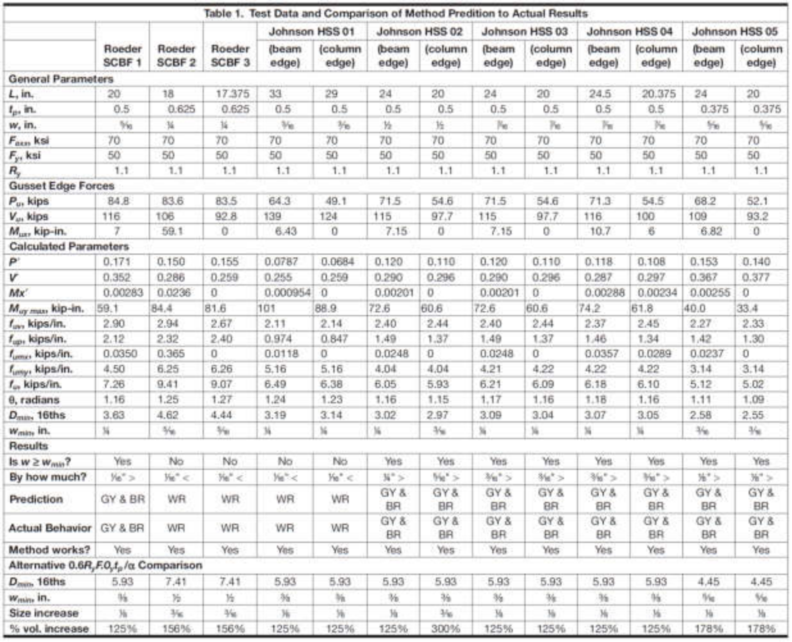

Table 1. Test Data and Comparison of Method Predition to Actual Results

				Johnson HSS 01		Johnson HSS 02		Johnson HSS 03		Johnson HSS 04		Johnson HSS 05	
	Roeder SCBF 1	Roeder SCBF 2	Roeder SCBF 3	(beam edge)	(column edge)	(beam edge)	(column edge)	(beam edge)	(column edge)	(beam edge)	(column edge)	(beam edge)	(column edge)
General Parameters													
L, in.	20	18	17.375	33	29	24	20	24	20	24.5	20.375	24	20
t_p, in.	0.5	0.625	0.625	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.375	0.375
w, in.	5/16	1/4	1/4	3/16	3/16	1/2	1/2	7/16	7/16	7/16	7/16	5/16	5/16
F_{exx}, ksi	70	70	70	70	70	70	70	70	70	70	70	70	70
F_y, ksi	50	50	50	50	50	50	50	50	50	50	50	50	50
R_y	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Gusset Edge Forces													
P_u, kips	84.8	83.6	83.5	64.3	49.1	71.5	54.6	71.5	54.6	71.3	54.5	68.2	52.1
V_u, kips	116	106	92.8	139	124	115	97.7	115	97.7	116	100	109	93.2
M_{ux}, kip-in.	7	59.1	0	6.43	0	7.15	0	7.15	0	10.7	6	6.82	0
Calculated Parameters													
P'	0.171	0.150	0.155	0.0787	0.0684	0.120	0.110	0.120	0.110	0.118	0.108	0.153	0.140
V'	0.352	0.286	0.259	0.255	0.259	0.290	0.296	0.290	0.296	0.287	0.297	0.367	0.377
Mx'	0.00283	0.0236	0	0.000954	0	0.00201	0	0.00201	0	0.00288	0.00234	0.00255	0
$M_{uy\,max}$, kip-in.	59.1	84.4	81.6	101	88.9	72.6	60.6	72.6	60.6	74.2	61.8	40.0	33.4
f_{uv}, kips/in.	2.90	2.94	2.67	2.11	2.14	2.40	2.44	2.40	2.44	2.37	2.45	2.27	2.33
f_{up}, kips/in.	2.12	2.32	2.40	0.974	0.847	1.49	1.37	1.49	1.37	1.46	1.34	1.42	1.30
f_{umx}, kips/in.	0.0350	0.365	0	0.0118	0	0.0248	0	0.0248	0	0.0357	0.0289	0.0237	0
f_{umy}, kips/in.	4.50	6.25	6.26	5.16	5.16	4.04	4.04	4.21	4.22	4.22	4.22	3.14	3.14
f_u, kips/in.	7.26	9.41	9.07	6.49	6.38	6.05	5.93	6.21	6.09	6.18	6.10	5.12	5.02
θ, radians	1.16	1.25	1.27	1.24	1.23	1.16	1.15	1.17	1.16	1.18	1.16	1.11	1.09
D_{min}, 16ths	3.63	4.62	4.44	3.19	3.14	3.02	2.97	3.09	3.04	3.07	3.05	2.58	2.55
w_{min}, in.	1/4	5/16	5/16	1/4	1/4	1/4	3/16	1/4	1/4	1/4	1/4	3/16	3/16
Results													
Is $w \ge w_{min}$?	Yes	No	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
By how much?	1/16" >	1/16" <	1/16" <	1/16" <	1/16" <	1/4" >	5/16" >	3/16" >	3/16" >	3/16" >	3/16" >	1/8" >	1/8" >
Prediction	GY & BR	WR	WR	WR	WR	GY & BR	GY & BR	GY & BR	GY & BR	GY & BR	GY & BR	GY & BR	GY & BR
Actual Behavior	GY & BR	WR	WR	WR	WR	GY & BR	GY & BR	GY & BR	GY & BR	GY & BR	GY & BR	GY & BR	GY & BR
Method works?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Alternative $0.6R_yF.0_yt_p/\alpha$ Comparison													
D_{min}, 16ths	5.93	7.41	7.41	5.93	5.93	5.93	5.93	5.93	5.93	5.93	5.93	4.45	4.45
w_{min}, in.	3/8	1/2	1/2	3/8	3/8	3/8	3/8	3/8	3/8	3/8	3/8	5/16	5/16
Size increase	1/8	3/16	3/16	1/8	1/8	1/8	3/16	1/8	1/8	1/8	1/8	1/8	1/8
% vol. increase	125%	156%	156%	125%	125%	125%	300%	125%	125%	125%	125%	178%	178%

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SCBF – Gusset Plate Welds (F2.6c.4)

$$D_{min} = \frac{f_u}{1.392\left(1.0 + 0.5\sin^{1.5}\theta\right)}$$

$$\theta = \tan^{-1}\left(\frac{f_{up} + f_{um_x} + f_{um_y}}{f_{uv}}\right)$$

$$f_u = \sqrt{f_{uv}^2 + \left(f_{up} + f_{um_x} + f_{um_y}\right)^2}$$

where

$f_{uv} = \dfrac{V_u}{2L}$, kips/in.

$f_{up} = \dfrac{P_u}{2L}$, kips/in.

$f_{um_x} = \dfrac{2M_{ux}}{L^2}$, kips/in.

$f_{um_y} = \dfrac{M_{uy\;max}}{\left(t_p + 0.5w\right)L}$, kips/in.

w = weld size, in.

