CONXTECH®

Simply Faster

Engineering Standard

Design of Connections

CX-ENG-STD-000001

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VERSION	**DATE**	**AUTHOR**	**CHECKED BY**

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1 General

1.1 Scope

The following document presents the design standards for the ConX connections that are not designed using standard specifications. The document contains design procedures and load path verification for all three of the ConX moment frame connection systems, in addition to beam to beam shear connections, beam to column shear connections, column splice connections, and column base connections. The calculations are presented for both LRFD and ASD available strength for each connection. In order to properly use this standard, an analysis of the steel framed structure should be performed in which all applicable loads and load combinations are included, and the beams and columns are modeled with the appropriate section properties, boundary conditions, and member connection type assumptions. The member end forces resulting from the envelope of design results taken from the analysis model of the frame should be used to verify that the available strength of each connection exceeds the required strength.

1.2 References

This design standard is based on the latest edition of the references below, unless otherwise noted.

1.2.1 ConXtech References

ConXtech Engineering References

CX-ENG-RFR-000001	*ConXtech System Reference Tables*

ConXtech Engineering Standards

CX-ENG-STD-000010	*Design of Beam to Column Connections*
CX-ENG-STD-000011	*Design of Beam to Beam Connections*

1.2.2 Industry Codes & Standards

California Building Code (CBC)

2019 CBC	*California Code of Regulations, Title 24.*

International Building Code (IBC)

2018 IBC	*International Building Code*

American Concrete Institute (ACI)

ACI 318/318R-14	*Building Code Requirements for Structural Concrete and Commentary*

American Institute of Steel Construction (AISC)

AISC	*Steel Construction Manual – 15th Edition*

AISC 360-16	*Specification for Structural Steel Buildings*
ANSI/AISC 341-16	*Seismic Provisions for Structural Steel Buildings*
AISC 358-16	*Pre-Qualified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, Including Supplement No. 1*
AISC DG1	*Design Guide 1: Base Plate and Anchor Rod Design (Second Edition)*
AISC DG4	*Design Guide 4: Extended End-Plate Moment Connections Seismic and Wind Applications (Second Edition*
AISC DG16	*Design Guide 16: Flush and Extended Multiple-Row Moment End-Plate Connections.*
AISC DG19	*Design Guide 19: Vertical Bracing Connections - Analysis and Design.*
RCSC	*Research Council on Structural Connections Specification for Structural Joints Using ASTM A325 or ASTM A490 Bolts*

<u>American Iron and Steel Institute (AISI)</u>

AISI S100-2016	*North American Specification for the Design of Cold-Formed Steel Structural Members*

<u>American Society of Civil Engineers (ASCE)</u>

ASCE/SEI 7-16	*Minimum Design Loads for Buildings and Other Structures*
ASCE - Report	*Guidelines for Seismic Evaluation and Design of Petrochemical Facilities*
ASCE - Report	*Wind Loads and Anchor Bolt Design for Petrochemical Facilities*

<u>American Society of Mechanical Engineers (ASME)</u>

ASME B31.3	*Process Piping*

<u>The American Welding Society (AWS)</u>

AWS D1.1/ D1.1M-2015	*Structural Welding Code – Steel*
AWS D1.8/D1.8M-2016	*Structural Welding Code – Seismic Supplement*

<u>Steel Deck Institute (SDI)</u>

Design Manual for Composite Decks, Form Decks and Roof Decks

<u>Steel Joist Institute (SJI)</u>

Standard Specifications and Load Tables for Steel Joists and Joist Girders

<u>Federal Standards and Instructions of the Occupational Safety and Health Administration (OSHA 1910)</u>

US Department of Labor, Occupational Safety and Health Administration (OSHA)

OSHA 29 CFR 1910	*Occupational Safety and Health Standards*
OSHA 29 CFR 1926	*Safety and Health Regulations for Construction*

2 ConX Moment Connections

2.1 ConXL400

2.1.1 Moment Connection Description

The ConXtech ConXL-400 moment connection is a steel moment-resisting beam to column connection for resisting gravity, wind and seismic forces imposed on a building/non-building structure. As can be seen in Figure 1, the collar connection is designed to join hollow structural section columns (or built-up box columns) and wide flange steel beams at all four sides of the column, although the connection can be used for as few as one beam.

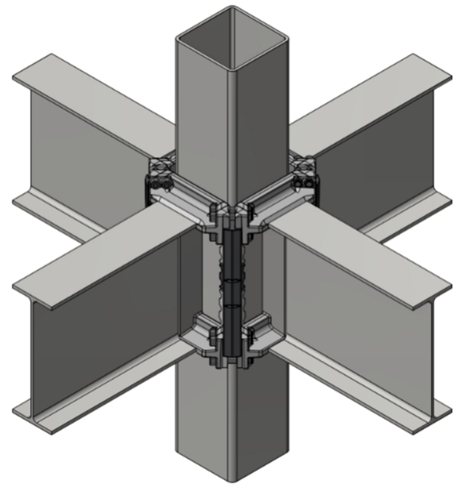


Figure 1: ConXL-400 Moment Connection Assembly.

Figure 2 shows an isolated view of a fully assembled moment collar. The moment collar is comprised of various components. The components that make up each face of the collar are joined together by welds, and the full collar assembly is held together by sixteen pretensioned high strength bolts (not shown in the figure)

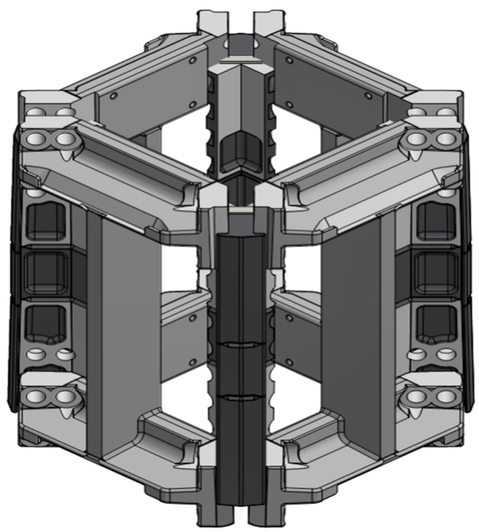


Figure 2: 3D-Perspective View of ConXL-400 Moment Collar Assembly

The ConXL-400 connection is only used on columns that are 16 *in*. (406.4 mm) square combined with wide flange beams of the sizes shown in Table 1. Note that the connection can only be used with moment beams from the same nominal depth family.

Table 1: List of Compatible Beam Sizes for the ConXL-400 System.

W18	W21	W24	W27	W30
W18X35	W21X44	W24X55	W27X84	W30X90
W18X40	W21X50	W24X62	W27X94	W30X99
W18X46	W21X57	W24X68	W27X102	W30X108
W18X50	W21X48	W24X76	W27X114	W30X116
W18X55	W21X55	W24X84		W30X124
W18X60	W21X62	W24X94		W30X132
W18X65	W21X68	W24X103		
W18X71	W21X73			
W18X76	W21X83			
W18X86	W21X93			
W18X97				
W18X106				

A Collar Flange Assembly (CFA) is welded to the end of a moment beam and the Collar Corner Assembly (CCA) is welded to the moment column. The ConXL-400 moment connection assembly is comprised of four CFAs at each face of the moment column and four CCAs at each corner of the square column.

The CCAs are drilled for through bolting and welded at the corners of the moment column at each floor level.

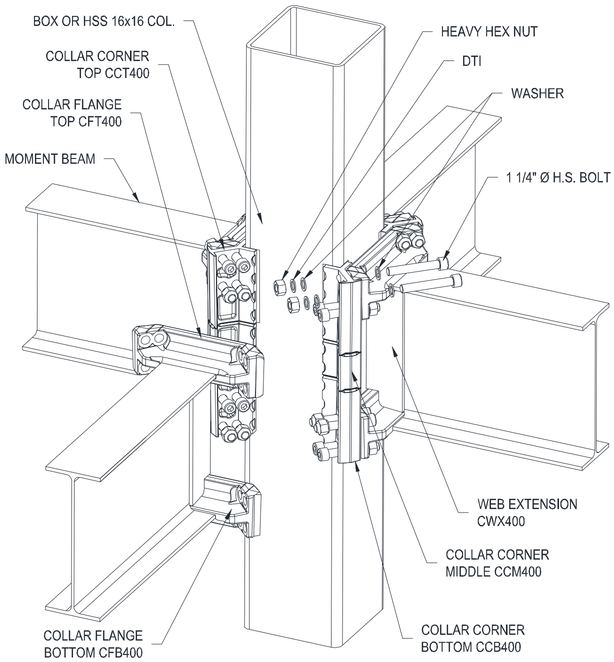


Figure 3: ConXL-400 Collar System.

The CFA consists of a top and bottom Collar Flange, each welded to the top and bottom beam flange, and a Collar Web Extension (CWX), which is welded to the beam web and is welded to and spans between the top and bottom Collar Flanges.

The CWX comes in a variety of depths, while the top and bottom Collar Flanges are standardized modular units. The CFA can be assembled for a variety of beam sizes by adjusting the depth of the CWX.

The CFA is also drilled along its outside edges to receive Collar Bolts. Assembly in the field is accomplished by lowering the beam-CFA onto the column-CCA on each face of the column.

The CCA has shear lugs at the bottom of the CCA (see Figure 3) that provide temporary vertical support for the beam and CFA until the bolts are pre-tensioned. Once the bolts are pretensioned, the friction due to the clamping force between the moment collar and the column provides vertical (upward or downward) carrying capacity for the beam.

Once beam-CFA assemblies have been located on each of the four faces of the column, high strength Collar Bolts are inserted into the holes in the adjoining CFAs and pretensioned, clamping the CFA to the CCA/column, forming a rigid collar assembly that surrounds the column.

The proceeding four pages illustrate the ConXL-400 connection in more detail. Standard details of the CFAs, CWX and CCAs are shown, as well as perspective images of beams fitted with the CFA.

2.1.2 Collar Flange Assembly

COLLAR FLANGE TOP (XL400-CFT)

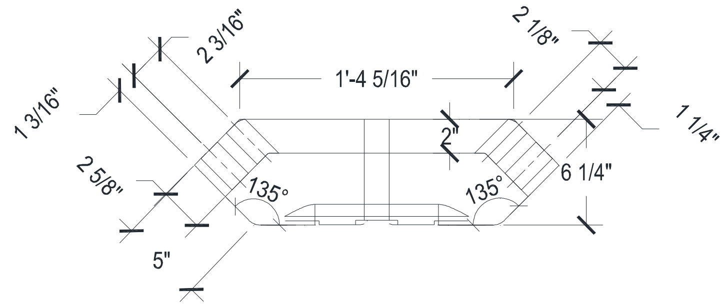
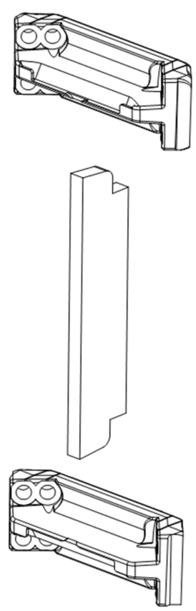


Figure 4: Top View of Collar Flange Top.

Figure 4 above shows the top view of a Collar Flange Top.

The figure on the right shows an exploded CFA. The figure on the far right (Figure 5) is a section cut of a fully assembled Collar Flange Assembly.

Figure 6 shows the top view of a Collar Flange Bottom.



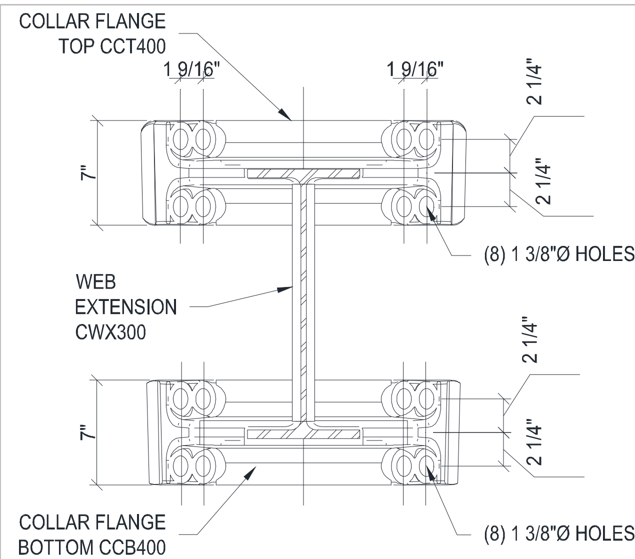


Figure 5: Collar Flange Assembly (XL400-CFA).

COLLAR FLANGE BOTTOM (XL400-CFB)

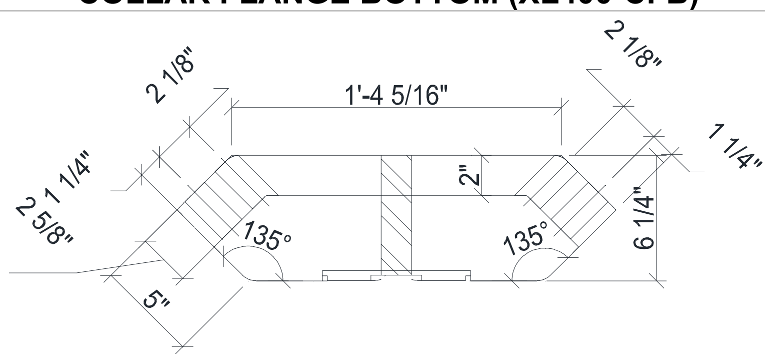


Figure 6: Top View of Collar Flange Bottom.

2.1.3 Beam-To-Collar Flange Assembly

The figures on this page illustrate the three types of beam configurations used in the ConXL-400 system.

The beam at the top is fitted with Collar Flanges on both ends. This beam configuration is referred to as a Beam-Moment-Moment (BMM).

The beam in the middle is fitted with a Collar Flange on one end. This beam is referred to as a Beam-Moment-Gravity (BMG) or a Beam-Gravity-Moment (BGM).

The beam on the bottom has no Collar Flanges on its ends and is referred to as a Beam-Gravity-Gravity (BGG).

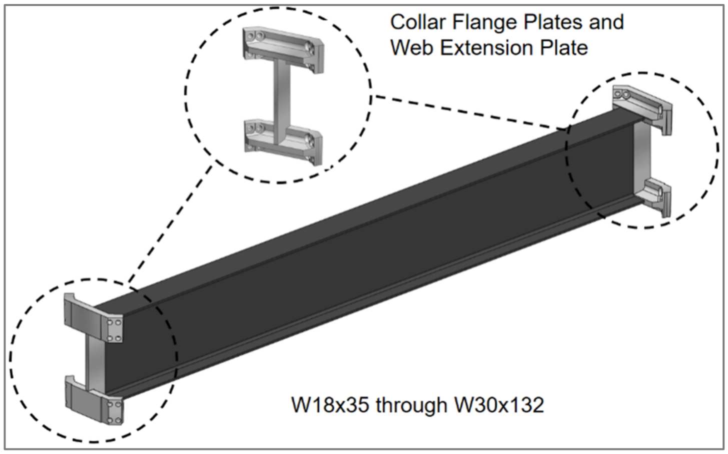


Figure 7: BMM.

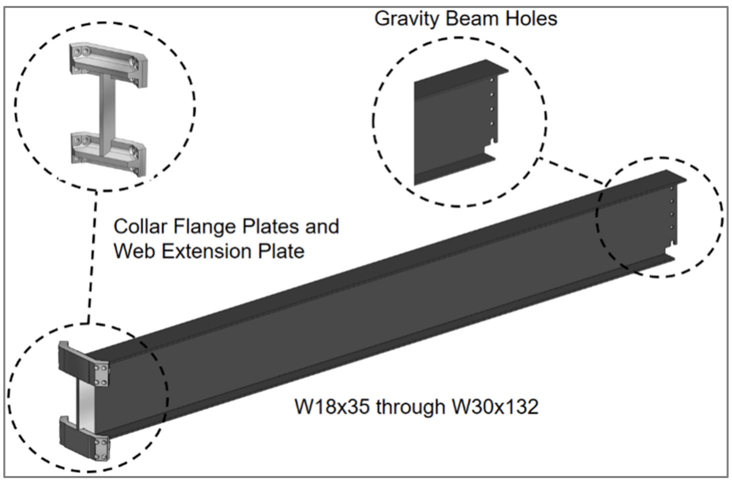


Figure 8: BMG or BGM.

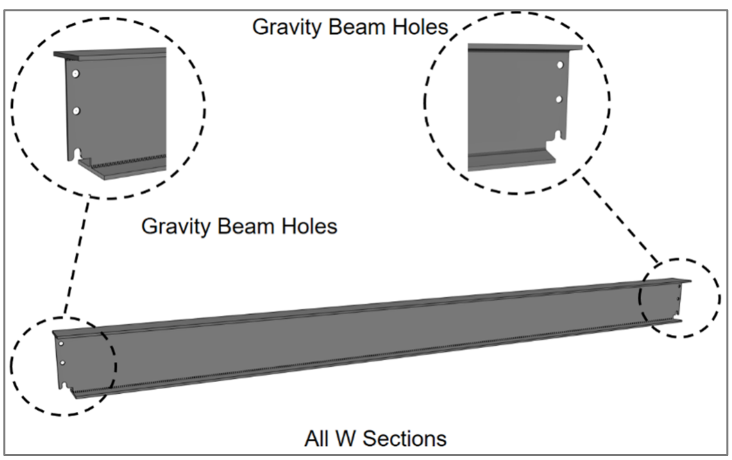


Figure 9: BGG.