Alternative $0.6R_yF.0_yt_p/\alpha$ Comparison													
D_{min}, 16ths	5.93	7.41	7.41	5.93	5.93	5.93	5.93	5.93	5.93	5.93	5.93	4.45	4.45
w_{min}, in.	3/8	1/2	1/2	3/8	3/8	3/8	3/8	3/8	3/8	3/8	3/8	5/16	5/16
Size increase	1/8	3/16	3/16	1/8	1/8	1/8	3/16	1/8	1/8	1/8	1/8	1/8	1/8
% vol. increase	125%	156%	156%	125%	125%	125%	300%	125%	125%	125%	125%	178%	178%

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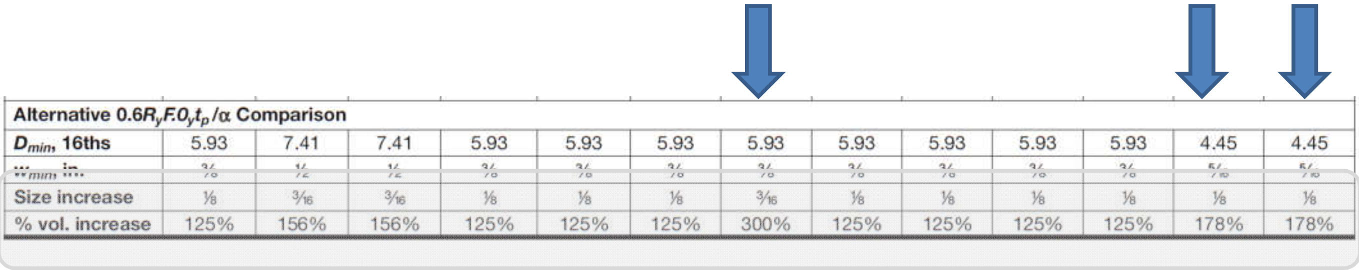
SCBF – Gusset Plate Welds (F2.6c.4)

The alternative method is a bit more complicated relative to $0.6R_yF_yt_p/\alpha_s$, but can be far more economical.



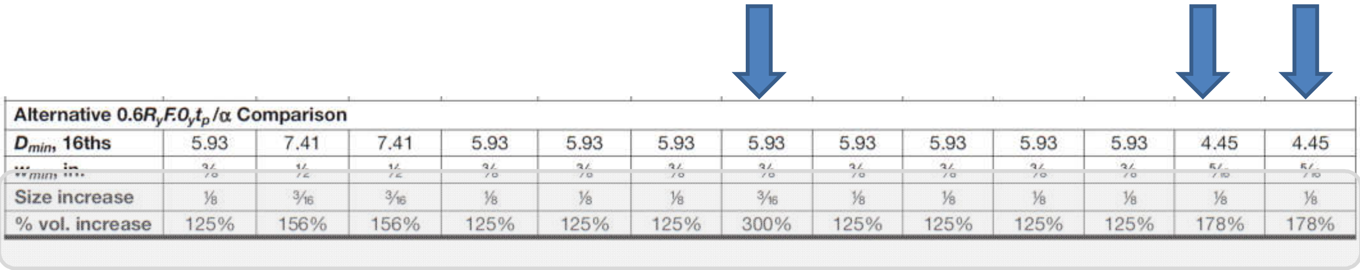
Alternative $0.6R_yF.0_yt_p/\alpha$ Comparison													
D_{min}, 16ths	5.93	7.41	7.41	5.93	5.93	5.93	5.93	5.93	5.93	5.93	5.93	4.45	4.45
w_{min}, in.	⅜	½	½	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	5⁄16	5⁄16
Size increase	⅛	3⁄16	3⁄16	⅛	⅛	⅛	3⁄16	⅛	⅛	⅛	⅛	⅛	⅛
% vol. increase	125%	156%	156%	125%	125%	125%	300%	125%	125%	125%	125%	178%	178%

Carter, Charles J.; Muir, Larry S.; Dowswell, Bo (2016). "Establishing and Developing the Weak-Axis Strength of Plates Subjected to Applied Loads," *Engineering Journal*, American Institute of Steel Construction, Vol. 53, pp. 147-157.

SCBF – Gusset Plate Welds (F2.6c.4)

The alternative method is a bit more complicated relative to $0.6R_yF_yt_p/\alpha_s$, but can be far more economical.

Regardless of the method used, and in general, when welds develop strength, the 1.25 weld ductility requirement should not be imposed on this weld design



Alternative $0.6R_yF.0_yt_p/\alpha$ Comparison													
D_{min}, 16ths	5.93	7.41	7.41	5.93	5.93	5.93	5.93	5.93	5.93	5.93	5.93	4.45	4.45
w_{min}, in.	⅜	½	½	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	5⁄16	5⁄16
Size increase	⅛	3⁄16	3⁄16	⅛	⅛	⅛	3⁄16	⅛	⅛	⅛	⅛	⅛	⅛
% vol. increase	125%	156%	156%	125%	125%	125%	300%	125%	125%	125%	125%	178%	178%

Carter, Charles J.; Muir, Larry S.; Dowswell, Bo (2016). "Establishing and Developing the Weak-Axis Strength of Plates Subjected to Applied Loads," *Engineering Journal*, American Institute of Steel Construction, Vol. 53, pp. 147-157.

Prequalified Moment Connections – AISC 358-16

ANSI/AISC 358-16
An American National Standard

Prequalified Connections for
Special and Intermediate
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May 12, 2016

Supersedes ANSI/AISC 358-10, ANSI/AISC 358s1-11, ANSI/AISC 358s2-14
and all previous versions

Approved by the Connection Prequalification Review Panel

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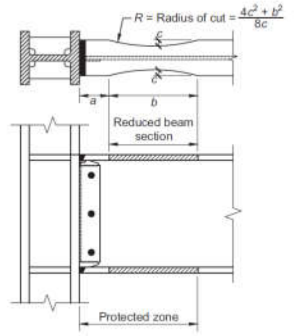
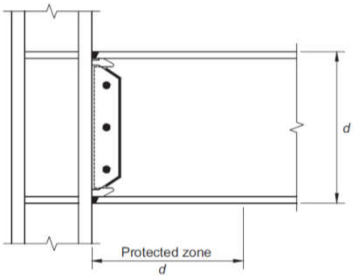
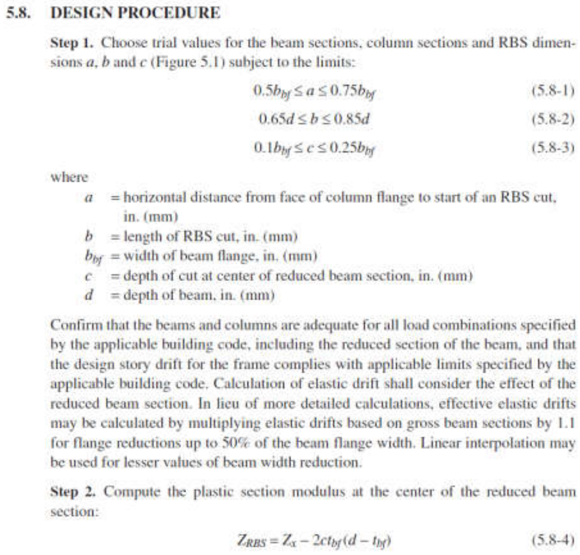




General Layout

In general, the analysis/design procedures for each type of connection are provided in a step-by-step format. A Design Procedure section is included in each chapter



5.8. DESIGN PROCEDURE

Step 1. Choose trial values for the beam sections, column sections and RBS dimensions *a*, *b* and *c* (Figure 5.1) subject to the limits:

$$0.5b_{bf} \le a \le 0.75b_{bf} \quad (5.8\text{-}1)$$

$$0.65d \le b \le 0.85d \quad (5.8\text{-}2)$$

$$0.1b_{bf} \le c \le 0.25b_{bf} \quad (5.8\text{-}3)$$

where

a = horizontal distance from face of column flange to start of an RBS cut, in. (mm)
b = length of RBS cut, in. (mm)
b_{bf} = width of beam flange, in. (mm)
c = depth of cut at center of reduced beam section, in. (mm)
d = depth of beam, in. (mm)