2.1.4 Collar Corner Assembly

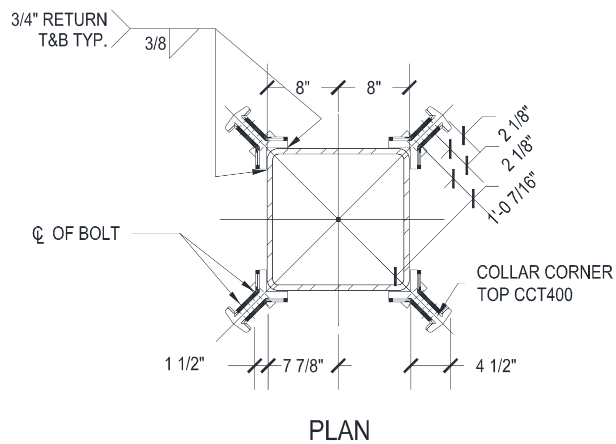


Figure 10: Plan View of ConXL-400 Collar Corners and Column.

The figures on this page show the plan, perspective and elevation views of the Collar Corner Assembly.

As can be seen, the Collar Corner Assemblies are attached to the moment column with fillet welds.

Note that the Collar Corner Middle is present only when the connected moment beam is deeper than a W18.

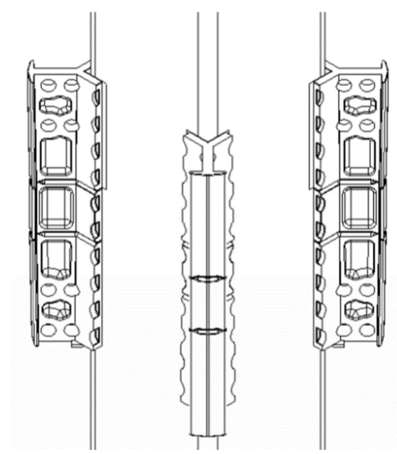


Figure 11: Perspective View of Collar Corner Assemblies welded onto a Column.

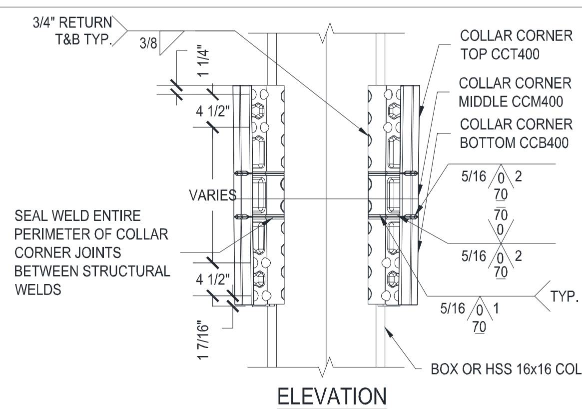


Figure 12: Standard Detail of Collar Corner Assemblies.

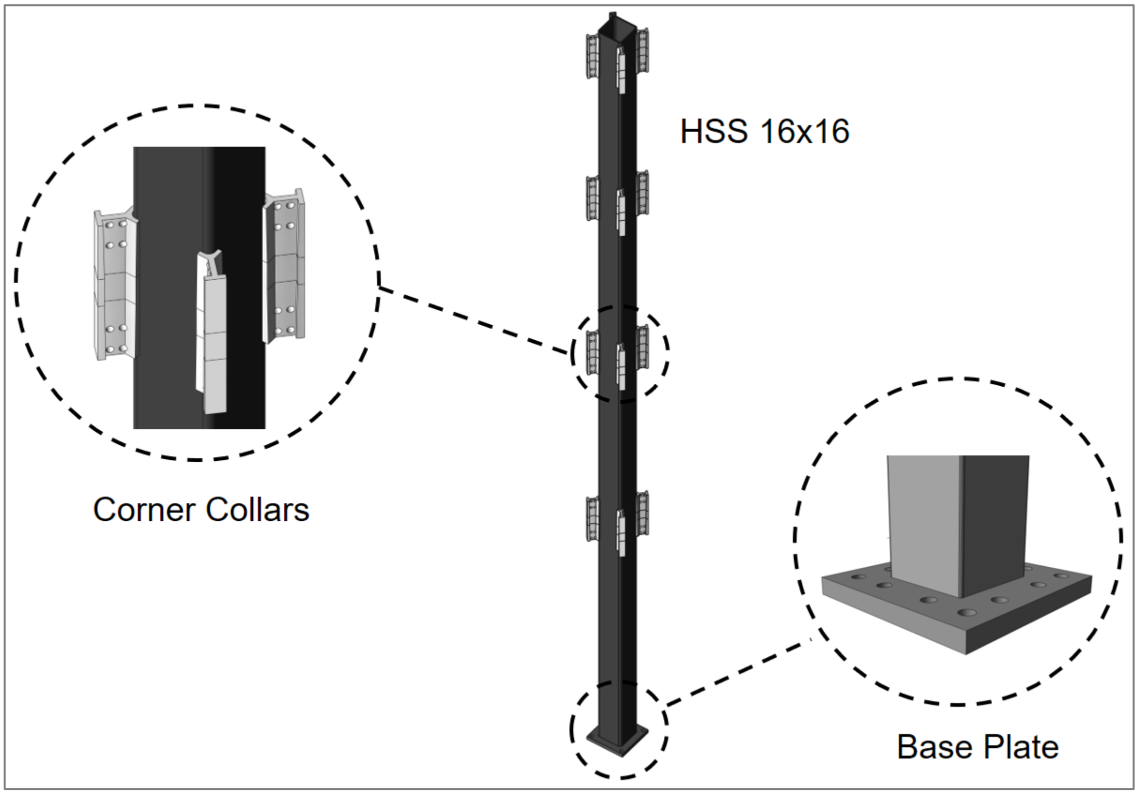


Figure 13: ConXL-400 Moment Column Assembly.

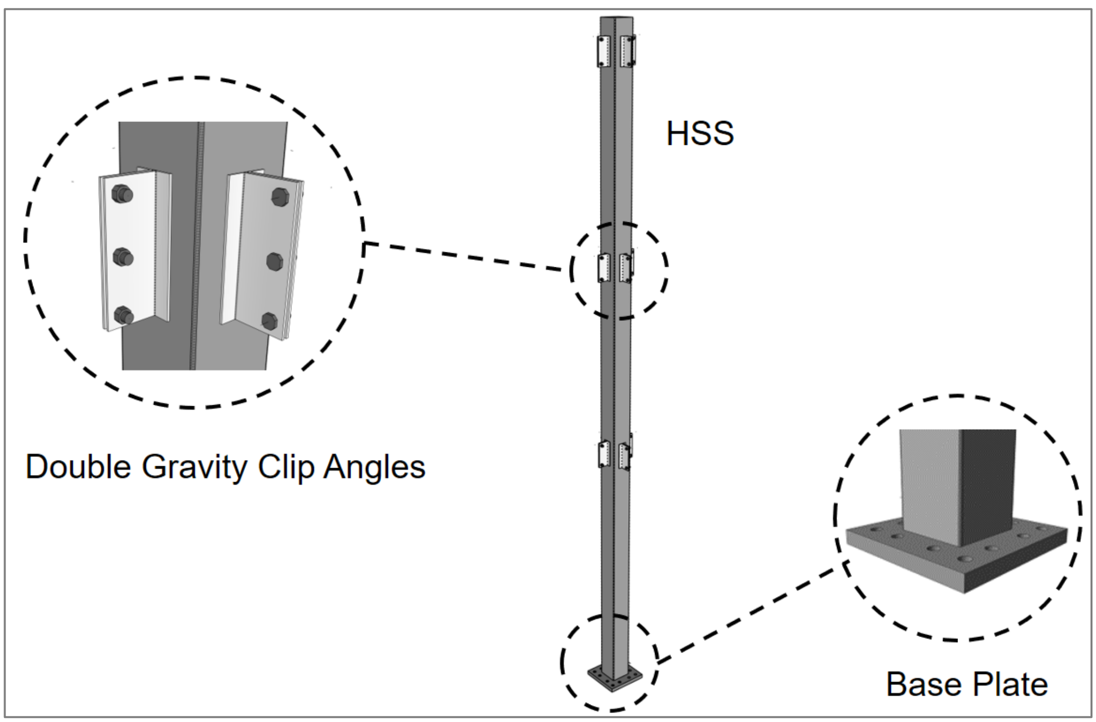


Figure 14: ConXL-400 Gravity Column Assembly.

2.1.5 Beam and Moment Collar Assembly

The figure below shows the plan and elevation standard details of the ConXL-400 moment connection. As can be seen in the figure, the Collar Corner Assemblies are attached to the 16 *in*. square column via fillet welds. The Collar Corner Top, Middle and Bottom can be seen in the elevation view.

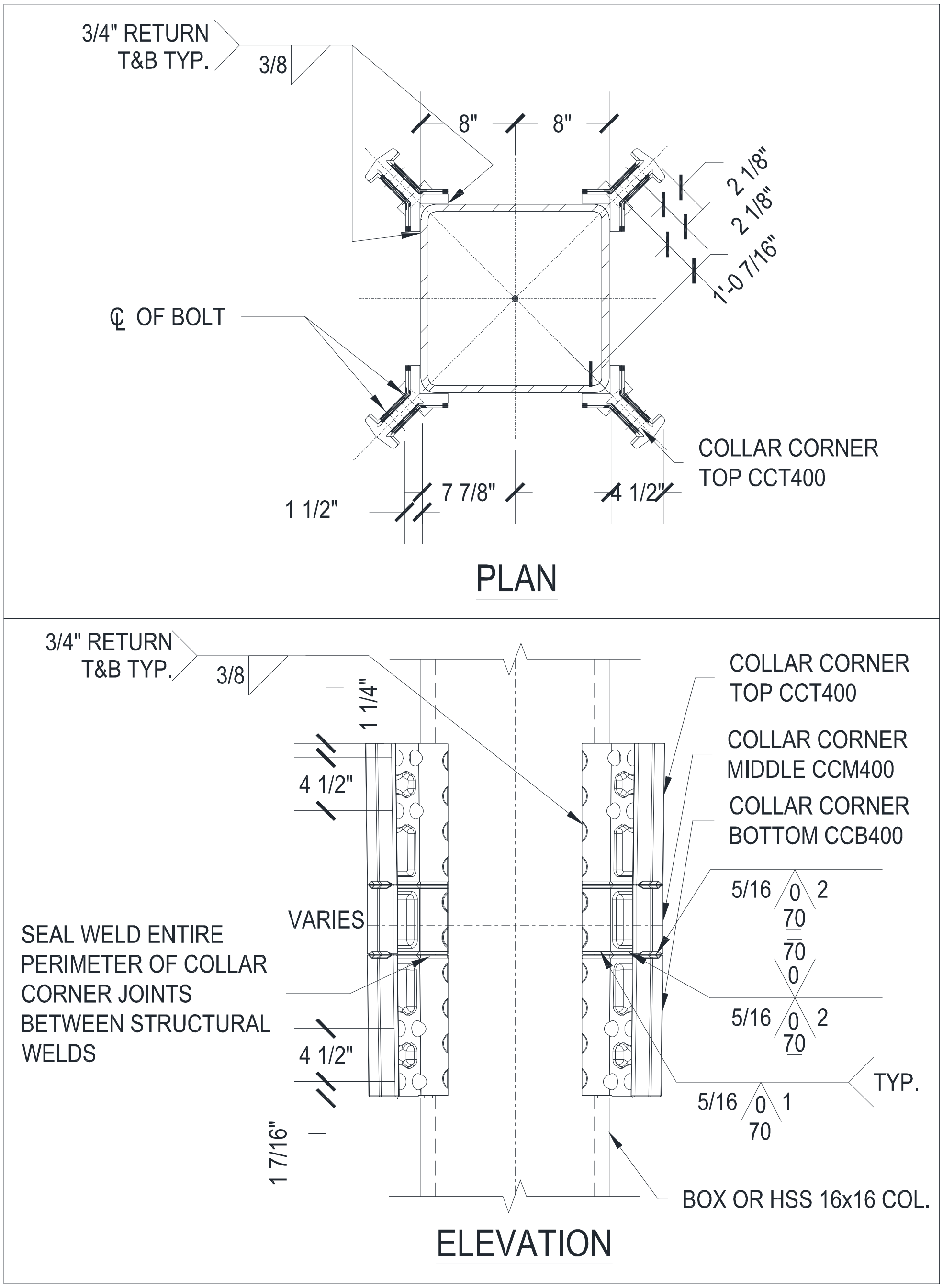


Figure 15: Elevation and Plan Views of the ConXL-400 Moment Connection.

2.1.6 Load Transfer Mechanism

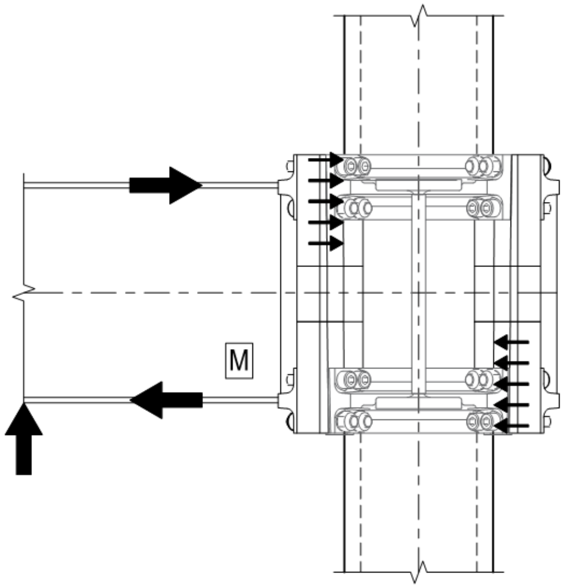


Figure 16: ConXL-400 Beam-To-Column Moment Transfer Mechanism.

Moment transfer between the beams and column is accomplished through direct compressive bearing as illustrated in

Figure 16. Compressive flexural forces are transferred from the beam's flange to the Collar Flange through the CJP weld connecting them.

The Collar Flange, acting as a beam spanning between the CCAs, delivers the compressive force through flexure to the CCAs via direct bearing. The CCAs delivers this bearing load to the column via fillet welds. Tensile flexural forces are transferred from the beam's flange to the Collar Flange through the CJP weld connecting them.

The Collar Flange, acting as a beam spanning between Collar Bolts, delivers the tensile load force through flexure to Collar Bolts, which transfers this force in tension to the orthogonal CFAs, which transfers this tensile force to the far Collar Bolts, which transfers the tension force to the "rear" CFA. The "rear" CFA, acting as a beam spanning between CCAs, transfers the force in flexure to the CCAs via direct bearing. The CCAs deliver this bearing load to the column via fillet welds.

The CFA at the end of a moment beam is checked for required strengths per loading conditions. Through statics, these CFA loads are reduced to the loads subjected to the individual Collar Flange. These loads are then used to check the required strengths for the individual Collar Flange.

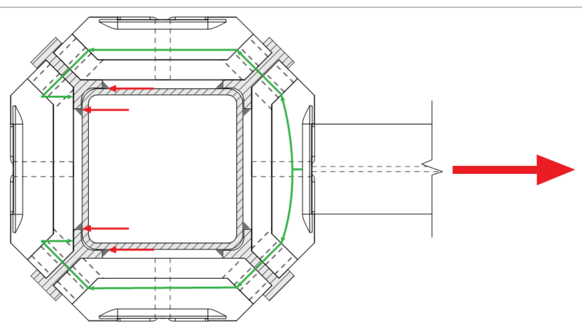


Figure 17: Load Path - Flange Tension Force to Column.

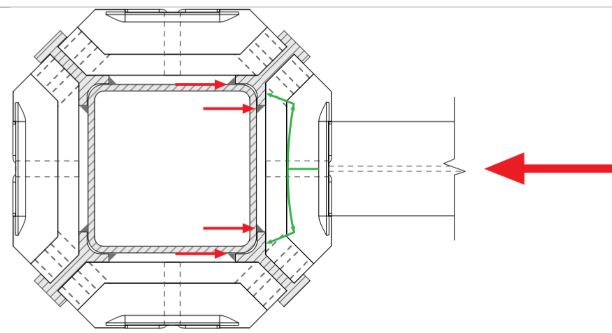


Figure 18: Load Path - Flange Compressive Force to Column.

2.1.7 Design Requirements

The design requirements detailed in this document for the ConXL-400 moment connection pertain to the fundamental strength of the moment connection for non-seismic applications.

This document provides the design formulations for the following:

- Collar Flange Connection - Design and Allowable Flexural Strength.
- Bolted Connection - X-axis Design and Allowable Flexural Strength.
- Bolted Connection - Y-axis Design and Allowable Flexural Strength.
- Bolted Connection - Design and Allowable Strength for Combined Flexural, Shear, & Axial Load.
- Weld at Beam Web/Collar Web Extension - Design and Allowable Shear Strength.
- Weld at Collar Web Extension/Collar Flange - Design and Allowable Shear Strength.
- Weld at Collar Corner Assembly/Column - Design and Allowable Shear Strength.
- Collar Corner Assembly - Design and Allowable Shear Strength.
- Friction at Collar Flange/Collar Corner - Design and Allowable Slip Resistance Strength.
- Bearing at Collar Flange and Collar Corner - Design and Allowable Bearing Strength.
- Collar Corner Assembly - Design and Allowable Tensile Strength.
- Collar Corner Assembly - Design and Allowable Strength for Combined Shear Load.
- Panel Zone - Design and Allowable Shear Strength.
- Local Yielding of HSS/Box Column Sidewalls - Design and Allowable Strength.
- Local Crippling of HSS/Box Column Sidewalls - Design and Allowable Strength.

Connection Design Notes:
The connection design procedure presented below is based on the *Specification*.

Definitions:

CF = Collar Flange: one CF welded to each flange of beam.

CFC = Collar Flange Connection: Top and bottom Collar Flange working in conjunction with Collar Web Extension, connecting end of beam to column.

CWX = Collar Web Extension: welded to top and bottom Collar Flange; beam web is welded to CWX.

CC = Collar Corner: welded to column; top and bottom Collar Flanges are clamped into CC's via bolt pretension; surface of CC and CF in contact with each other are machined surfaces.

M_{ux} = LRFD Load Combinations Required Flexural Strength – Beam X-axis, *k-ft* (kN-m)

M_{uy} = LRFD Load Combinations Required Flexural Strength – Beam Y-axis, *k-ft* (kN-m)

P_{ub} = LRFD Load Combinations Required Axial Strength – Beam, *kips* (kN)

V_{ux} = LRFD Load Combinations Required Shear Strength – Beam X-axis, *kips* (kN)

V_{uy} = LRFD Load Combinations Required Shear Strength – Beam Y-axis, *kips* (kN)

P_{uc} = LRFD Load Combinations Required Axial Strength – Column, *kips* (kN)

V_{uPZ} = LRFD Load Combinations Required Shear Strength – Panel Zone, *kips* (kN)

w_u = LRFD Load Combinations Uniform Applied Load – Beam, *k/ft* (kN/m)

2.1.8 Flexural Strength of Collar Flange Connection

The design check treats the Collar Flange as a simply supported beam spanning between the bolts that connect it to the Collar Corners. The flanges of the moment beam are welded to the Collar Flange with a complete joint penetration weld; thus, the Collar Flange (spanning between the bolts) at the beam flange is considered restrained. Consequently, the location for critical shear/moment for the Collar Flange occurs just outside of the beam flange.

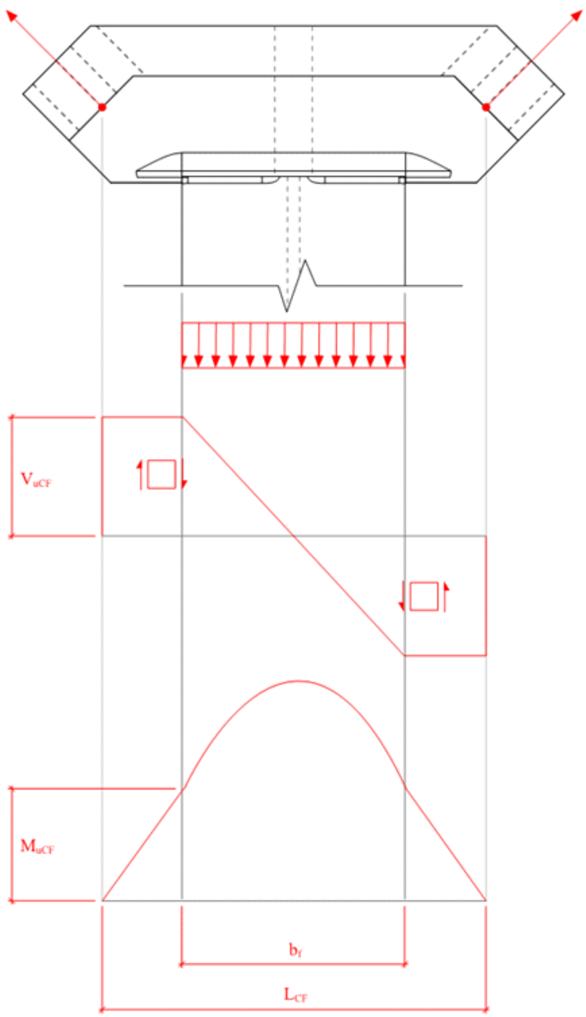


Figure 19: Shear and Moment Diagram of Collar Flange Under Loading.

V_{rCF} = Collar Flange req'd shear strength

M_{rCF} = Collar Flange req'd flex. strength

d = depth of beam

t_f = thickness of beam flange

L_{CF} = effective span of Collar Flange

Z_{CF} = plastic modulus of Collar Flange

A_{vCF} = shear area of Collar Flange

d_b = diameter of XL-400 bolt (ASTM A574)

d_b = 1.25 *in*. (31.8 mm)

F_{yCF} = yield stress of Collar steel (ASTM A572 Gr. 50)

F_{yCF} = 50 *ksi* (345 MPa)

As can be seen in Figure 19, the flexural capacity of the Collar Flange is the smaller of either the flexural strength of the CF, $F_{yCF}Z_{CF}$, *or* the shear strength of the CF, V_{nCF}, multiplied by the distance $(L_{CF} – b_f)/2$.

The section properties of the Collar Flange are shown in Figure 20.

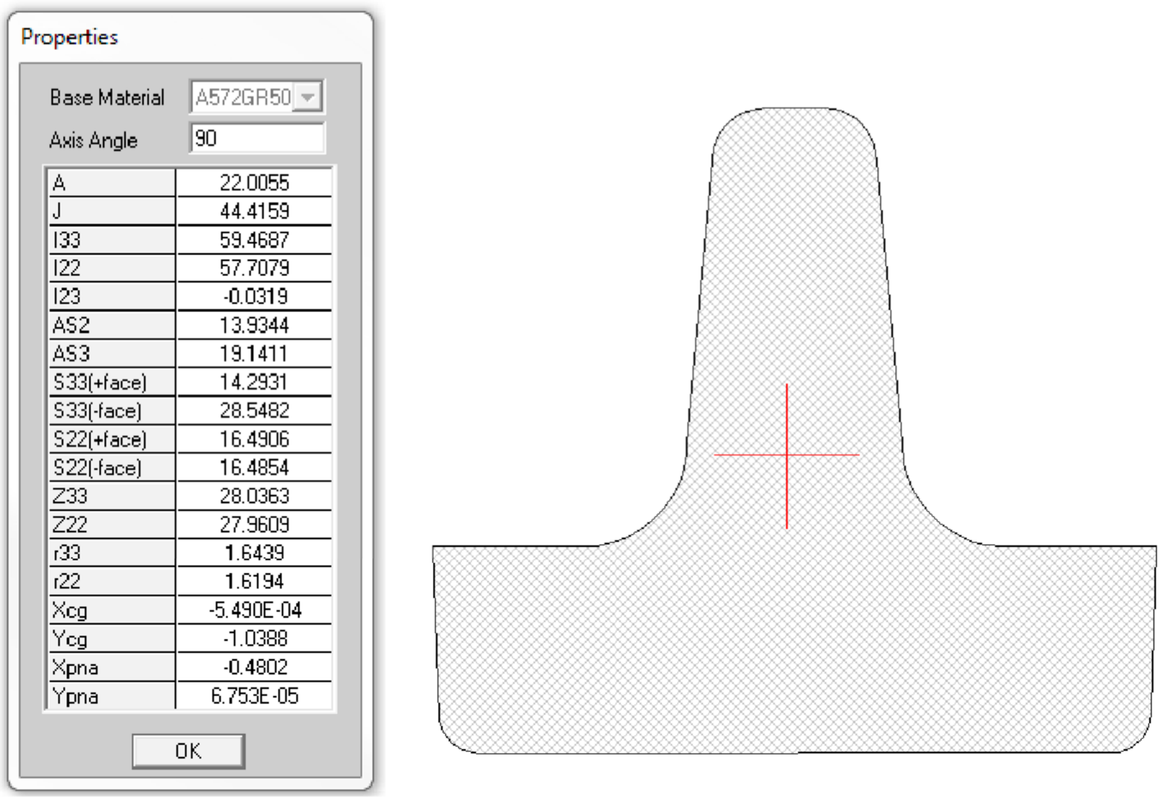
$$L_{CF400} = 16.3\, in. \quad (414\text{ mm})$$

$$A_{vCF400} = 13.93\, in.^2 \quad (8{,}987\text{ mm}^2)$$

$$Z_{CF400} = 28.04\, in.^3 \quad (459{,}493\text{ mm}^3)$$

$$n_{b400} = \text{number of bolts per beam end} = 16$$

$$cb_{400} = \text{width of compression block} = 0.991\, in. \quad (25.2\text{ mm})$$



A	22.0055
J	44.4159
I33	59.4687
I22	57.7079
I23	-0.0319
AS2	13.9344
AS3	19.1411
S33(+face)	14.2931
S33(-face)	28.5482
S22(+face)	16.4906
S22(-face)	16.4854
Z33	28.0363
Z22	27.9609
r33	1.6439
r22	1.6194
Xcg	-5.490E-04
Ycg	-1.0388
Xpna	-0.4802
Ypna	6.753E-05

Figure 20: ConXL-400 Collar Flange Section Properties.

As explained previously on page 8, the flexural capacity of the Collar Flange is the smaller of either the flexural strength of the CF, $F_{yCF}Z_{CF}$, *or* the shear strength of the CF, V_{nCF}, multiplied by the distance ($L_{CF} - b_f$)/2.

The nominal flexural strength of the CF is $M_n = F_{yCF}Z_{CF}$. The design flexural strength (DFS) and the allowable flexural strength of the Collar Flange are calculated as shown below:

$$F_{yCF} = 50\ ksi \quad (345\ \text{MPa})$$
$$Z_{CF} = 28.04\ in.^3 \quad (459{,}493\ \text{mm}^3)$$

LRFD	ASD
$\phi M_{nCF} = \phi F_{yCF} Z_{CF} = 0.90(50\ ksi)(28.04\ in.^3)$ $\phi M_{nCF} = 1{,}262\ kip-in.\ (142.6\ \text{kN-m})$	$\frac{M_{nCF}}{\Omega} = \frac{Z_{CF}F_{yCF}}{\Omega} = \frac{(50\ ksi)(28.04\ in.^3)}{1.67}$ $\frac{M_{nCF}}{\Omega} = 839.5\ kip-in. \quad (94.85\ \text{kN-m})$

The nominal shear strength, V_{nCF}, of the Collar Flange is calculated below:

$$V_{nCF} = 0.60 F_{yCF} A_{vCF}$$

where

$F_{yCF} = 50\ ksi \quad (345\ \text{MPa})$

$A_{vCF} = 13.93\ in.^2 \quad (8{,}987\ \text{mm}^2)$

Therefore, the design and the allowable critical moment demand of the Collar Flange created by a shear load equal to the shear strength of the Collar Flange are as follows:

LRFD	ASD
$\phi M_{rCF} = \phi V_{nCF}\left(\frac{L_{CF} - b_f}{2}\right)$	$\frac{M_{rCF}}{\Omega} = \frac{V_{nCF}}{\Omega}\left(\frac{L_{CF} - b_f}{2}\right)$

where

$L_{CF} = 16.3\ in.\ (414\ \text{mm})$

The flexural capacity of the Collar Flange is determined by comparing the value of the CF's flexural strength to the value of the critical moment demand created by a shear load equal to the shear strength of the Collar Flange. If the critical moment demand due to the shear strength of the Collar Flange is smaller than the flexural strength of the Collar Flange, then the Collar Flange shear strength governs; otherwise, the flexural strength of the Collar Flange governs.

The flexural capacity of the Collar Flange, can be expressed in a mathematical formula as shown below:

$$\text{Flexural Strength of Collar Flange } = \min\left(F_{yCF}Z_{CF}, V_{nCF}\left(\frac{L_{CF}-b_f}{2}\right)\right)$$

If the Collar Flange shear strength governs, the maximum beam moment that the connection can resist when the Collar Flange is the limiting component is as shown below:

LRFD	ASD
$\phi M_{nCFC} = \phi V_{nCF}(2)\left(d-t_f\right)$	$\frac{M_{nCFC}}{\Omega} = \frac{V_{nCF}}{\Omega}(2)\left(d-t_f\right)$

If the Collar Flange flexural strength governs, the maximum beam moment that the connection can resist when the Collar Flange is the limiting component is as shown below:

LRFD	ASD
$\phi M_{nCFC} = \frac{\phi M_{nCF}(2)\left(d-t_f\right)}{\left(\frac{L_{CF}-b_f}{2}\right)}$	$\frac{M_{nCFC}}{\Omega} = \frac{M_{nCF}(2)\left(d-t_f\right)}{\Omega\left(\frac{L_{CF}-b_f}{2}\right)}$

The Demand Capacity Ratio for the design and allowable Flexural Strength of the Collar Flange Connection is calculated as shown below:

LRFD	ASD
$DCR = \frac{M_{ux}}{\phi M_{nCFC}}$	$DCR = \frac{M_{ux}}{\left(\frac{M_{nCFC}}{\Omega}\right)}$

2.1.9 Bolted Connection: X-Axis Flexural Strength

As stated previously in this document, the ConXL-400 collar flanges transfer a beam's moment into the moment collar through pretensioned bolts. Because the size and number of bolts in the connection are a known constant, the maximum moment capacity of the connection can be determined in terms of the strength of the bolts.

A moment load in the strong-axis of the connected beam resolves into tension and compression forces within the beam's flange.

As can be seen in the figures below, the bolts are oriented 45 degrees relative to the longitudinal axis of the connected moment beam. Therefore, a tensile force from a beam's flange can be resolved into a shear force and a tension force in the bolt, as shown below.

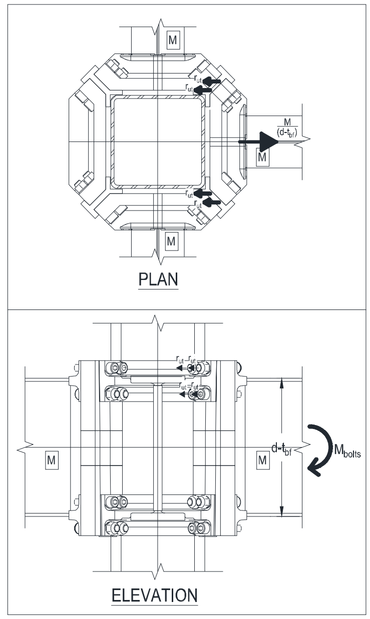


Figure 21: ConXL-400 Bolted Moment Connection Under Flexural Load.

$$f_r \cos(45°) = f_v = f_t$$

$$f_r = \sqrt{f_t^2 + f_v^2} = \sqrt{f_t^2 + f_t^2} = \sqrt{2} f_t$$

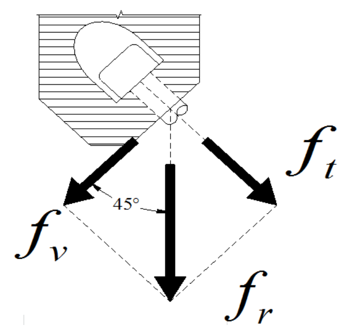


Figure 22: Bolt Load Resolved into a Tensile and Shear Force Components.

The following section determines the maximum available bolt load, considering the effects of combined loading as stipulated in the *Specifications*.