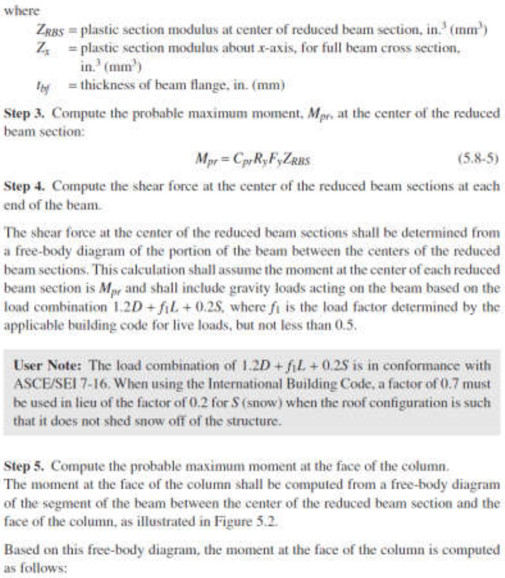
Confirm that the beams and columns are adequate for all load combinations specified by the applicable building code, including the reduced section of the beam, and that the design story drift for the frame complies with applicable limits specified by the applicable building code. Calculation of elastic drift shall consider the effect of the reduced beam section. In lieu of more detailed calculations, effective elastic drifts may be calculated by multiplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50% of the beam flange width. Linear interpolation may be used for lesser values of beam width reduction.

Step 2. Compute the plastic section modulus at the center of the reduced beam section:

$$Z_{RBS} = Z_x - 2ct_{bf}(d - t_{bf}) \quad (5.8\text{-}4)$$

General Layout

In general, the analysis/design procedures for each type of connection are provided in a step-by-step format. A Design Procedure section is included in each chapter



where

Z_{RBS} = plastic section modulus at center of reduced beam section, in.^{3} (mm^3)
Z_x = plastic section modulus about x-axis, for full beam cross section, in.^{3} (mm^3)
t_{bf} = thickness of beam flange, in. (mm)

Step 3. Compute the probable maximum moment, M_{pr}, at the center of the reduced beam section:

$$M_{pr} = C_{pr}R_yF_yZ_{RBS} \tag{5.8-5}$$

Step 4. Compute the shear force at the center of the reduced beam sections at each end of the beam.

The shear force at the center of the reduced beam sections shall be determined from a free-body diagram of the portion of the beam between the centers of the reduced beam sections. This calculation shall assume the moment at the center of each reduced beam section is M_{pr} and shall include gravity loads acting on the beam based on the load combination $1.2D + f_1L + 0.2S$, where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5.

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Step 5. Compute the probable maximum moment at the face of the column.
The moment at the face of the column shall be computed from a free-body diagram of the segment of the beam between the center of the reduced beam section and the face of the column, as illustrated in Figure 5.2.

Based on this free-body diagram, the moment at the face of the column is computed as follows:

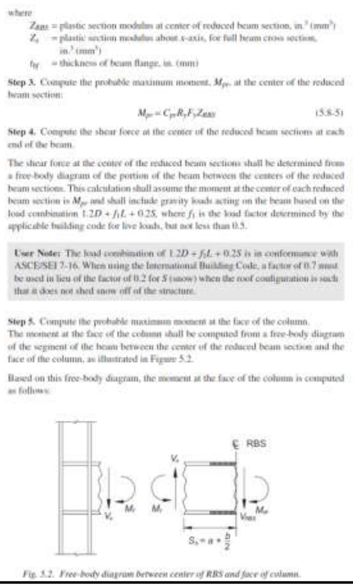


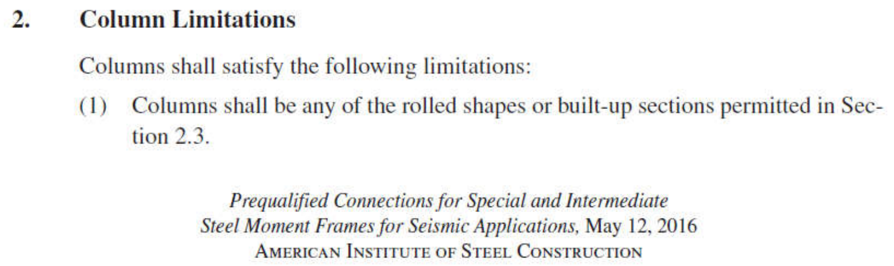
Fig. 5.2. Free-body diagram between center of RBS and face of column.

Limitations

- Columns
- Beams
- Connections
- Geometry

- Varies with connection type. Be sure to review and understand these limitations

Limitations (e.g., AISC 358-16 Section 7.5 – BFP)



2. Column Limitations

Columns shall satisfy the following limitations:

(1) Columns shall be any of the rolled shapes or built-up sections permitted in Section 2.3.

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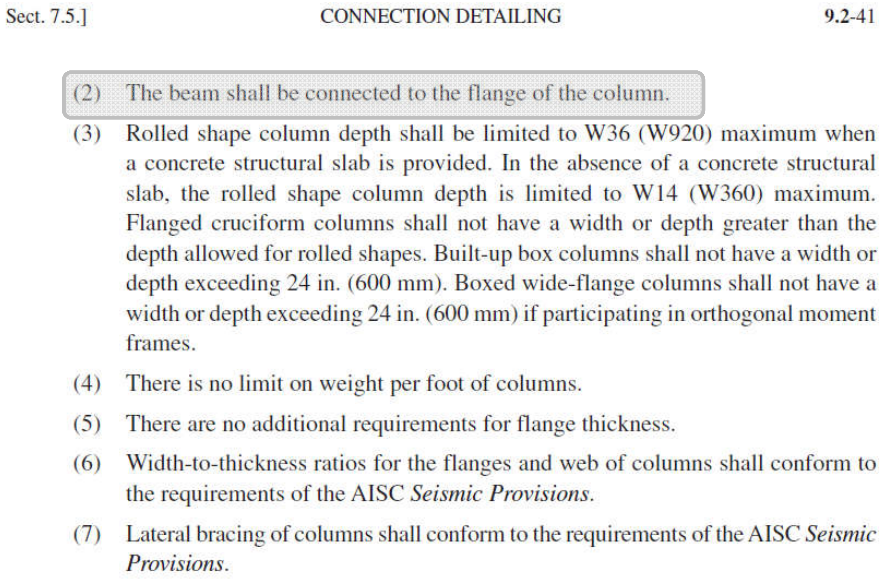
Limitations (e.g., AISC 358-16 Section 7.5 – BFP)



Sect. 7.5.] CONNECTION DETAILING 9.2-41

(2) The beam shall be connected to the flange of the column.

(3) Rolled shape column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, the rolled shape column depth is limited to W14 (W360) maximum. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 24 in. (600 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (600 mm) if participating in orthogonal moment frames.

(4) There is no limit on weight per foot of columns.

(5) There are no additional requirements for flange thickness.

(6) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions.*

(7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions.*

IMF/SMF – General Comments

- AISC 358 contains several prequalified connections
  - Some of them are proprietary
- The choice of connection is generally made by the SEoR. However, some issues should be considered
  - Shop versus field welding
    - Shop welding is generally easier
    - Weld inspection and potential rework is easier in the shop
    - Field welding should be avoided in areas with stringent fire-watch areas (can be extremely expensive)
    - Relative to required hardware and shop hours, field welded connections can be less expensive (depending on cost of field welding)

IMF/SMF – Beam FBD - BFP

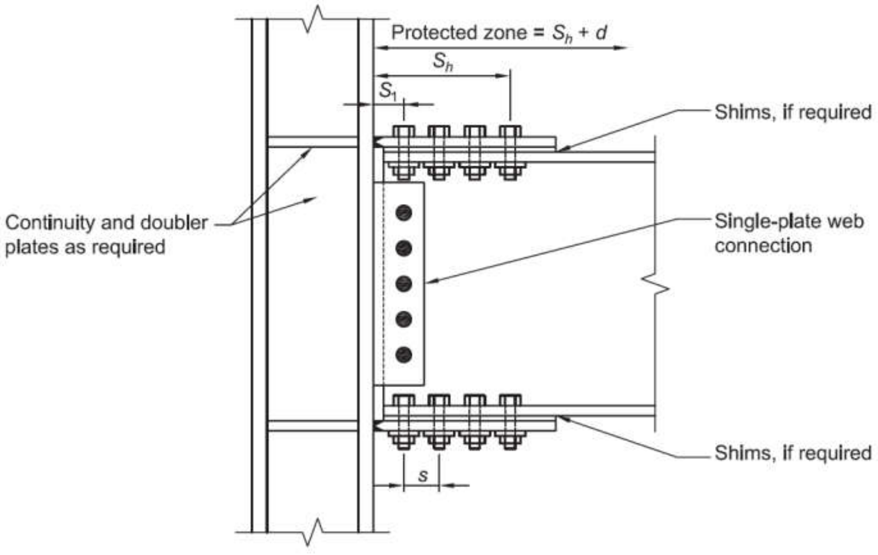
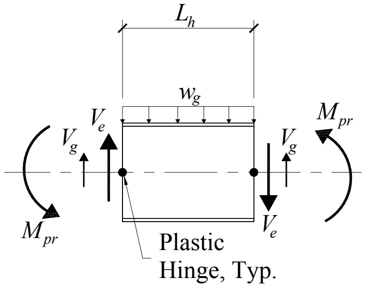


Fig. 7.1. Bolted flange plate moment connection.

IMF/SMF – Beam FBD - BFP

Based on the beam's probable maximum moment, M_{pr} …



$$M_{pr} = C_{pr} R_y F_y Z_e \qquad \text{(EQ. 2.4-1)}$$

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \qquad \text{(EQ. 2.4-2)}$$

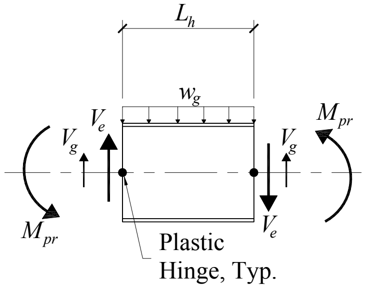
SMF – Beam FBD - BFP

Based on the beam's probable maximum moment, M_{pr} …

Note that for A992-50, $C_{pr}R_yF_y$ is…

$$M_{pr} = \left(\frac{50\text{ ksi} + 65\text{ ksi}}{(2)(50\text{ ksi})}\right)(1.1)(50\text{ ksi})Z_e$$

$$M_{pr} = 63.3Z_e$$



$$M_{pr} = C_{pr} R_y F_y Z_e \qquad \text{(EQ. 2.4-1)}$$

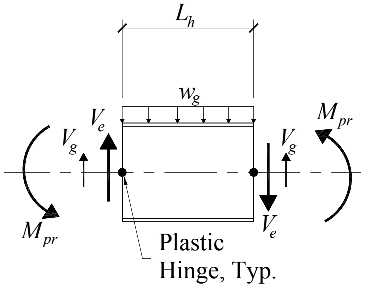
$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \qquad \text{(EQ. 2.4-2)}$$

SMF – Beam FBD - BFP

Based on the beam's probable maximum moment, M_{pr} …

Shear at the plastic hinge is…

$$V_h = \frac{2M_{pr}}{L_h} + \frac{w_g L_h}{2}$$



$$M_{pr} = C_{pr} R_y F_y Z_e \qquad \text{(EQ. 2.4-1)}$$

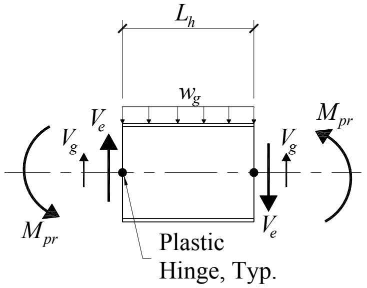
$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \qquad \text{(EQ. 2.4-2)}$$