For the formulations that follow, the definitions below apply:

f_t = required tensile stress, *ksi* (MPa)
f_v = required shear stress, *ksi* (MPa)
f_t = f_v (bolts are 45° to applied load)
f_r = required resultant stress resisting applied load, *ksi* (MPa)
F_{nt} = nominal tensile stress = 0.75F_u, *ksi* (MPa) (*Specification* C-J3-2)
F_{nv} = nominal shear stress = 0.563F_u, *ksi* (MPa) (*Specification* C-J3-3)
F_u = specified minimum tensile strength of bolt, *ksi* (MPa)
ϕ_{bolts} = 0.75, resistance factor for bolts
Ω_{bolts} = 2.00, safety factor for bolts
R_n = nominal strength of bolt resisting applied load, *kips* (kN)
A_b = nominal unthreaded body area of bolt, *in.*^{2}

Continuing from the previous page,

LRFD	ASD
Specification (C-J3-5a) $$\left(\frac{f_t}{\phi_{bolts}F_{nt}}\right)^2+\left(\frac{f_v}{\phi_{bolts}F_{nv}}\right)^2=1$$	*Specification* (C-J3-5b) $$\left(\frac{\Omega_{bolts}f_t}{F_{nt}}\right)^2+\left(\frac{\Omega_{bolts}f_v}{F_{nv}}\right)^2=1$$
$$\left(\frac{f_t}{\phi_{bolts}0.75F_{nt}}\right)^2+\left(\frac{f_v}{\phi_{bolts}0.563F_{nv}}\right)^2=1$$ $$\left(\frac{f_t}{\phi_{bolts}0.75F_u}\right)^2+\left(\frac{f_t}{\phi_{bolts}0.563F_u}\right)^2=1$$ $$\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{1}{0.75^2}+\frac{1}{0.563^2}\right)=1$$ $$\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{0.563^2}{0.75^2\cdot 0.563^2}+\frac{0.75^2}{0.563^2\cdot 0.75^2}\right)=1$$ $$\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{0.563^2+0.75^2}{(0.563)^2(0.75)^2}\right)=1$$ $$f_t=\sqrt{\frac{(0.563)^2(0.75)^2}{\left(0.563^2+0.75^2\right)}\cdot\left(\phi_{bolts}F_u\right)^2}$$ $$f_t=0.45\left(\phi_{bolts}F_u\right)$$	$$\left(\frac{\Omega_{bolts}f_t}{0.75F_u}\right)^2+\left(\frac{\Omega_{bolts}f_v}{0.563F_u}\right)^2=1$$ $$\left(\frac{\Omega_{bolts}f_t}{0.75F_u}\right)^2+\left(\frac{\Omega_{bolts}f_t}{0.563F_u}\right)^2=1$$ $$\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{1}{0.75^2}+\frac{1}{0.563^2}\right)=1$$ $$\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{0.563^2}{0.75^2\cdot 0.563^2}+\frac{0.75^2}{0.563^2\cdot 0.75^2}\right)=1$$ $$\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{0.563^2+0.75^2}{(0.563)^2(0.75)^2}\right)=1$$ $$f_t=\sqrt{\frac{(0.563)^2(0.75)^2}{\left(0.563^2+0.75^2\right)}\cdot\left(\frac{F_u}{\Omega_{bolts}}\right)^2}$$ $$f_t=0.45\left(\frac{F_u}{\Omega_{bolts}}\right)$$

The *nominal* tensile capacity of a bolt is shown below:

$$R_n=f_rA_b=\sqrt{2}f_tA_b=\sqrt{2}\left(0.45\right)F_uA_b=0.636F_uA_b$$

The design tension and the allowable tension of a bolt is determined as shown below:

LRFD	ASD
$$\phi_{bolts}=0.75$$ $$\phi_{bolts}R_n=\phi_{bolts}f_rA_b=\phi_{bolts}\sqrt{2}f_tA_b$$ $$\phi_{bolts}R_n=\left(0.75\right)\sqrt{2}\left(0.45\right)F_uA_b=0.477F_uA_b$$	$$\Omega_{bolts}=2.00$$ $$\frac{R_n}{\Omega_{bolts}}=\frac{f_rA_b}{\Omega_{bolts}}=\frac{\sqrt{2}f_tA_b}{\Omega_{bolts}}$$ $$\frac{R_n}{\Omega_{bolts}}=\frac{\sqrt{2}\left(0.45\right)F_uA_b}{2.00}=0.318F_uA_b$$

Continuing from the previous page, it follows that the nominal flexural strength of the bolted moment connection is:

$$M_{ntx} = 0.636 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$$

where

- F_u = specified minimum tensile strength of bolt, *k*si (MPa)
- n_b = number of bolts in connection = 16
- A_b = nominal unthreaded body area of bolt = 1.227 *in.*^{2} (791.6 mm^2); diameter of bolt is 1.25 *in*. (31.8 mm)

Therefore, the design and allowable flexural strength of the bolted connection is as shown below:

LRFD	ASD
$\phi_{bolts} = 0.75$	$\Omega_{bolts} = 1.50$
$\phi_{bolts} M_{ntx} = \phi_{bolts} 0.636 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$	$\frac{M_{ntx}}{\Omega_{bolts}} = \frac{0.636 F_u A_b}{\Omega_{bolts}} \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$
$\phi_{bolts} M_{ntx} = (0.75) 0.636 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$	$\frac{M_{ntx}}{\Omega_{bolts}} = \frac{0.636 F_u A_b}{2.00} \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$
$\phi_{bolts} M_{ntx} = 0.477 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$	$\frac{M_{ntx}}{\Omega_{bolts}} = 0.318 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$

It follows that the Demand Capacity Ratio (DCR) for the design and allowable flexural strength of the bolted connection is as shown below:

LRFD	ASD
$DCR = \frac{M_{ux}}{\phi_{bolts} M_{ntx}}$	$DCR = \frac{M_{ux}}{\left(M_{ntx} / \Omega_{bolts}\right)}$

2.1.10 Bolted Connection: Y-Axis Flexural Strength

This section discusses the weak-axis flexural mechanics of the ConXL-400 collar connection. The weak-axis flexural capacity of the bolted connection will be determined.

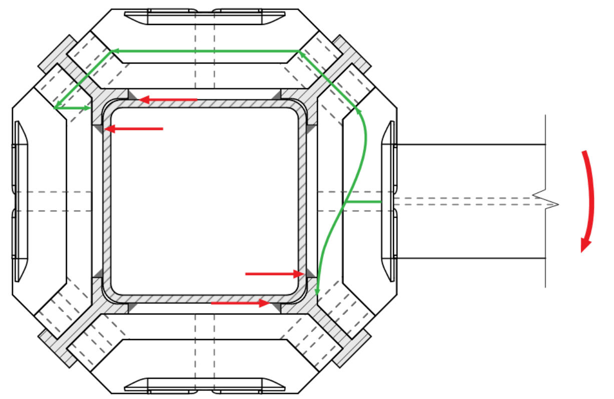


Figure 23: Y-Axis Bending Moment - Load Path Through Collars and Bolts to Column.

A moment load in a connected beam's weak-axis causes flexure in the two Collar Flanges attached to the beam, as depicted in Figure 23. The flexural load in the Collar Flanges causes tension on one side of the Collar Flanges and compression on the other side of the Collar Flanges. The tensile force in the Collar Flanges is transferred into the Collar Corners through the pretensioned bolts.

The compressive force causes bearing pressure between the Collar Flanges and the Collar Corners, as illustrated in Figure 24,

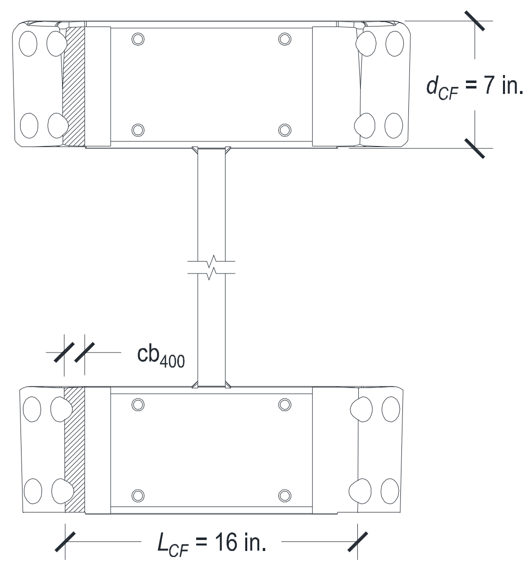


Figure 24: Cross-Section View of a ConXL-400 Collar Flange in Weak-Axis Flexure.

where

d_{CF} = effective depth of Collar Flange for Y-Axis bolt moment

L_{CF} = effective length of Collar Flange for Y-Axis bolt moment

cb_{400} = width of compression block for Y-Axis bolt moment

The width of the compression block in the Collar Flange is calculated by setting the bolt strength equal to the collar bearing strength. The ConXL-400 system can accept High Strength or Super High Strength bolts. The depth of the compression block in the CF is a function of the bolt strength. The stronger the bolts are, the longer the depth of the compression block, and therefore, the smaller the moment arm. Super High Strength bolts have a specified minimum tensile strength of 200 *ksi*. Thus, in the calculations that follow, an ultimate strength F_u of 200 *ksi* (1,379 MPa) is assumed for the bolts because this is conservative in determining the Y-Axis flexural capacity of the bolted connection.

Calculations of the width of the collar compression block are shown below. The bolt tension load is as follows:

LRFD	ASD
$F_u = 200\ ksi\ \ (1{,}379\ \text{MPa})$ $A_b = 1.227\ in.^2\ \ (791.6\ \text{mm}^2)$ $n_b = 16$ $\phi_{bolts} R_n = 0.477 F_u A_b \left(\frac{n_b}{2}\right)$ $= 0.477(200\ ksi)(1.227\ in.^2)\left(\frac{16}{2}\right)$ $= 936.4\ kips\ \ (4{,}165\ \text{kN})$	$F_u = 200\ ksi\ \ (1{,}379\ \text{MPa})$ $A_b = 1.227\ in.^2\ \ (791.6\ \text{mm}^2)$ $n_b = 16$ $\frac{R_n}{\Omega_{bolts}} = 0.318 F_u A_b \left(\frac{n_b}{2}\right)$ $= 0.318(200\ ksi)(1.227\ in.^2)\left(\frac{16}{2}\right)$ $= 624.3\ kips\ \ (2{,}777\ \text{kN})$

In accordance with *Specification* Section J7, the nominal bearing strength, R_n, for finished (milled) surfaces is as shown below:

$$R_n = 1.8 F_y A_{pb} \qquad \text{(J7-1)}$$

$$\phi = 0.75\ \ \text{(LRFD)} \qquad \Omega = 2.00\ \ \text{(ASD)}$$

where

F_y = specified minimum yield stress, *ksi* (MPa)
A_{pb} = projected bearing area, *in.*^{2} (mm^2)

The width of the compression block in the collar is determined by setting the bolt tension load equal to the bearing strength of the collar, as shown on the next page.

The calculations below consider both the top and the bottom Collar Flanges, and thus, the projected bearing area is twice the bearing area of one Collar Flange.

LRFD	ASD
$F_y = 50\ ksi\ \left(345\ \text{MPa}\right)$ $d_{CF} = 7.0\ in.\ \left(178\ \text{mm}\right)$ $A_{pb} = 2d_{CF}cb_{400}$ $\phi R_n = \phi 1.8 F_y A_{pb} = 936.4\ kips$ $0.75\left(1.8\left(50\ ksi\right)2\left(7.0\ in.\right)cb_{400}\right) = 936.4\ kips$ $cb_{400} = 0.991\ in.\ \left(25.2\ \text{mm}\right)$	$F_y = 50\ ksi\ \left(345\ \text{MPa}\right)$ $d_{CF} = 7.0\ in.\ \left(178\ \text{mm}\right)$ $A_{pb} = 2d_{CF}cb_{400}$ $\dfrac{R_n}{\Omega} = \dfrac{1.8 F_y A_{pb}}{\Omega} = 624.3\ kips$ $1.8\left(50\ ksi\right)2\left(7.0\ in.\right)cb_{400} = 2.00\left(624.3\ kips\right)$ $cb_{400} = 0.991\ in.\ \left(25.2\ \text{mm}\right)$

Now that the depth of the compression stress block has been determined, the Y-Axis flexural strength of the bolted connection can be calculated. Using d_c to denote the depth of the column, the design or allowable Y-Axis flexural strength of the bolted connection is as shown below:

LRFD	ASD
$d_c = 16\ in.\ \left(406.4\ \text{mm}\right)$ $cb_{400} = 0.991\ in.\ \left(25.2\ \text{mm}\right)$ $\phi_{bolts} M_{nty} = 0.477 F_u A_b \left(\dfrac{n_b}{2}\right)\left(\dfrac{L_{CF}}{2} + \dfrac{d_C}{2} - \dfrac{cb_{400}}{2}\right)$	$d_c = 16\ in.\ \left(406.4\ \text{mm}\right)$ $cb_{400} = 0.991\ in.\ \left(25.2\ \text{mm}\right)$ $\dfrac{M_{nty}}{\Omega_{bolts}} = 0.318 F_u A_b \left(\dfrac{n_b}{2}\right)\left(\dfrac{L_{CF}}{2} + \dfrac{d_C}{2} - \dfrac{cb_{400}}{2}\right)$

The Demand Capacity Ratio for the design or allowable Y-Axis flexural strength of the bolted connection is as shown below:

LRFD	ASD
$DCR = \dfrac{M_{uy}}{\phi_{bolts} M_{nty}}$	$DCR = \dfrac{M_{uy}}{\left(M_{nty}/\Omega_{bolts}\right)}$

2.1.11 Bolted Connection: Combined Flexural, Shear, and Axial Loading

Where there is a combined flexural (in both X and Y axes), shear, and axial load on the bolted moment connection, the Demand Capacity Ratio is determined as shown below:

LRFD
$DCR = \dfrac{M_{ux}}{\phi_{bolts} M_{ntx}} + \left(\dfrac{V_{ux}}{\phi R_{ncf}}\right)^2 + \dfrac{M_{uy}}{\phi_{bolts} M_{nty}} + \dfrac{P_{ub}}{n_b \phi_{bolts} R_n}$
ASD
$DCR = \dfrac{M_{ux}}{M_{ntx}/\Omega_{bolts}} + \left(\dfrac{V_{ux}}{R_{ncf}/\Omega}\right)^2 + \dfrac{M_{uy}}{M_{nty}/\Omega_{bolts}} + \dfrac{P_{ub}}{n_b R_n/\Omega_{bolts}}$

where

V_{ux} = required strong-axis beam shear load from load combinations.

P_{ub} = required beam axial load from load combinations.

R_{ncf} = available slip resistance from Section 2.1.17.

2.1.12 X-Axis Shear Transfer from Beam to Column

In the ConXL-400 moment collar connection, strong-axis shear is transferred to the Collar Flange Assembly via fillet welds between the beam web and the Collar Web Extension (CWX). The Collar Web Extension transfers the shear load via fillet welds between the top and bottom Collar Flanges (CFs) and the Collar Web Extension. This shear load is transferred to the Collar Corners (CC) through friction via direct bearing between machined surfaces of the top and bottom Collar Flanges and the Collar Corners clamped together by pretensioned bolts. The Collar Corners transfer the shear load to the column via fillet welds between the Collar Corners and the column.

The strength of the aforementioned welds is calculated in accordance with *Specification* Section J2.4, as follows:

$$R_n = F_{nw} A_{we} \qquad \text{(J2-4)}$$

$$\phi = 0.75 \;\; \text{(LRFD)} \qquad \Omega = 2.00 \;\; \text{(ASD)}$$

For fillet welds,

$$F_{nw} = 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5}(\theta)\right) \qquad \text{(J2-5)}$$

where

F_{EXX} = filler metal classification strength, *ksi* (MPa)

θ = angle of loading measured from the weld longitudinal axis, degrees

A_{we} = effective area of the weld, *in.*^{2} (mm^2)

The proceeding pages show calculations to determine the strength of the beam web to Collar Web Extension welds, Collar Web Extension to Collar Flange welds and Collar Corners to Column welds.

2.1.13 Weld at Beam Web to Collar Web Extension

The shear strength of the fillet welds between the Beam Web and the Collar Web Extension (CWX) is determined as follows:

The ConXL-400 *beam web* to Collar Web Extension (CWX) connection uses a 5/16 *in*. (7.94 mm) fillet weld on each side of the beam web, as shown in Figure 25.

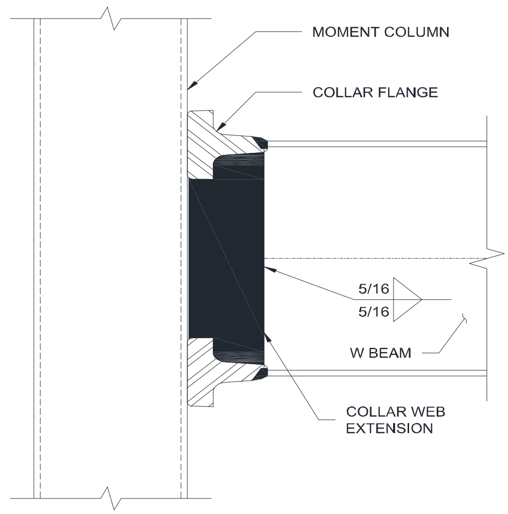


Figure 25: Beam Web-To-Collar Web Extension Welds.