SMF – Beam FBD - BFP

Based on the beam's probable maximum moment, M_{pr} …

Shear at the plastic hinge is…

$$V_h = \frac{2M_{pr}}{L_h} + \frac{w_g L_h}{2}$$



$$V_h = V_e + V_g$$

$$V_h = \frac{2M_{pr}}{L_h} + \frac{w_g L_h}{2}$$

$$M_{pr} = C_{pr} R_y F_y Z_e \qquad \text{(EQ. 2.4-1)}$$

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \qquad \text{(EQ. 2.4-2)}$$

$$V_g = \frac{w_g L_h}{2}$$

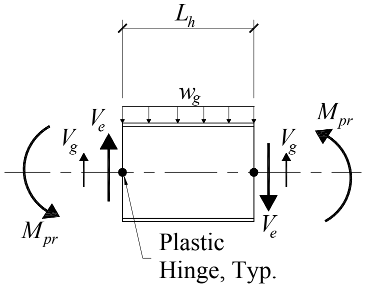
SMF – Beam FBD - BFP

Based on the beam's probable maximum moment, M_{pr} …

Shear at the plastic hinge is…

$$V_{h.\max} = \frac{2M_{pr}}{L_h} + \frac{w_g L_h}{2}$$

$$V_{h.\min} = \frac{2M_{pr}}{L_h} - \frac{w_g L_h}{2}$$



$$V_h = V_e + V_g$$

$$M_{pr} = C_{pr} R_y F_y Z_e \qquad \text{(EQ. 2.4-1)}$$

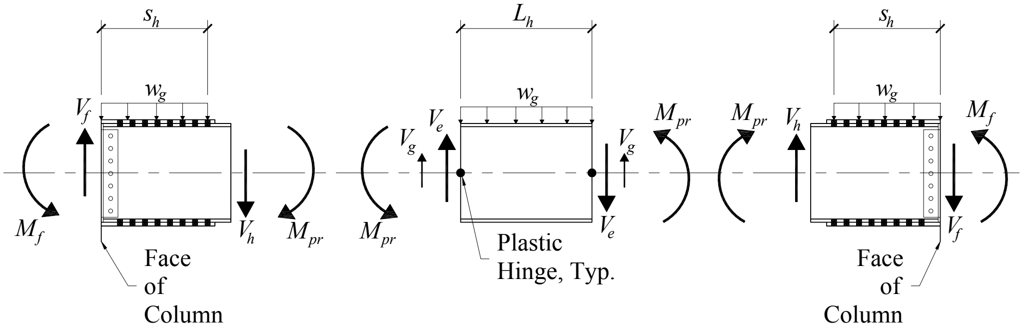
$$V_g = \frac{w_g L_h}{2}$$

$$V_h = \frac{2M_{pr}}{L_h} + \frac{w_g L_h}{2}$$

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \qquad \text{(EQ. 2.4-2)}$$

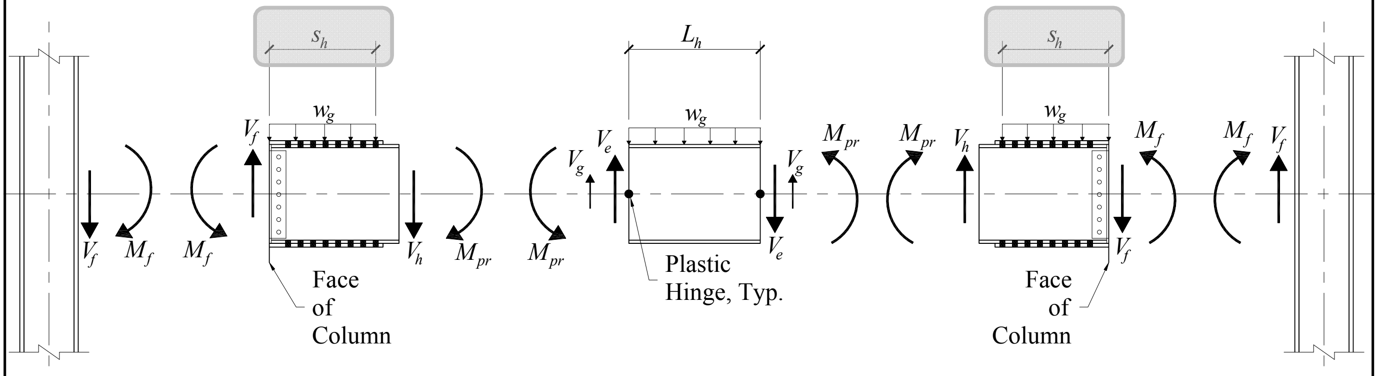
SMF – Beam FBD - BFP

Based on the beam's probable maximum moment, M_{pr} …



SMF – Beam FBD - BFP

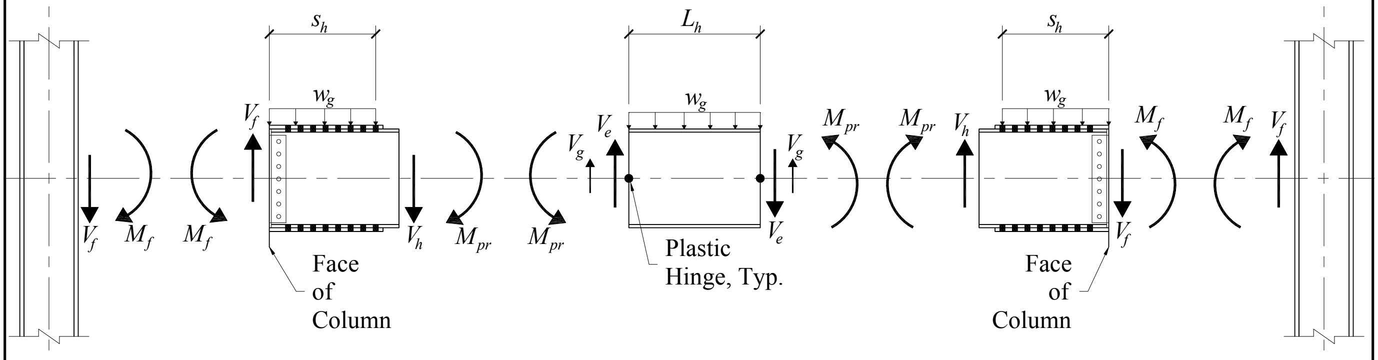
Based on the beam's probable maximum moment, M_{pr} …



Note that the extent of s_h varies with the type of prequalified connection used. Refer to AISC 358.

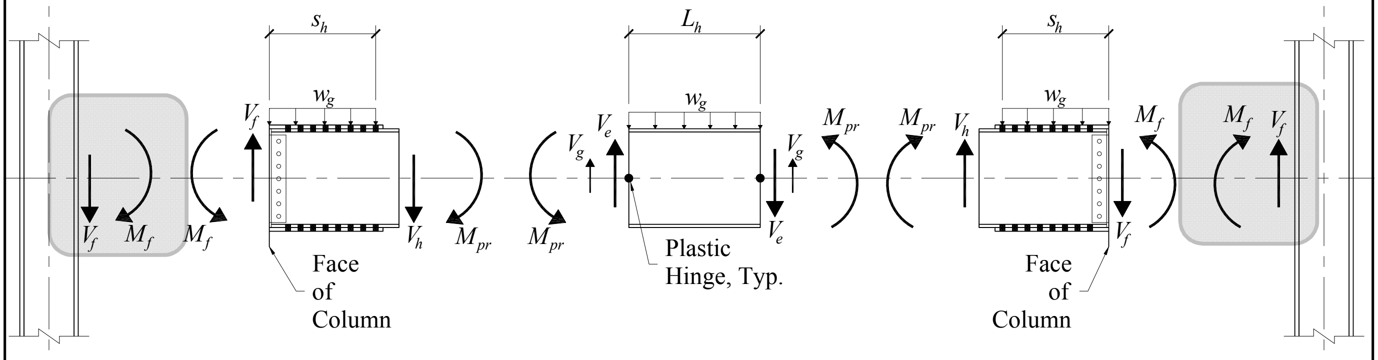
SMF – Beam FBD - BFP

Based on the beam's probable maximum moment, M_{pr} …



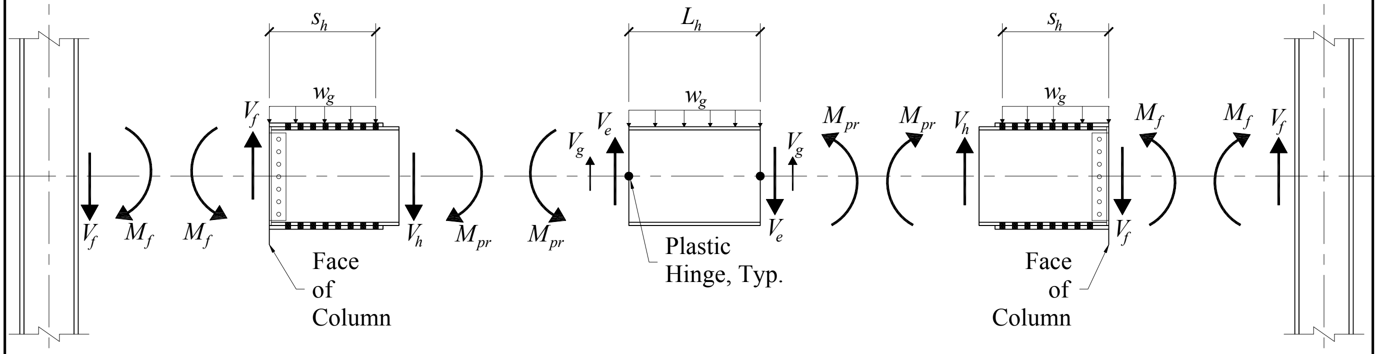
SMF – Beam FBD - BFP

Based on the beam's probable maximum moment, M_{pr} …



Required moment and shear for connection design is <u>maximum</u> M_f. And V_f.

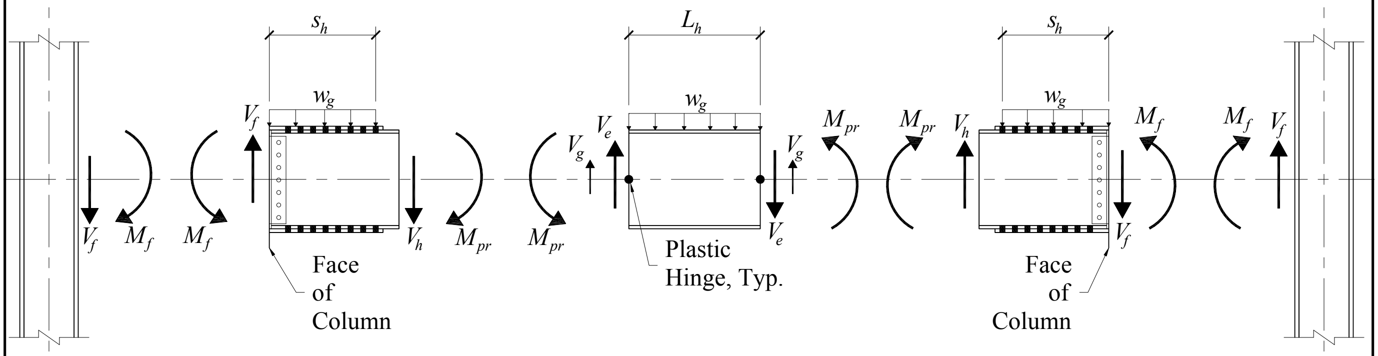
SMF – Beam FBD - BFP



$$M_f = M_{pr} + V_h s_s + \frac{w_g s_h^2}{2}$$

$$V_f = V_h + w_g s_h$$

SMF – Beam FBD - BFP



$$M_f = M_{pr} + V_h s_s + \frac{w_g s_h^2}{2}$$

$$V_f = V_h + w_g s_h$$

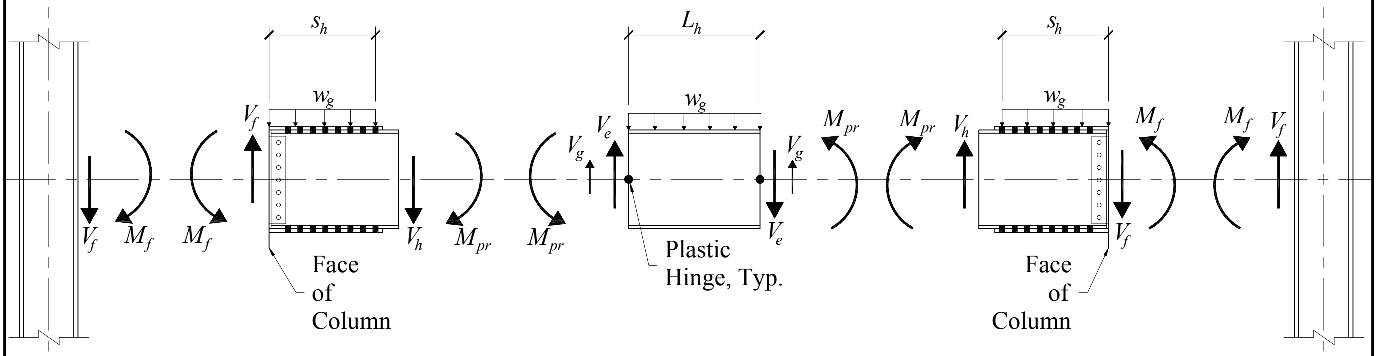
Step 7. Calculate the moment expected at the face of the column flange.

$$M_f = M_{pr} + V_h S_h \qquad (7.6\text{-}6)$$

where V_h is the larger of the two values of shear force at the beam hinge location at each end of the beam, kips (N).

Equation 7.6-6 neglects the gravity load on the portion of the beam between the plastic hinge and the face of the column. The gravity load on this small portion of the beam is permitted to be included.

SMF – Beam FBD - BFP



$$M_f = M_{pr} + V_h s_s + \cancel{\frac{w_g s_h^2}{2}}$$

$$V_f = V_h + w_g s_h$$

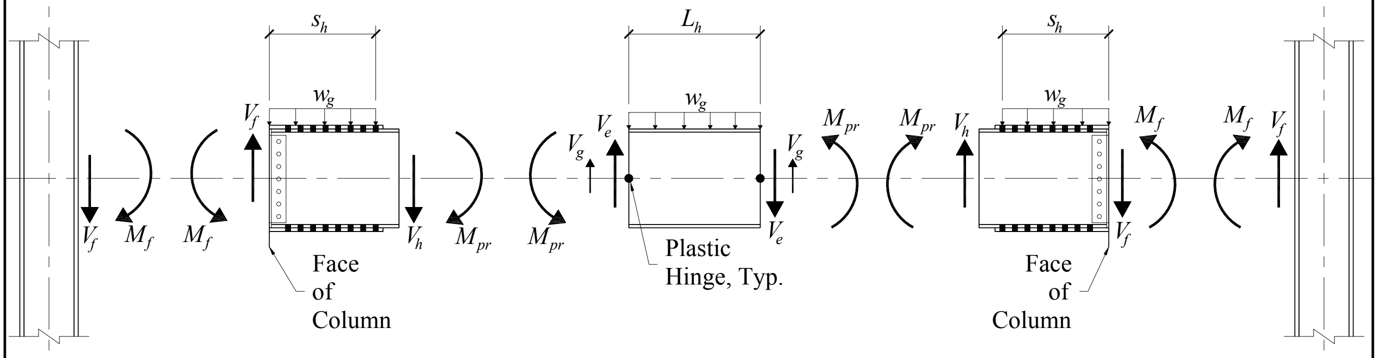
Step 7. Calculate the moment expected at the face of the column flange.

$$M_f = M_{pr} + V_h S_h \qquad (7.6\text{-}6)$$

The gravity load on this portion of the beam increases V_f and M_f.

ASIC 358 permits this portion of the gravity load to be neglected.

SMF – Beam FBD - BFP



$$M_f = M_{pr} + V_h s_s + \cancel{\frac{w_g s_h^2}{2}}$$

$$V_f = V_h + w_g s_h$$

Step 7. Calculate the moment expected at the face of the column flange.

$$M_f = M_{pr} + V_h S_h \qquad (7.6\text{-}6)$$

It is crucial that that the gravity loads be provided to the connection designer.

UDL loads make the limitations of the prequalified connections virtually impossible to satisfy!

SMF – Beam FBD - RBS

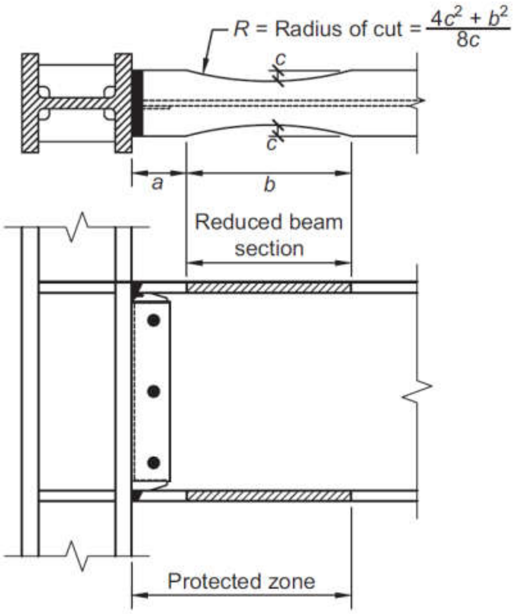
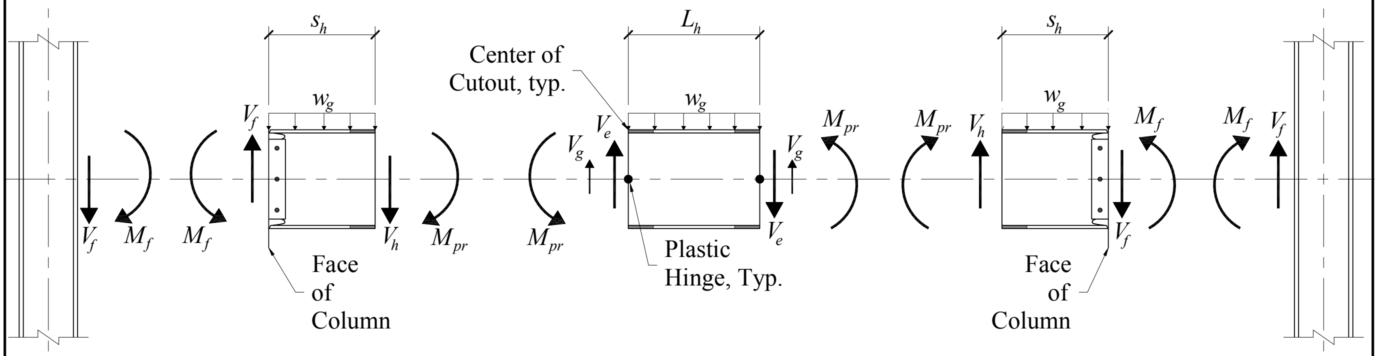


Fig. 5.1. Reduced beam section connection.