The length of the fillet weld is:

$$l_{wb} = 2(d - 2k)\ in.\ (\mathrm{mm})$$

The size of the fillet weld is:

$$t_{we} = \frac{\sqrt{2}}{2} \cdot \frac{5}{16}\ in. = 0.221\ in.$$

The area of the fillet weld is:

$$A_{we} = t_{we} l_{wb}$$

The strength of the fillet weld is:

$$\theta = 0°$$

$$F_{EXX} = 70\ ksi\ \ (483\ \mathrm{MPa})$$

$$\begin{aligned} R_n &= 0.60 F_{EXX}\left(1.0 + 0.50 \sin^{1.5}(\theta)\right) A_{we} \\ &= 0.60(70\ ksi)\left(1.0 + 0.50 \sin^{1.5}(0°)\right)(0.221\ in.) \times 2(d - 2k) \\ &= 0.60(70\ ksi)(1.0)(0.221\ in.) \times 2(d - 2k) \end{aligned}$$

$$R_n = \left(18.56 \frac{kip}{in.}\right)(d - 2k)$$

The design and allowable shear strength of the Beam Web to Collar Web Extension fillet welds is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(18.56\,\dfrac{kip}{in.}\right)(d-2k)$ ENGLISH $\phi R_n = \left(13.92\,\dfrac{kip}{in.}\right)(d-2k)$ METRIC $\phi R_n = \left(2.438\,\dfrac{\text{kN}}{\text{mm}}\right)(d-2k)$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{\left(18.56\,\dfrac{kip}{in.}\right)(d-2k)}{2.00}$ ENGLISH $\dfrac{R_n}{\Omega} = \left(9.28\,\dfrac{kip}{in.}\right)(d-2k)$ METRIC $\dfrac{R_n}{\Omega} = \left(1.625\,\dfrac{\text{kN}}{\text{mm}}\right)(d-2k)$

The Demand Capacity Ratio (DCR) for the design and allowable shear strength of the Beam Web to Collar Web Extension (CWX) fillet welds is calculated as shown below:

LRFD	ASD
$DCR = \dfrac{V_{ux}}{\phi R_n}$	$DCR = \dfrac{V_{ux}}{\left(R_n/\Omega\right)}$

2.1.14 Weld at Collar Web Extension to Collar Flanges

The ConXL-400 Collar Web Extension (CWX) to Collar Flange connections use 5/16 *in*. (7.94 mm) fillet welds. As can be seen in the figure on the right, the weld consists of both a parallel and a perpendicular part relative to the direction of the beam shear load.

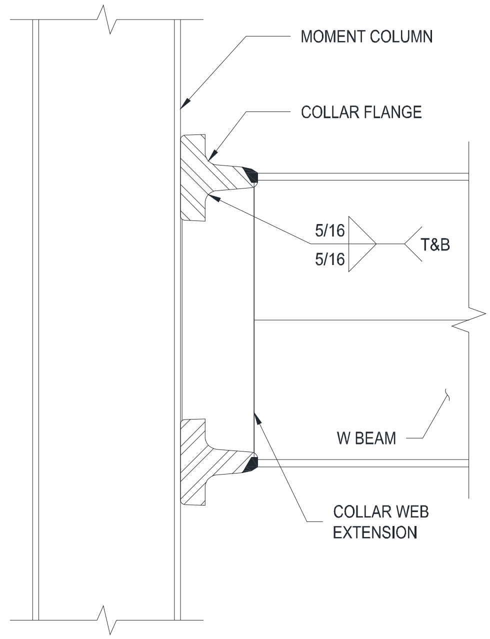


Figure 26: CWX-To-Collar Flange Welds.

The size of the fillet weld is:

$$t_{we} = \frac{\sqrt{2}}{2} \cdot \frac{5}{16}\, in. = 0.221\, in.$$

The total length and area of the perpendicular fillet welds is:

$$l_{wb_\perp} = 4(5.0\, in.) = 20.0\, in.$$
$$A_{we_\perp} = t_{we} l_{wb_\perp} = (0.221\, in.)(20.0\, in) = 4.42\, in.^2$$

The total length and area of the parallel fillet welds is:

$$l_{wb_\parallel} = 4(2.0\, in.) = 8.0\, in.$$
$$A_{we_\parallel} = t_{we} l_{wb_\parallel} = (0.221\, in.)(8.0\, in) = 1.77\, in.^2$$

The strength of the weld is the sum of the strengths of the parallel and the perpendicular welds. It is calculated below:

$$\theta_\parallel = 0^\circ$$
$$F_{EXX} = 70\, ksi \;\; (483 \text{ MPa})$$
$$\theta_\perp = 90^\circ$$
$$R_n = 0.60 F_{EXX}\left(1.0 + 0.50 \sin^{1.5}(\theta_\parallel)\right) A_{we\parallel} + 0.60 F_{EXX}\left(1.0 + 0.50 \sin^{1.5}(\theta_\perp)\right) A_{we\perp}$$

$$R_n = 0.60(70\, ksi)\left(1.0 + 0.50 \sin^{1.5}(0^\circ)\right)(1.77\, in.^2) \ldots$$
$$+ 0.60(70\, ksi)\left(1.0 + 0.50 \sin^{1.5}(90^\circ)\right)(4.42\, in.^2)$$

$$R_n = 0.60(70\, ksi)(1.77\, in.^2) + 0.60(70\, ksi)(1.5)(4.42\, in.^2)$$
$$R_n = 352.8\, kips$$

The design and allowable shear strength of the Collar Web Extension to Collar Flange fillet welds is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(352.8\ kips)$ $\phi R_n = 264.6\ kips\ \ (1{,}177\ \text{kN})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{352.8\ kips}{2.00}$ $\frac{R_n}{\Omega} = 176.4\ kips\ \ (784.6\ \text{kN})$

The Demand Capacity Ratio for design and allowable shear strength of the Collar Web Extension (CWX) to Collar Flange fillet welds is:

LRFD	ASD
$\text{DCR} = \frac{V_{ux}}{\phi R_n}$ ENGLISH $\text{DCR} = \frac{V_{ux}}{264.6\ kips}$ METRIC $\text{DCR} = \frac{V_{ux}}{1{,}177\ \text{kN}}$	$\text{DCR} = \frac{V_{ux}}{\left(R_n / \Omega\right)}$ ENGLISH $\text{DCR} = \frac{V_{ux}}{176.4\ kips}$ METRIC $\text{DCR} = \frac{V_{ux}}{784.6\ \text{kN}}$

2.1.15 Weld at Collar Corner Assembly and Column (X-Axis Shear)

The ConXL-400 Collar Corner Assembly to column connection is a 3/8 *in*. (9.53 mm) fillet weld. As can be seen in the figure below, the weld is parallel to the direction of the strong-axis beam shear load.

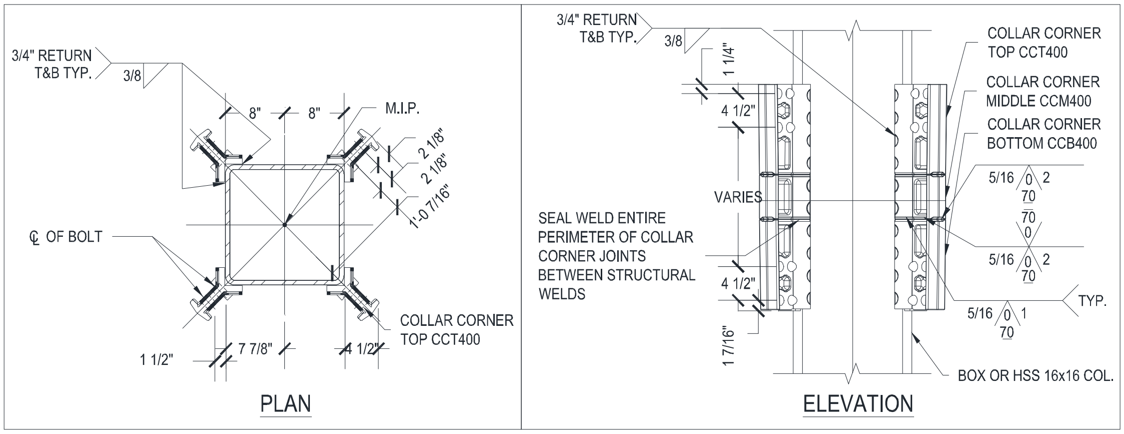


Figure 27: Collar Corner Assembly-To-Column Welds.

Using d_{nom} to denote the nominal depth of the connected beam, the total length of the fillet weld is:

$$l_{wc} = 2\left(d_{nom} + 6.0\ in.\right)$$

The size of the fillet weld is:

$$t_{wc} = \frac{\sqrt{2}}{2} \cdot \frac{3}{8}\ in. = 0.265\ in.$$

The area of the fillet weld is:

$$A_{wc} = t_{wc} l_{wc} = \left(0.265\ in.\right) 2\left(d_{nom} + 6.0\ in.\right)$$

The strength of the fillet weld is:

$$\theta = 0^\circ$$

$$\begin{aligned} R_n &= 0.60 F_{EXX}\left(1.0 + 0.50 \sin^{1.5}(\theta)\right) A_{we} \\ &= 0.60\left(70\ ksi\right)\left(1.0 + 0.50 \sin^{1.5}\left(0^\circ\right)\right)\left(0.53\ in.\right)\left(d_{nom} + 6.0\ in.\right) \\ &= 0.60\left(70\ ksi\right)\left(1.0 + 0\right)\left(0.53\ in.\right)\left(d_{nom} + 6.0\ in.\right) \end{aligned}$$

$$R_n = 22.3 \frac{kips}{in.}\left(d_{nom} + 6.0\ in.\right)$$

The design and allowable shear strength of the Collar Corner to column fillet welds is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(22.3\,\dfrac{kips}{in.}\left(d_{nom} + 6.0\ in.\right)\right)$	$\dfrac{R_n}{\Omega} = \dfrac{22.3\,\dfrac{kips}{in.}\left(d_{nom} + 6.0\ in.\right)}{2.00}$
ENGLISH	ENGLISH
$\phi R_n = 16.7\,\dfrac{kips}{in.}\left(d_{nom} + 6.0\ in.\right)$	$\dfrac{R_n}{\Omega} = 11.1\,\dfrac{kips}{in.}\left(d_{nom} + 6.0\ in.\right)$
METRIC	METRIC
$\phi R_n = 2.92\,\dfrac{\text{kN}}{\text{mm}}\left(d_{nom} + 152.40\ \text{mm}\right)$	$\phi R_n = 1.94\,\dfrac{\text{kN}}{\text{mm}}\left(d_{nom} + 152.40\ \text{mm}\right)$

The Demand Capacity Ratio for design and allowable shear strength of the Collar Corner to column fillet welds is:

LRFD	ASD
ENGLISH	ENGLISH
$DCR = \dfrac{V_{ux}}{\phi R_n} = \dfrac{V_{ux}}{16.7\,\dfrac{kips}{in.}\left(d_{nom} + 6.0\ in.\right)}$	$DCR = \dfrac{V_{ux}}{\left(R_n/\Omega\right)} = \dfrac{V_{ux}}{11.1\,\dfrac{kips}{in.}\left(d_{nom} + 6.0\ in.\right)}$
METRIC	METRIC
$DCR = \dfrac{V_{ux}}{\phi R_n} = \dfrac{V_{ux}}{2.92\,\dfrac{\text{kN}}{\text{mm}}\left(d_{nom} + 152.4\ \text{mm}\right)}$	$DCR = \dfrac{V_{ux}}{\phi R_n} = \dfrac{V_{ux}}{1.94\,\dfrac{\text{kN}}{\text{mm}}\left(d_{nom} + 152.4\ \text{mm}\right)}$

2.1.16 Shear Yielding and Shear Rupture in Collar Corner Assemblies (X-Axis Shear)

In accordance with *Specification* Section J4.2, the available shear strength of the affected and connecting elements shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

a. For shear yielding of the element:

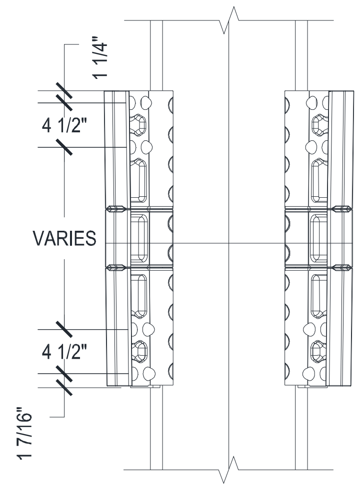
$$R_n = 0.60 F_y A_{gv} \qquad \text{(J4-3)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

b. For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \qquad \text{(J4-4)}$$

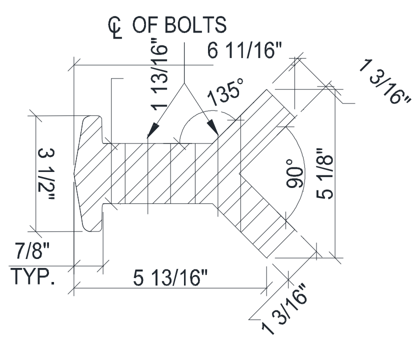
$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$



ELEVATION

The Collar Corner Assemblies are made from ASTM A572 Grade 50 material. This material has the following specifications:

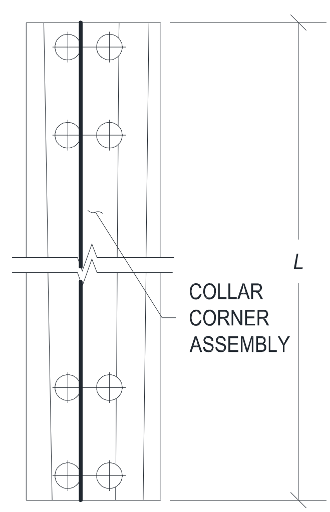
$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*



Plan View of Collar Corner Section; maximum dimension shown.

As stated below the figure directly on the right, the thickness of the CCA varies along its length. It starts out at 1-in. at the Collar Corner Top and is 1-13/16 *in*. at the Collar Corner Bottom. The average thickness of the CCA is 1.41 *in*. (35.8 mm).

The thickened vertical line in the figure on the right shows the shear plane of the gross section. The length of the shear plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism.



Shear Yielding in the CCA.

For shear yielding in the gross-section:

$$A_{gv} = (1.41\,in.)\,L = (1.41\,in.)(d_{nom})$$

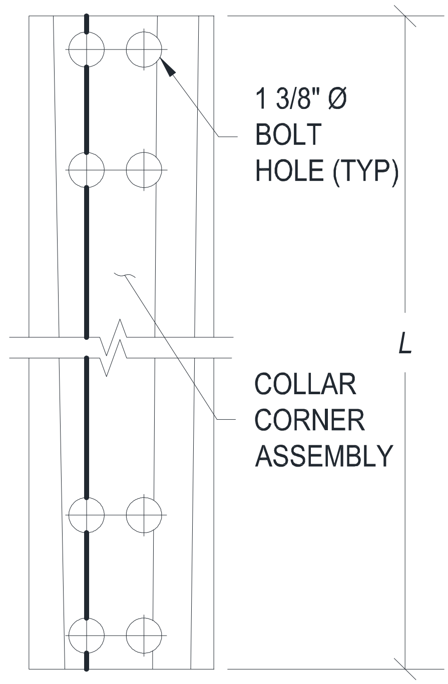
$$\begin{aligned} R_n &= 0.60 F_y A_{gv}\ x\ 2\ \text{CCAs} \\ &= 0.60(50\ ksi)(1.41\,in.)(d_{nom})\,x2 \\ &= \left(84.6\,\frac{kip}{in.}\right) d_{nom} \end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00\left(84.6\,\frac{kip}{in.}\right) d_{nom}$ ENGLISH $\phi R_n = \left(84.6\,\frac{kip}{in.}\right) d_{nom}$ METRIC $\phi R_n = \left(14.8\,\frac{\text{kN}}{\text{mm}}\right) d_{nom}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{\left(84.6\,\frac{kip}{in.}\right) d_{nom}}{1.50}$ ENGLISH $\frac{R_n}{\Omega} = \left(56.4\,\frac{kip}{in.}\right) d_{nom}$ METRIC $\frac{R_n}{\Omega} = \left(9.89\,\frac{\text{kN}}{\text{mm}}\right) d_{nom}$

The thickened vertical lines in the figure on the right show the shear plane of the net section. In conformance with *Specification* Section B4.3b, in computing the net area for tension and shear, the width of a bolt hole shall be taken as 1/16 *in*. (2 mm) greater than the nominal dimension of the hole.

The length of the shear plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism. The average thickness of the CCA was previously calculated to be 1.41 *in*. (35.8 mm).



Shear Rupture in the CCA.

For shear rupture in the net-section:

$$A_{nv} = \left(L - 4\left(d_h + \frac{1}{16}\,in.\right)\right)(1.41\,in.)$$

$$= \left(d_{nom} - 4\left(1\frac{3}{8}\,in. + \frac{1}{16}\,in.\right)\right)(1.41\,in.)$$

$$= (d_{nom} - 5.76\,in.)(1.41\,in.)$$

$$R_n = 0.60 F_u A_{nv}$$

$$= 0.60(65\,ksi)\left((d_{nom} - 5.76\,in.)1.41\,in.\right)\,x\ 2\ \text{CCAs}$$

$$= 110.0\,\frac{kips}{in.}(d_{nom} - 5.76\,in.)$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(110.0\,\frac{kips}{in.}(d_{nom} - 5.76\,in.)\right)$	$\frac{R_n}{\Omega} = \frac{110.0\,\frac{kips}{in.}(d_{nom} - 5.76\,in.)}{2.00}$
ENGLISH	ENGLISH
$\phi R_n = \left(82.5\,\frac{kip}{in.}\right)(d_{nom} - 5.76\,in.)$	$\frac{R_n}{\Omega} = \left(55.0\,\frac{kip}{in.}\right)(d_{nom} - 5.76\,in.)$
METRIC	METRIC
$\phi R_n = \left(14.4\,\frac{\text{kN}}{\text{mm}}\right)(d_{nom} - 146\text{ mm})$	$\frac{R_n}{\Omega} = \left(9.63\,\frac{\text{kN}}{\text{mm}}\right)(d_{nom} - 146\text{ mm})$

Note: Based on the preceding calculations, shear rupture is the governing limit state of the two.