SMF – Beam FBD - RBS



$$M_f = M_{pr} + V_h s_s + \xcancel{\frac{w_g s_h^2}{2}}$$

$$V_f = V_h + w_g s_h$$

Likewise, the location of the plastic hinge varies with the type of connection.

Refer to AISC 358 for specific details.

SMF – Beam FBD - WUF-W

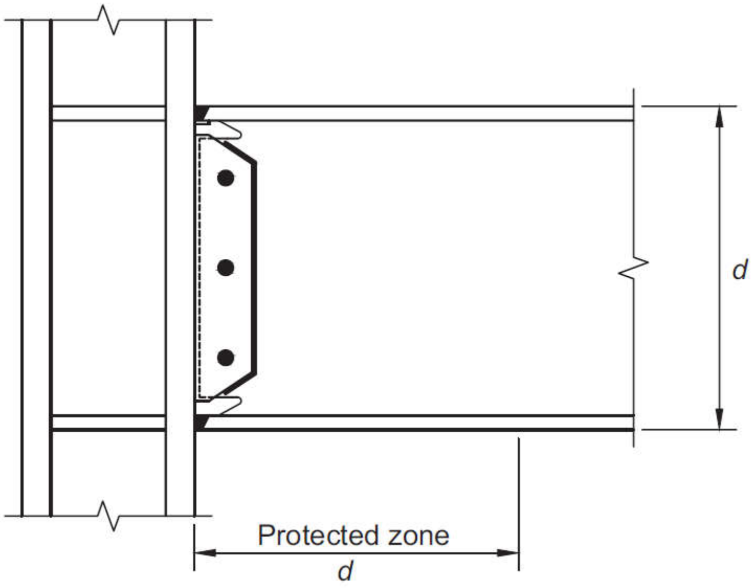
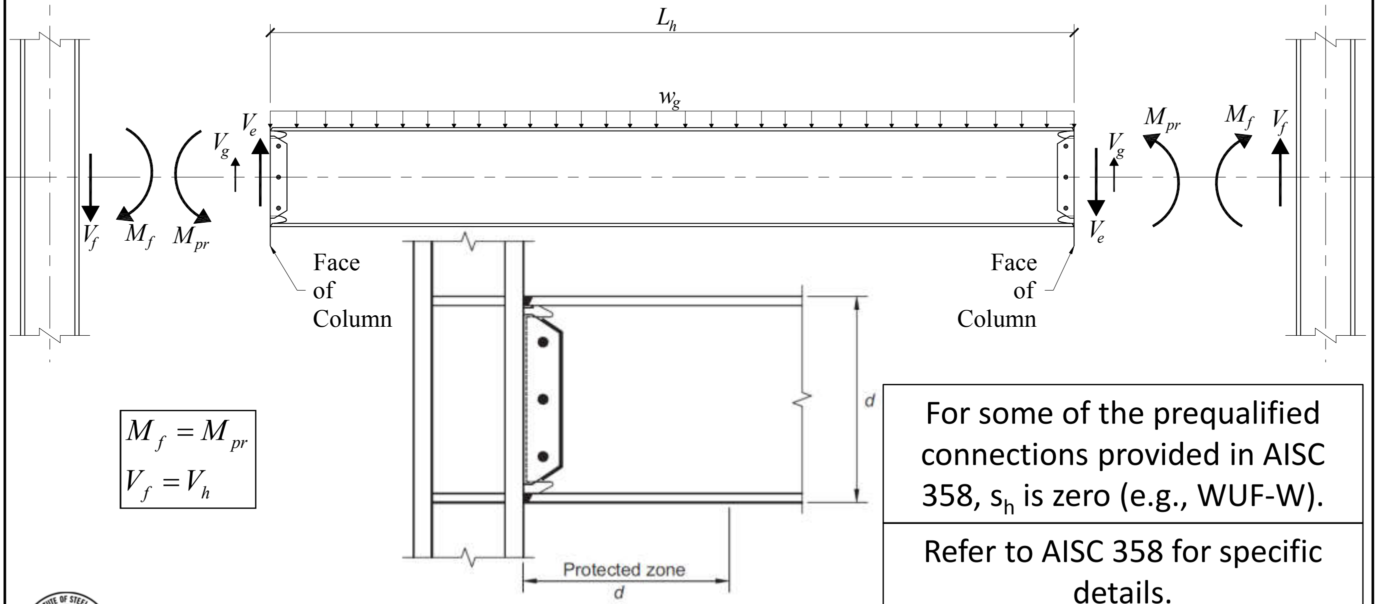


Fig. 8.1. WUF-W moment connection.

SMF – Beam FBD - WUF-W



$$M_f = M_{pr}$$
$$V_f = V_h$$

For some of the prequalified connections provided in AISC 358, s_h is zero (e.g., WUF-W).

Refer to AISC 358 for specific details.

Fig. 8.1. WUF-W moment connection.

Prequalified Moment Connections – AISC 358-16

ANSI/AISC 358-16
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and all previous versions

Approved by the Connection Prequalification Review Panel

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Seismic Connections

FYI:

Mike Englehardt gave a very informative presentation on

“AISC 358 Prequalified Connections”

NASCC 2012.

Visit www.aisc.org/educationarchives

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- REINFORCEMENT – Reinforce what you learn tonight. Get more out of the course.

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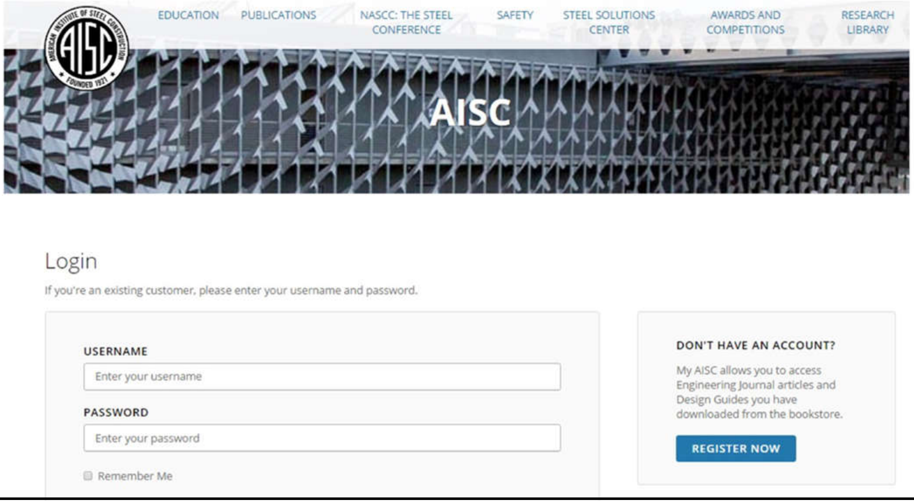

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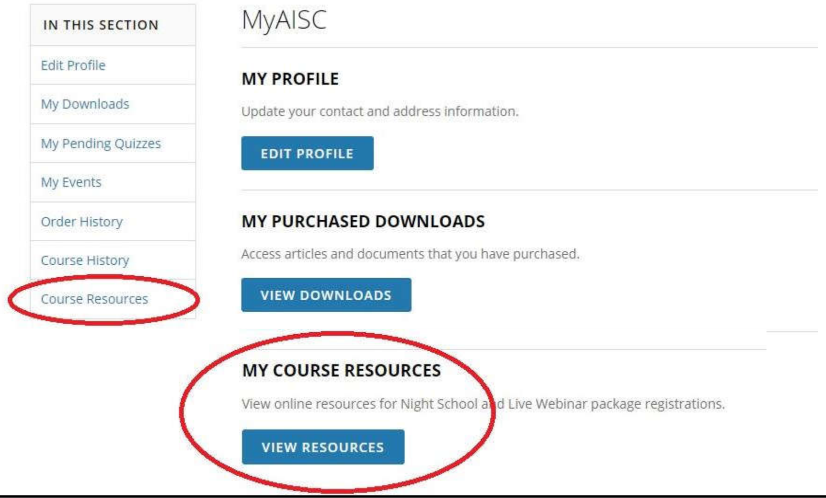

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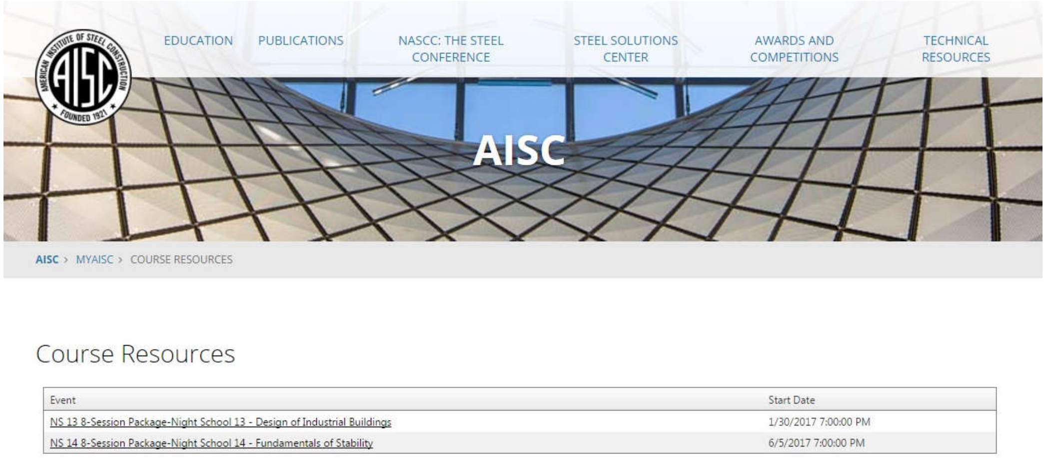


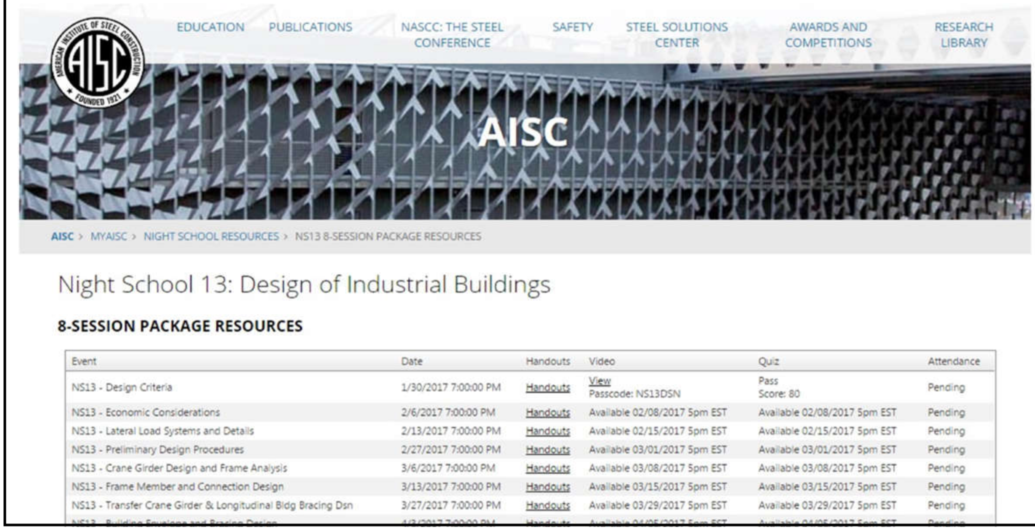
Course Resources

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

Smarter.
Stronger.
Steel.

8-Session Registrants

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 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending

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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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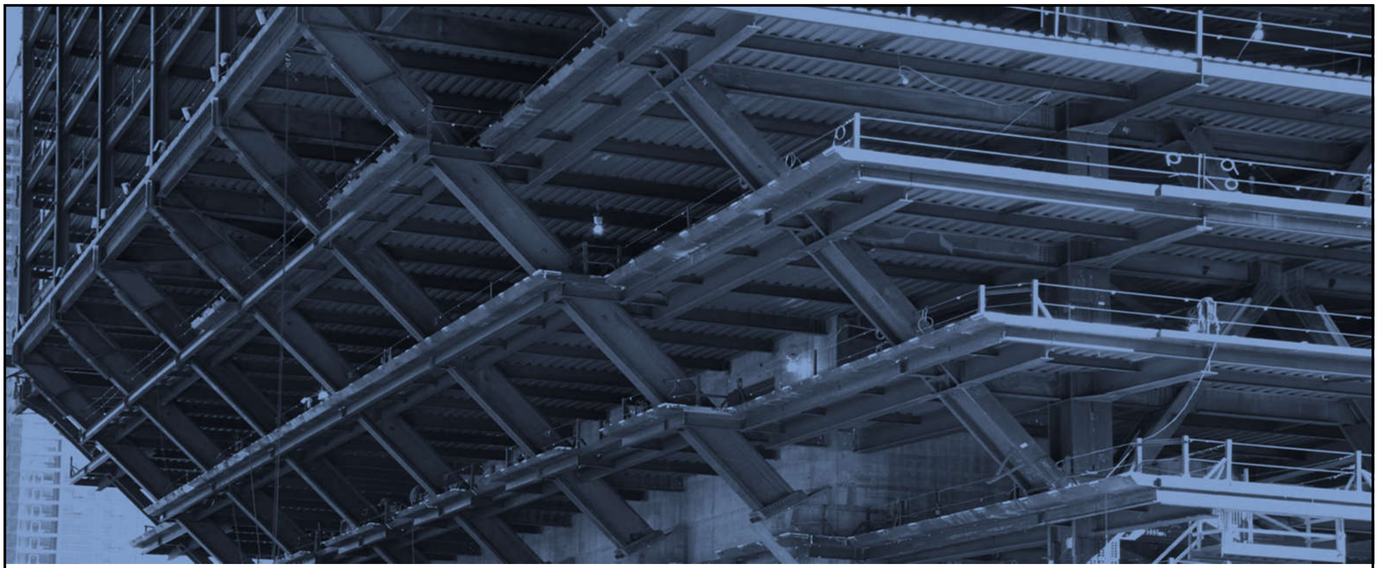
Steel.



AISC | Thank you

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Stronger.
Steel.