2.1.17 Friction at Collar Flange / Collar Corners and Column

The next step in the design process is to determine the slip resistance of the friction force between the CFA and CCA and the Column.

In accordance with *Specification* Section J3.8, the design slip resistance, ϕR_n, and the allowable slip resistance, R_n/Ω, shall be determined for the limit state of slip as follows:

$$R_n = \left(\mu D_u h_f T_b n_s\right) n_b \qquad \text{(J3-4)}$$

For standard size and short-slotted holes perpendicular to the direction of the load,

$$\phi = 1.00 \ \ \text{(LRFD)} \qquad\qquad \Omega = 1.50 \ \ \text{(ASD)}$$

For the limit state of slip resistance, the following definitions apply:

μ =	mean slip coefficient for Class A or B surfaces. The CFA and CCA have Class A surfaces and therefore μ = 0.30.
D_u =	1.13, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension.
T_b =	minimum fastener tension given in *Specification* Section Table J3.1, *kips*, or Table J3.1M, kN. As specified in the ConXL-400 Standard Details, the ASTM A574 bolts are pretensioned to ASTM A490 bolt specifications. Per *Specification* Table J3.1, T_b = 102 kips for a 1-1/4 *in*. diameter bolt.
h_f =	factor for filler. There are no fillers present, thus, h_f =1.0
n_s =	number of slip planes. For this case, n_s = 1.0. Only one slip plane is used for each bolt because only the side pertaining to one beam is considered.
n_b =	number bolts. For this case, n_b = 16.

Using the information above, the slip resistance is determined as follows:

$$T_b = 102\ kips$$

$$R_n = \left(\mu D_u h_f T_b n_s\right) n_b$$

$$R_n = 0.30(1.13)(1.0)(102\ kips)(1.0)(16)$$

$$R_{ncf} = 553\ kips$$

The design slip resistance, ϕR_n, and the allowable slip resistance, R_n/Ω, is

LRFD	ASD
$\phi = 1.00$ $\phi R_{ncf} = 1.00(553.2\ kips) = 553.2\ kips$	$\Omega = 1.50$ $\dfrac{R_{ncf}}{\Omega} = \dfrac{553.2\ kips}{1.50} = 368.7\ kips$

The Demand Capacity Ratio for the shear strength of the friction force at the Collar Flange Collar Corner and column interface is shown below:

LRFD	ASD
$DCR = \dfrac{V_{ux}}{\phi R_{ncf}}$ ENGLISH $DCR = \dfrac{V_{ux}}{553.2\ kips}$ METRIC $DCR = \dfrac{V_{ux}}{2{,}461\ \text{kN}}$	$DCR = \dfrac{V_{ux}}{\left(R_{ncf}/\Omega\right)}$ ENGLISH $DCR = \dfrac{V_{ux}}{368.7\ kips}$ METRIC $DCR = \dfrac{V_{ux}}{1{,}640\ \text{kN}}$

2.1.18 Y-Axis Shear Transfer from Beam to Column

In the ConXL-400 moment collar connection, weak-axis shear is transferred to the Collar Flange Assembly via CJP welds between the beam flanges and the Collar Flanges (CFs). The Collar Flanges transfer the shear load via direct bearing against the Collar Corners (CCs); the weak-axis shear thus does not contribute to the interaction of forces resisted by the bolts. The Collar Corners transfer the shear load to the column via fillet welds between the Collar Corners and the column.

2.1.19 Bearing Strength at Collar Flange and Collar Corner

In accordance with *Specification* Section J7, the available bearing strength of surfaces in contact shall be determined for the limit state of bearing as follows:

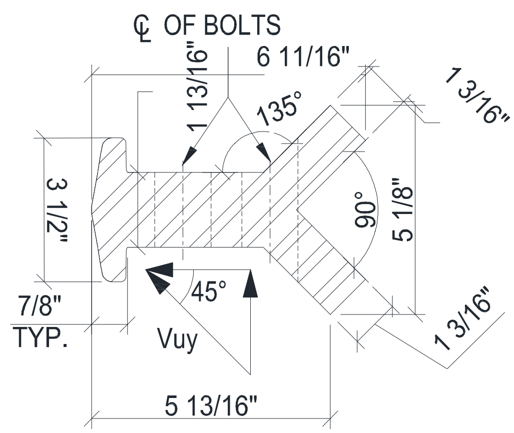
$$R_n = 1.8 F_y A_{pb} \qquad \text{(J7-1)}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

The Collar Corner Assemblies are made from ASTM A572 Grade 50 material. This material has the following specifications:

$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*

As shown in the figure on the right, the weak-axis shear is resisted by bearing of the Collar Flange against two surfaces of the Collar Corner. One component of the shear is resisted by the head of the Collar Corner and the other component is resisted by the neck of the Collar Corner.



Plan View of Collar Corner Section; maximum dimension shown.

The dimensions of the tapered Collar Corner that result in the minimum bearing areas will be used,

where:

$d_{CFb} =$ effective depth of Collar Flange for bearing
$w_{cch} =$ width of Collar Corner head
$w_{ccn} =$ width of Collar Corner neck
$t_{ccn} =$ thickness of Collar Corner neck

For bearing at the head of the Collar Corner:

$$A_{pbh} = 2d_{CF}\left(\frac{w_{cch} - t_{ccn}}{2}\right) = 2(7.00\,in.)\left(\frac{3.50\,in. - 1.8125\,in.}{2}\right)$$
$$= 11.81\,in.^2$$

$$R_{nbh} = 1.8F_yA_{pbh}$$
$$= 1.8(50\,ksi)(11.81\,in.^2)$$
$$= 1{,}063\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{nbh} = 0.75(1063\,kips)$ ENGLISH $\phi R_{nbh} = 797.2\,kips$ METRIC $\phi R_{nbh} = 3{,}546\,\text{kN}$	$\Omega = 2.00$ $\frac{R_{nbh}}{\Omega} = \frac{1063\,kips}{2.00}$ ENGLISH $\frac{R_{nbh}}{\Omega} = 531.5\,kips$ METRIC $\frac{R_{nbh}}{\Omega} = 2{,}364\,\text{kN}$

For bearing at the neck of the Collar Corner:

$$A_{pbn} = 2d_{CF}w_{ccn} = 2(7.00\,in.)(3.34\,in.)$$
$$= 46.76\,in.^2$$

$$R_{nbn} = 1.8F_yA_{pbn}$$
$$= 1.8(50\,ksi)(46.76\,in.^2)$$
$$= 4,208\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{nbn} = 0.75(4208\,kips)$ ENGLISH $\phi R_{nbn} = 3,156\,kips$ METRIC $\phi R_{nbn} = 14,040\,\text{kN}$	$\Omega = 2.00$ $\frac{R_{nbn}}{\Omega} = \frac{4208\,kips}{2.00}$ ENGLISH $\frac{R_{nbn}}{\Omega} = 2,104\,kips$ METRIC $\frac{R_{nbh}}{\Omega} = 9,359\,\text{kN}$

The Demand Capacity Ratio for the bearing strength of the Collar Corner Assembly is shown below:

LRFD	ASD
$DCR = \frac{V_{uy}\cos(45°)}{\phi R_{nbh}} + \frac{V_{uy}\sin(45°)}{\phi R_{nbn}}$	$DCR = \frac{V_{uy}\cos(45°)}{R_{nbh}/\Omega} + \frac{V_{uy}\sin(45°)}{R_{nbn}/\Omega}$

where

V_{uy} = required weak-axis beam shear load from load combinations.

2.1.20 Tensile Yielding and Tensile Rupture in Collar Corner Assembly

In accordance with *Specification* Section J4.1, the available tensile strength of the affected and connecting elements shall be the lower value obtained according to the limit states of tensile yielding and tensile rupture:

a. For tensile yielding of the element:

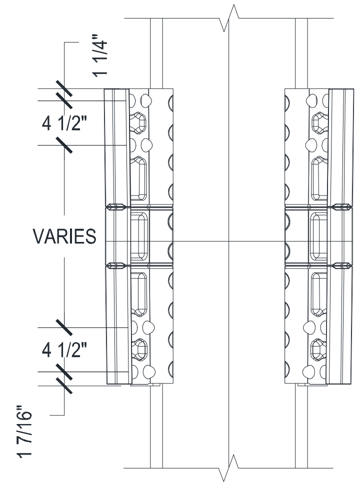
$$R_n = F_y A_g \quad \text{(J4-1)}$$

$\phi = 0.90\ \text{(LRFD)} \qquad \Omega = 1.67\ \text{(ASD)}$

b. For tensile rupture of the element:

$$R_n = F_u A_e \quad \text{(J4-4)}$$

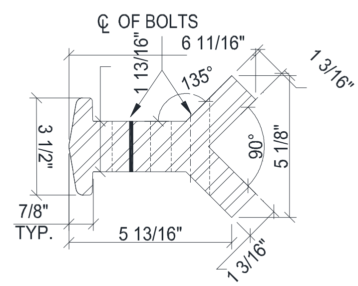
$\phi = 0.75\ \text{(LRFD)} \qquad \Omega = 2.00\ \text{(ASD)}$



ELEVATION

The Collar Corner Assemblies are made from ASTM A572 Grade 50 material. This material has the following specifications:

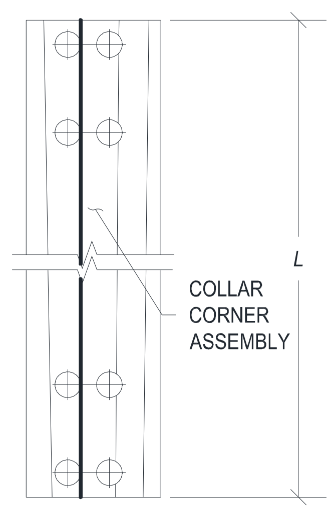
$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*



Plan View of Collar Corner Section with maximum dimensions shown; tension plane indicated by thickened vertical line.

As stated below the figure directly on the right, the thickness of the CCA varies along its length. It starts out at 1-in. at the Collar Corner Top and is 1-13/16 *in*. at the Collar Corner Bottom. The average thickness of the CCA is 1.41 *in*. (35.8 mm).

The thickened vertical line in the figure on the right shows the tension plane of the gross section. The length of the tension plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism.



Tensile Yielding in the CCA.

For tensile yielding in the gross-section:

$$A_g = (1.41 in.) L = (1.41 in.) d_{nom}$$

$$\begin{aligned} R_n &= F_y A_g \\ &= (50 ksi)(1.41 in.) d_{nom} \\ &= \left(70.5 \frac{kip}{in.}\right) d_{nom} \end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90\left(70.5 \frac{kip}{in.}\right) d_{nom}$ ENGLISH $\phi R_n = \left(63.4 \frac{kip}{in.}\right) d_{nom}$ METRIC $\phi R_n = \left(11.1 \frac{\text{kN}}{\text{mm}}\right) d_{nom}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{\left(70.5 \frac{kip}{in.}\right) d_{nom}}{1.67}$ ENGLISH $\frac{R_n}{\Omega} = \left(42.2 \frac{kip}{in.}\right) d_{nom}$ METRIC $\frac{R_n}{\Omega} = \left(7.39 \frac{\text{kN}}{\text{mm}}\right) d_{nom}$

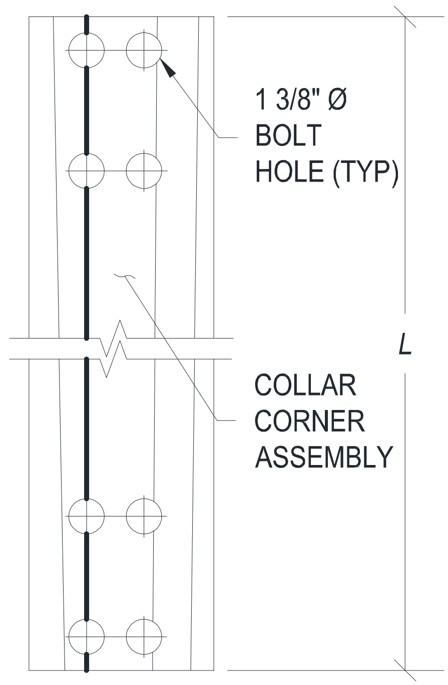
The thickened vertical lines in the figure on the right show the tension plane of the net section. In conformance with *Specification* Section B4.3b, in computing the net area for tension and shear, the width of a bolt hole shall be taken as 1/16 *in*. (2 mm) greater than the nominal dimension of the hole.

The length of the tension plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism. The average thickness of the CCA was previously calculated to be 1.41 *in*. (35.8 mm).

For tensile rupture on the effective net area:

$$A_e \approx A_{nt} = \left(L - 4\left(d_h + \frac{1}{16}in.\right)\right)(1.41 in.)$$
$$= \left(d_{nom} - 4\left(1\frac{3}{8}in. + \frac{1}{16}in.\right)\right)(1.41 in.)$$
$$= (d_{nom} - 5.76 in.)(1.41 in.)$$

$$R_n = F_u A_e$$
$$= (65 ksi)(d_{nom} - 5.76 in.)(1.41 in.)$$
$$= 91.6 \frac{kip}{in.}(d_{nom} - 5.76 in.)$$



Tensile Rupture in the CCA.

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(91.6 \frac{kip}{in.}(d_{nom} - 5.76\ in.)\right)$ ENGLISH $\phi R_n = \left(68.7 \frac{kip}{in.}\right)(d_{nom} - 5.76\ in.)$ METRIC $\phi R_n = \left(12.0 \frac{\text{kN}}{\text{mm}}\right)(d_{nom} - 146\ \text{mm})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{91.6 \frac{kip}{in.}(d_{nom} - 5.76\ in.)}{2.00}$ ENGLISH $\frac{R_n}{\Omega} = \left(45.8 \frac{kip}{in.}\right)(d_{nom} - 5.76\ in.)$ METRIC $\frac{R_n}{\Omega} = \left(8.02 \frac{\text{kN}}{\text{mm}}\right)(d_{nom} - 146\ \text{mm})$

Note: based on the preceding calculations, tensile rupture is the governing limit state of the two.

The Demand Capacity Ratio for the tensile strength of the Collar Corner Assembly is shown below:

LRFD	ASD
$DCR = \frac{V_{uy}\cos(45°)}{\phi R_n}$	$DCR = \frac{V_{uy}\cos(45°)}{R_n/\Omega}$

2.1.21 Shear Yielding and Shear Rupture in Collar Corner Assembly (at Neck, Y-Axis Shear)

The shear planes for shear yielding and shear rupture in the Collar Corner are identical to those described in Section 2.1.16. Since the Collar Flanges bear against one Collar Corner Assembly, only one Collar Corner Assembly resists the weak-axis shear (instead of two as is the case for strong-axis shear). Therefore, the available design strengths and allowable strengths will be one-half of those computed in Section 2.1.16.

For shear yielding in the gross-section, the available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00\left(84.6\,\frac{kip}{in.}\right)d_{nom} \times \frac{1}{2}$ ENGLISH $\phi R_n = \left(42.3\,\frac{kip}{in.}\right)d_{nom}$ METRIC $\phi R_n = \left(7.41\,\frac{\text{kN}}{\text{mm}}\right)d_{nom}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{\left(84.6\,\frac{kip}{in.}\right)d_{nom}}{1.50} \times \frac{1}{2}$ ENGLISH $\frac{R_n}{\Omega} = \left(25.3\,\frac{kip}{in.}\right)d_{nom}$ METRIC $\frac{R_n}{\Omega} = \left(4.44\,\frac{\text{kN}}{\text{mm}}\right)d_{nom}$

For shear rupture in the net-section, the available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(110.0\,\frac{kips}{in.}\left(d_{nom} - 5.76\,in.\right)\right)\times {}^{1}\!/_{2}$ ENGLISH $\phi R_n = \left(41.3\,\frac{kip}{in.}\right)\left(d_{nom} - 5.76\,in.\right)$ METRIC $\phi R_n = \left(7.22\,\frac{\text{kN}}{\text{mm}}\right)\left(d_{nom} - 146\text{ mm}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{110.0\,\frac{kips}{in.}\left(d_{nom} - 5.76\,in.\right)}{2.00}\times {}^{1}\!/_{2}$ ENGLISH $\frac{R_n}{\Omega} = \left(27.5\,\frac{kip}{in.}\right)\left(d_{nom} - 5.76\,in.\right)$ METRIC $\frac{R_n}{\Omega} = \left(4.82\,\frac{\text{kN}}{\text{mm}}\right)\left(d_{nom} - 146\text{ mm}\right)$

Note: Based on the preceding calculations, shear rupture is the governing limit state of the two.

The Demand Capacity Ratio for the shear strength of the Collar Corner Assembly at the neck is shown below:

LRFD	ASD
$DCR = \frac{V_{uy}\sin(45°)}{\phi R_n}$	$DCR = \frac{V_{uy}\sin(45°)}{R_n/\Omega}$

2.1.22 Shear Yielding in Collar Corner Assembly (at Head, Y-Axis Shear)

In accordance with *Specification* Section J4.2, the available shear strength of the affected and connecting elements shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

a. For shear yielding of the element:

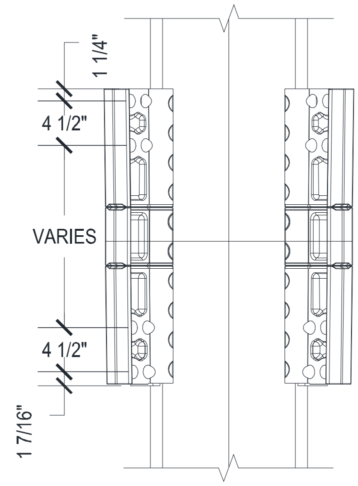
$$R_n = 0.60 F_y A_{gv} \qquad \text{(J4-3)}$$

$$\phi = 1.00\ \text{(LRFD)} \qquad \Omega = 1.50\ \text{(ASD)}$$

b. For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \qquad \text{(J4-4)}$$

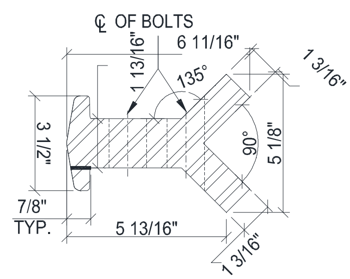
$$\phi = 0.75\ \text{(LRFD)} \qquad \Omega = 2.00\ \text{(ASD)}$$



ELEVATION

The Collar Corner Assemblies are made from ASTM A572 Grade 50 material. This material has the following specifications:

$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*

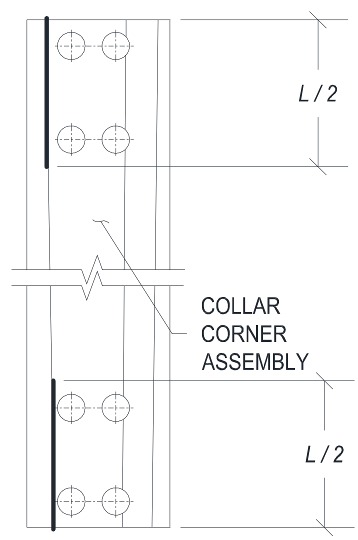


Plan View of Collar Corner Section with maximum dimensions shown; shear plane indicated by thickened horizontal line.

The limit state of shear rupture on the net-section will not be evaluated as there are no bolt holes interrupting the shear plane.

The thickened vertical lines in the figure on the right show the shear plane of the gross section. The length of the shear plane, L, is taken as twice the depth of the Collar Flange, d_{CF}, for conservatism.

The thickness of the Collar Corner head, t_{cch}, will be taken as the minimum thickness at the tip for conservatism



Shear Yielding in the CCA.

For shear yielding in the gross-section:

$$A_{gv} = 2d_{CF}t_{cch} = 2(7.00\,in.)(0.806\,in.)$$
$$= 11.28\,in.^2$$

$$R_n = 0.60F_yA_{gv}$$
$$= 0.60(50\,ksi)(11.28\,in.^2)$$
$$= 338.4\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(338.4\,kips)$ ENGLISH $\phi R_n = 338.4\,kips$ METRIC $\phi R_n = 1,505\,\text{kN}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{(338.4\,kips)}{1.50}$ ENGLISH $\frac{R_n}{\Omega} = 225.6\,kips$ METRIC $\frac{R_n}{\Omega} = 1,003\,\text{kN}$

The Demand Capacity Ratio for the shear strength of the Collar Corner Assembly at the head is shown below:

LRFD	ASD
$DCR = \frac{V_{uy}\cos(45°)}{\phi R_n}$	$DCR = \frac{V_{uy}\cos(45°)}{R_n/\Omega}$

2.1.23 Combined Shear and Tension in Collar Corner Assembly (Y-Axis Shear)

For the interaction of shear and tension in the Collar Corner Assembly, the Demand Capacity Ratio is determined as shown below:

LRFD	ASD
$DCR = \sqrt{\left(\frac{V_{uy}\cos(45°)}{\phi R_{ntr}}\right)^2 + \left(\frac{V_{uy}\sin(45°)}{\phi R_{nsr}}\right)^2}$	$DCR = \sqrt{\left(\frac{V_{uy}\cos(45°)}{R_{ntr}/\Omega}\right)^2 + \left(\frac{V_{uy}\sin(45°)}{R_{nsr}/\Omega}\right)^2}$

where:

R_{ntr} = tensile rupture strength at the neck from Section 2.1.20

R_{nsr} = shear rupture strength at the neck from Section 2.1.21

2.1.24 Weld at Collar Corner Assembly and Column (Y-Axis Shear)

The ConXL-400 Collar Corner Assembly to column connection is a 3/8 *in*. (9.53 mm) fillet weld. The weld is perpendicular to the direction of the weak-axis beam shear load. The load is oriented 90° to the longitudinal axis of the weld.

Using d_{nom} to denote the nominal depth of the connected beam, the total length of the fillet weld is:

$$l_{wc} = 2\left(d_{nom} + 6.0\,in.\right)$$

The size of the fillet weld is:

$$t_{wc} = \frac{\sqrt{2}}{2}\cdot\frac{3}{8}\,in. = 0.265\,in.$$

The area of the fillet weld is:

$$A_{wc} = t_{wc} l_{wc} = \left(0.265\,in.\right)2\left(d_{nom} + 6.0\,in.\right)$$

The strength of the fillet weld is:

$$\begin{aligned} \theta &= 90^{\circ} \\ R_n &= 0.60 F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right) A_{we} \\ &= 0.60\left(70\,ksi\right)\left(1.0 + 0.50\sin^{1.5}\left(90^{\circ}\right)\right)\left(0.53\,in.\right)\left(d_{nom} + 6.0\,in.\right) \\ &= 0.60\left(70\,ksi\right)\left(1.0 + 0.50\right)\left(0.53\,in.\right)\left(d_{nom} + 6.0\,in.\right) \end{aligned}$$

$$R_n = 33.4\,\frac{kips}{in.}\left(d_{nom} + 6.0\,in.\right)$$

The design and allowable shear strength of the Collar Corner to column fillet welds is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(33.4\frac{kips}{in.}(d_{nom} + 6.0\,in.)\right)$	$\frac{R_n}{\Omega} = \frac{33.4\frac{kips}{in.}(d_{nom} + 6.0\,in.)}{2.00}$
ENGLISH	ENGLISH
$\phi R_n = 25.0\frac{kips}{in.}(d_{nom} + 6.0\,in.)$	$\frac{R_n}{\Omega} = 16.7\frac{kips}{in.}(d_{nom} + 6.0\,in.)$
METRIC	METRIC
$\phi R_n = 4.38\frac{\text{kN}}{\text{mm}}(d_{nom} + 152.40\text{ mm})$	$\frac{R_n}{\Omega} = 2.92\frac{\text{kN}}{\text{mm}}(d_{nom} + 152.40\text{ mm})$

The Demand Capacity Ratio for design and allowable shear strength of the Collar Corner to column fillet welds is:

LRFD	ASD
ENGLISH	ENGLISH
$DCR = \frac{V_u}{\phi R_n} = \frac{V_u}{25.0\frac{kips}{in.}(d_{nom} + 6.0\,in.)}$	$DCR = \frac{V_u}{\left(R_n/\Omega\right)} = \frac{V_u}{16.7\frac{kips}{in.}(d_{nom} + 6.0\,in.)}$
METRIC	METRIC
$DCR = \frac{V_u}{\phi R_n} = \frac{V_u}{4.38\frac{\text{kN}}{\text{mm}}(d_{nom} + 152.4\text{ mm})}$	$DCR = \frac{V_u}{\left(R_n/\Omega\right)} = \frac{V_u}{2.92\frac{\text{kN}}{\text{mm}}(d_{nom} + 152.4\text{ mm})}$

2.1.25 Collar Corner Assembly: Combined Shear Loading

Where there is a combined shear load in both X and Y axes, the Demand Capacity Ratio is determined as shown below:

LRFD
$$DCR = \sqrt{\left(\frac{V_{ux}}{\phi R_{nsr}}\right)^2 + \left(\frac{V_{uy}\cos(45°)}{\phi R_{ntr}}\right)^2 + \left(\frac{V_{uy}\sin(45°)}{\phi R_{nsr}}\right)^2}$$
ASD
$$DCR = \sqrt{\left(\frac{V_{ux}}{R_{nsr}/\Omega}\right)^2 + \left(\frac{V_{uy}\cos(45°)}{R_{ntr}/\Omega}\right)^2 + \left(\frac{V_{uy}\sin(45°)}{R_{nsr}/\Omega}\right)^2}$$

2.1.26 Weld at Collar Corner Assembly: Combined Shear Loading

Where there is a combined shear load in both X and Y axes, the Demand Capacity Ratio is determined as shown below:

LRFD	ASD
$$DCR = \sqrt{\left(\frac{V_{ux}}{\phi R_{nwx}}\right)^2 + \left(\frac{V_{uy}}{\phi R_{nwy}}\right)^2}$$	$$DCR = \sqrt{\left(\frac{V_{ux}}{R_{nwx}/\Omega}\right)^2 + \left(\frac{V_{uy}}{R_{nwy}/\Omega}\right)^2}$$

where

R_{nwx} = available strong-axis shear strength from Section 2.1.15.

R_{nwy} = available weak-axis shear strength from Section 2.1.24.

2.1.27 Panel Zone Shear Strength

The final step of the ConXL-400 moment connection design process is determining the Shear Strength of the Panel Zone.

Panel zone shear is resisted by the column walls and the Collar Corner Assemblies. The Collar Corner Assemblies act as reinforcing plates for the column web.

The Collar Corner Assemblies are welded along their entire depth to the column, with their depth greater than the depth of the moment beam.

In accordance with *Specification* Section J10.6, the available strength of the web pane*l* zone for the limit state of shear yielding shall be determined as follows:

$$\phi = 0.90 \;\; (\text{LRFD}) \qquad \Omega = 1.67 \;\; (\text{ASD})$$

In the figure on the right, the shaded regions represent the area effective in resisting panel zone shear.

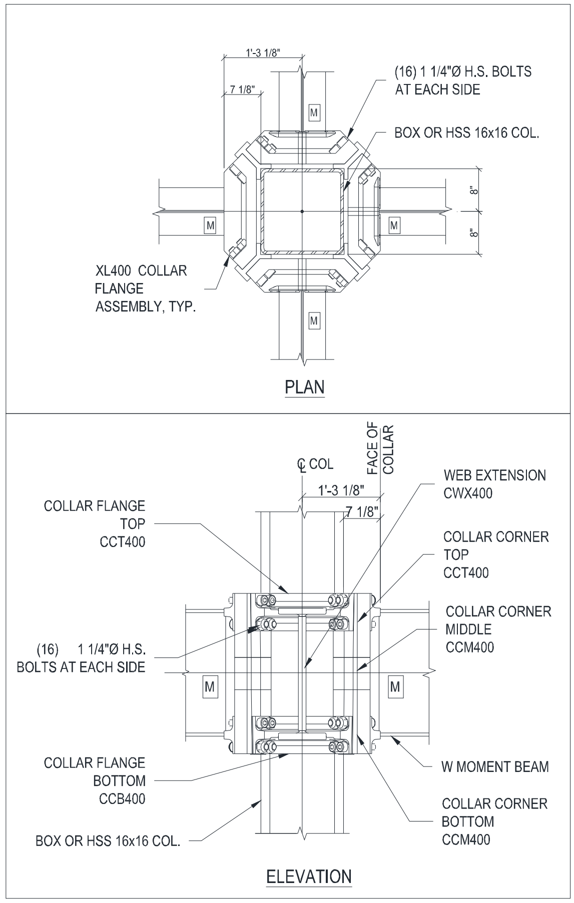


Figure 28: ConXL-400 Elevation and Plan View at Panel Zone.

The nominal strength, R_n, shall be determined as follows: when the effect of panel-zone deformation on frame stability is not considered in the analysis:

For $P_r \le 0.4P_c$
$$R_n = 0.60 F_y d_c t_w \qquad \text{(J10-9)}$$

For $P_r > 0.4P_c$
$$R_n = 0.60 F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \qquad \text{(J10-10)}$$

where

F_y = specified minimum yield stress of the column web, *ksi* (MPa)
d_c = depth of column, *in.* (mm)
t_w = thickness of column web, *in.* (mm)
P_r = required axial strength using LRFD or ASD load combinations, *kips* (kN)
P_c = $0.60P_y$, *kips* (kN)

The Collar Corner Assemblies contribute to the steel area resisting the panel zone shear. The properties of the column and CCAs pertaining to panel zone shear are shown below:

$A_{pz} =$ $2d_c t_c + 4d_{cc} t_{cc}$
$t_c =$ thickness of column wall, *in.* (mm)
$d_{cc} =$ effective depth of Collar Corner Assembly leg = 3.5 *in.* (88.9 mm)
$t_{cc} =$ effective thickness of Collar Corner Assembly leg = 0.5 *in.* (12.7 mm)
$A =$ column cross-sectional area, *in.*^{2} (mm^2)
$P_y =$ $F_y A$, axial yield strength of the column, *kips* (kN)

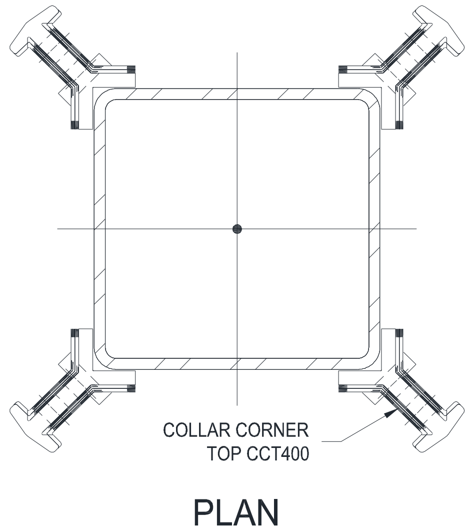


Figure 29: Effective Shear Areas of Column and CCA.

The axial yield strength of the column is determined as shown below:

$$P_c = P_y = F_y A = F_y \left(2d_c t_c + 4d_{cc} t_{cc}\right)$$

The shear strength of the panel zone is as follows:

$$\text{For } P_r \le 0.4P_c \qquad R_{nPZ} = 0.60F_y \left(2d_c t_c + 4d_{cc} t_{cc}\right)$$

$$\text{For } P_r > 0.4P_c \qquad R_{nPZ} = 0.60F_y \left(2d_c t_c + 4d_{cc} t_{cc}\right)\left(1.4 - \frac{P_r}{P_c}\right)$$

The design panel zone shear strength, ϕR_n, and the allowable panel zone shear strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi R_{nPZ} = 0.90R_{nPZ}$	$\frac{R_{nPZ}}{\Omega} = \frac{R_{nPZ}}{1.67}$

The Demand Capacity Ratio for the panel zone shear strength is shown below:

LRFD	ASD
$DCR = \frac{V_{uPZ}}{\phi R_{nPZ}}$	$DCR = \frac{V_{uPZ}}{\left(R_{nPZ}/\Omega\right)}$

2.1.28 Local Yielding of HSS Sidewalls

In accordance with Section J10.2 of the *Specification*, the limit state of Web Local Yielding (i.e., Local Yielding of HSS Sidewalls) is determined as follows:

$$R_n = F_{yw} t_w \left(5k + l_b\right) \qquad \text{(J10-2)}$$

$$\phi = 1.00 \ \text{(LRFD)} \qquad \Omega = 1.50 \ \text{(ASD)}$$

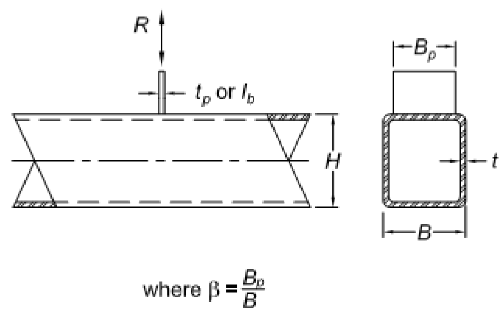
where

F_{yw} = specified minimum yield stress of HSS material, *ksi* (MPa)
l_b = bearing length, which equals the depth of the Collar Flange, d_{CF}.
k= outside HSS corner radius (0 for Box)

$$F_y = 50\,ksi \quad (345\,MPa)$$
$$t_w = t_{des}$$
$$l_b = d_{CF} = 7.0\,in.$$
$$k = 1.5t_{des}$$

Connection Type

Transverse Plate T- and Cross-Connections, Under Plate Axial Load



Note: since HSS members have two webs, the available strength resulting from Eq. J10-2 will be multiplied by 2.

Note: Eq. J10-2 only applies when the concentrated load is applied at a distance from the member end that is greater than the nominal depth of the HSS (i.e., 16 *in.*). If the concentrated load is applied at a distance from the member end that is less than or equal to the nominal depth of the HSS (e.g., if the collar is near the top of the column), then Eq. J10-3 is used instead as demonstrated on the following pages.

The available strength is therefore calculated as follows:

$$\begin{aligned} R_n &= F_{yw} t_w \left(5k + l_b\right) \times 2\,walls \\ &= \left(50\,ksi\right)\left(t_{des}\right)\left(5\left(1.5t_{des}\right) + 7.0\,in.\right) \times 2\,walls \\ &= \left(100\,ksi\right) t_{des} \left(7.5t_{des} + 7.0\,in.\right) \end{aligned}$$

The design wall local yielding strength, ϕR_n, and the allowable wall local yielding strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(100\ ksi)t_{des}(7.5t_{des} + 7.0\ in.)$ ENGLISH $\phi R_n = (100\ ksi)t_{des}(7.5t_{des} + 7.0\ in.)$ METRIC $\phi R_n = (689\ \text{MPa})t_{des}(7.5t_{des} + 177.8\ \text{mm})$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{(100\ ksi)t_{des}(7.5t_{des} + 7.0\ in.)}{1.50}$ ENGLISH $\dfrac{R_n}{\Omega} = (66.7\ ksi)t_{des}(7.5t_{des} + 7.0\ in.)$ METRIC $\dfrac{R_n}{\Omega} = (460\ \text{MPa})t_{des}(7.5t_{des} + 177.8\ \text{mm})$

When the concentrated force is applied at a distance from the member end that is less than or equal to the nominal depth of the HSS (i.e., 16 *in.*), the coefficient on *k* decreases to 2.5 (Eq. J10-3); for example, this condition should be checked when the collar is located near the top of the column. The design wall local yielding strength, ϕR_n, and the allowable wall local yielding strength, R_n/Ω, are then calculated as follows:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(100\ ksi)t_{des}(2.5(1.5t_{des}) + 7.0\ in.)$ ENGLISH $\phi R_n = (100\ ksi)t_{des}(3.75t_{des} + 7.0\ in.)$ METRIC $\phi R_n = (689\ \text{MPa})t_{des}(3.75t_{des} + 177.8\ \text{mm})$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{(100\ ksi)t_{des}(2.5(1.5t_{des}) + 7.0\ in.)}{1.50}$ ENGLISH $\dfrac{R_n}{\Omega} = (66.7\ ksi)t_{des}(3.75t_{des} + 7.0\ in.)$ METRIC $\dfrac{R_n}{\Omega} = (460\ \text{MPa})t_{des}(3.75t_{des} + 177.8\ \text{mm})$

The Demand Capacity Ratio for wall local yielding is shown below:

LRFD	ASD
$DCR = \dfrac{P_{rf}}{\phi R_n}$	$DCR = \dfrac{P_{rf}}{\left(R_n / \Omega \right)}$

where

$$P_{rf} = \left(\frac{M_{ux}}{d - t_f} \right) + \frac{P_{ub}}{2}$$

with M_{ux} and the P_{ub} are the required beam moment (X-axis) and the required beam axial demand, respectively. The term d denotes the depth of the beam and the term t_f is the thickness of the beam flange.

2.1.29 Local Crippling of HSS Sidewalls

In accordance with Section J10.3 of the *Specification*, the limit state of Web Local Yielding (i.e., Local Crippling of HSS Sidewalls) is determined as follows:	**Connection Type**

In accordance with Section J10.3 of the *Specification*, the limit state of Web Local Yielding (i.e., Local Crippling of HSS Sidewalls) is determined as follows:

$$R_n = 0.80 t_w^2 \left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f$$

(J10-4)

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$

where

E = modulus of elasticity of steel = 29,000 *ksi* (200,000 MPa)

F_{yw} = specified minimum yield stress of HSS main member material, *ksi* (MPa)

l_b = thickness of transverse plate, which equals the depth of the Collar Flange, d_{CF}.

t_w = design web thickness of HSS or Box column = t_{des}

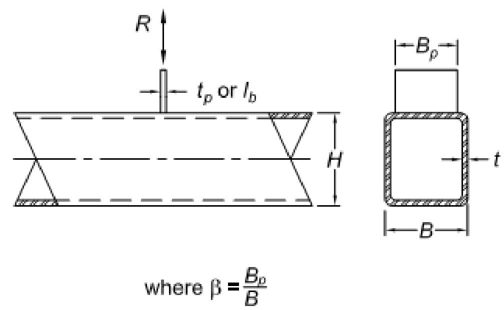
t_f = design flange thickness of HSS or Box column = t_{des}

d = depth of web, *in*. (mm) = $H - 3t_{des}$

Q_f = chord-stress interaction parameter

Connection Type

Transverse Plate T- and Cross-Connections, Under Plate Axial Load



Note: since HSS members have two webs, the available strength resulting from Eq. J10-4 will be multiplied by 2.

Note: Eq. J10-4 only applies when the concentrated compressive force is applied at a distance from the member end that is greater than or equal to $d/2$. If the concentrated compressive force is applied at a distance from the member end that is less than $d/2$ (e.g., if the collar is near the top of the column), then Eq. J10-5a or J10-5b should be used instead as demonstrated on the following pages.

From *Specification* Table K3.2, Q_f is determined as follows:

$Q_f = 1$ for connecting surface in tension,

$Q_f = 1.3 - 0.4\frac{U}{\beta}$ for connecting surface in compression.

Per *Specification* Section K3.1, the width ratio, β, is defined as the ratio of overall branch width (i.e., Collar Flange length) to chord width (i.e., column width) = B_b / B for rectangular HSS. Since the Collar Flange length is slightly larger than the column width, β = 1.0. The utilization ratio, *U*, is determined as follows using Eq. K2-4:

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right|$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS.

$P_{ro} = P_u$ for LRFD; P_a for ASD.
$M_{ro} = M_u$ for LRFD; M_a for ASD.

The available strength based on the limit state of local crippling is calculated as follows:

$$F_y = 50\,ksi \quad (345\,MPa)$$
$$H = 16\,in. \quad (406.4\,mm)$$
$$l_b = d_{CF} = 7.0\,in.$$

$$Q_f = 1.3 - 0.4\left(\frac{U}{1.0}\right) \le 1.0$$

$$R_n = 0.80 t_w^{\,2}\left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f \times 2\,walls$$
$$= 1.6 t_{des}^{\,2}\left(1 + 3\left(\frac{d_{CF}}{H - 3t_{des}}\right)\left(\frac{t_{des}}{t_{des}}\right)^{1.5}\right)\sqrt{\frac{EF_y t_{des}}{t_{des}}}Q_f$$
$$= 1.6 t_{des}^{\,2}\left(1 + \frac{3d_{CF}}{H - 3t_{des}}\right)\sqrt{EF_y}Q_f$$
$$= 1.6 t_{des}^{\,2}\left(\frac{H - 3t_{des} + 3d_{CF}}{H - 3t_{des}}\right)\sqrt{EF_y}Q_f$$
$$= 1.6 t_{des}^{\,2}\left(\frac{16\,in. - 3t_{des} + 3(7.0\,in.)}{16\,in. - 3t_{des}}\right)\sqrt{(29000\,ksi)(50\,ksi)}Q_f$$

$$R_n = (1{,}926\,ksi)\,t_{des}^{\,2}\left(\frac{37\,in. - 3t_{des}}{16\,in. - 3t_{des}}\right)Q_f$$

The design local wall crippling strength, ϕR_n, and the allowable local wall crippling strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(1{,}926\ ksi)t_{des}^{\ 2}\left(\dfrac{37\ in. - 3t_{des}}{16\ in. - 3t_{des}}\right)Q_f$ ENGLISH $\phi R_n = (1{,}445\ ksi)t_{des}^{\ 2}\left(\dfrac{37\ in. - 3t_{des}}{16\ in. - 3t_{des}}\right)Q_f$ METRIC $\phi R_n = (9{,}962\ \text{MPa})t_{des}^{\ 2}\left(\dfrac{939.8\ \text{mm} - 3t_{des}}{406.4\ \text{mm} - 3t_{des}}\right)Q_f$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{(1{,}926\ ksi)t_{des}^{\ 2}\left(\dfrac{37\ in. - 3t_{des}}{16\ in. - 3t_{des}}\right)Q_f}{2.00}$ ENGLISH $\dfrac{R_n}{\Omega} = (963\ ksi)t_{des}^{\ 2}\left(\dfrac{37\ in. - 3t_{des}}{16\ in. - 3t_{des}}\right)Q_f$ METRIC $\dfrac{R_n}{\Omega} = (6{,}639\ \text{MPa})t_{des}^{\ 2}\left(\dfrac{939.8\ \text{mm} - 3t_{des}}{406.4\ \text{mm} - 3t_{des}}\right)Q_f$

If the concentrated compressive force is applied at a distance from the member end that is less than $d/2$, then either Eq. J10-5a or J10-5b shall be used instead depending on the quantity l_b/d.

$$\frac{l_b}{d} = \frac{d_{CF}}{H - 3t_{des}} = \frac{7.0\,in.}{16.0\,in. - 3t_{des}}$$

The quantity l_b/d will always be greater than or equal to 0.2 regardless of t_{des}; therefore, Eq. J10-5b applies and the local wall crippling strength shall be determined as shown on the following page.

$$R_n = 0.40t_w^{\ 2}\left(1+\left(\frac{4l_b}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f \times 2\,walls$$

$$= 0.80t_{des}^{\ 2}\left(1+\left(\frac{4d_{CF}}{H-3t_{des}}-0.2\right)\left(\frac{t_{des}}{t_{des}}\right)^{1.5}\right)\sqrt{\frac{EF_y t_{des}}{t_{des}}}Q_f$$

$$= 0.80t_{des}^{\ 2}\left(1+\frac{4d_{CF}}{H-3t_{des}}-0.2\right)\sqrt{EF_y}Q_f$$

$$= 0.80t_{des}^{\ 2}\left(\frac{4d_{CF}+0.8\left(H-3t_{des}\right)}{H-3t_{des}}\right)\sqrt{EF_y}Q_f$$

$$= 0.80t_{des}^{\ 2}\left(\frac{4\left(7.0\,in.\right)+0.8\left(16\,in.\right)-2.4t_{des}}{16\,in.-3t_{des}}\right)\sqrt{\left(29{,}000\,ksi\right)\left(50\,ksi\right)}Q_f$$

$$R_n = \left(963\,ksi\right)t_{des}^{\ 2}\left(\frac{40.8\,in.-2.4t_{des}}{16\,in.-3t_{des}}\right)Q_f$$

The design local wall crippling strength, ϕR_n, and the allowable local wall crippling strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(963\ ksi\right)t_{des}^{\ 2}\left(\frac{40.8\ in.-2.4t_{des}}{16\ in.-3t_{des}}\right)Q_f$ ENGLISH $\phi R_n = \left(722\ ksi\right)t_{des}^{\ 2}\left(\frac{40.8\ in.-2.4t_{des}}{16\ in.-3t_{des}}\right)Q_f$ METRIC $\phi R_n = \left(4{,}980\ \mathrm{MPa}\right)t_{des}^{\ 2}\left(\frac{1{,}036\ \mathrm{mm}-2.4t_{des}}{406.4\ \mathrm{mm}-3t_{des}}\right)Q_f$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{\left(963\ ksi\right)t_{des}^{\ 2}\left(\frac{40.8\ in.-2.4t_{des}}{16\ in.-3t_{des}}\right)Q_f}{2.00}$ ENGLISH $\frac{R_n}{\Omega} = \left(481\ ksi\right)t_{des}^{\ 2}\left(\frac{40.8\ in.-2.4t_{des}}{16\ in.-3t_{des}}\right)Q_f$ METRIC $\frac{R_n}{\Omega} = \left(3{,}320\ \mathrm{MPa}\right)t_{des}^{\ 2}\left(\frac{1{,}036\ \mathrm{mm}-2.4t_{des}}{406.4\ \mathrm{mm}-3t_{des}}\right)Q_f$

The Demand Capacity Ratio for wall local crippling is shown below:

LRFD	ASD
$DCR = \frac{P_{rf}}{\phi R_n}$	$DCR = \frac{P_{rf}}{\left(R_n / \Omega \right)}$

2.2 ConXL300

2.2.1 Moment Connection Description

The ConXtech ConXL-300 moment connection is a steel moment-resisting beam to column connection for resisting gravity, wind and seismic forces imposed on a building/non-building structure. As can be seen in Figure 30, the collar connection is designed to join hollow structural section columns (or built-up box columns) and wide flange steel beams at all four sides of the column, although the connection can be used for as few as one beam.

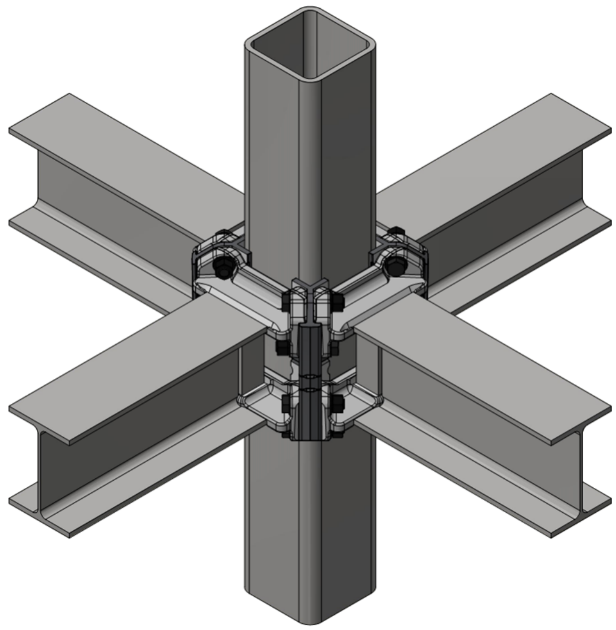


Figure 30: ConXL-300 Moment Connection Assembly.

Figure 31 shows an isolated view of a fully assembled moment collar. The moment collar is comprised of various components. The components that make up each face of the collar are joined together by welds, and the full collar assembly is held together by sixteen pretensioned high strength bolts.

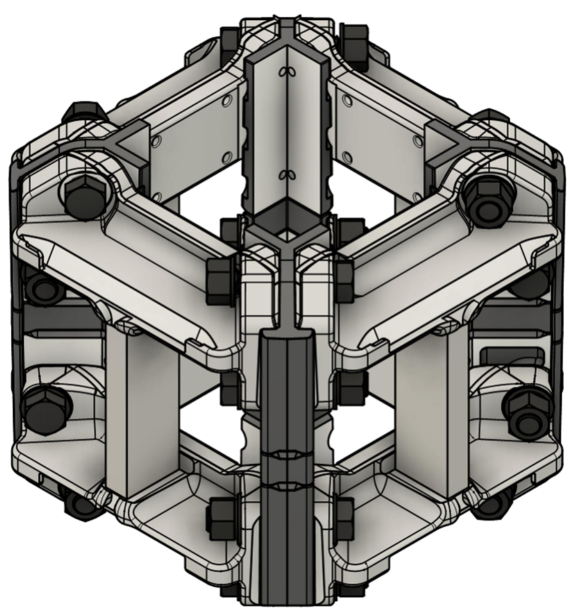


Figure 31: 3D-Perspective View of ConXL-300 Moment Collar Assembly.

The ConXL-300 connection is only used on columns that are 12 *in*. (304.8 mm) square combined with wide flange beams of the sizes shown in Table 2. Note that the connection can only be used with moment beams from the same nominal depth family.

Table 2: List of Compatible Beam Sizes for the ConXL-300 System.

W12	W14	W16	W18	W21
W12X19	W14X22	W16X26	W18X35	W21X44
W12X22	W14X26	W16X31	W18X40	W21X50
W12X26	W14X30	W16X36	W18X46	W21X57
W12X30	W14X34	W16X40	W18X50	W21X48
W12X35	W14X38	W16X45	W18X55	W21X55
W12X40	W14X43	W16X50	W18X60	W21X62
W12X45	W14X48	W16X57	W18X65	W21X68
W12X50	W14X53	W16X67	W18X71	W21X73
W12X58	W14X61			
	W14X68			

A Collar Flange Assembly (CFA) is welded to the end of a moment beam and the Collar Corner Assembly (CCA) is welded to the moment column. The ConXL-300 moment connection assembly is comprised of four CFAs at each face of the moment column and four CCAs at each corner of the square column.

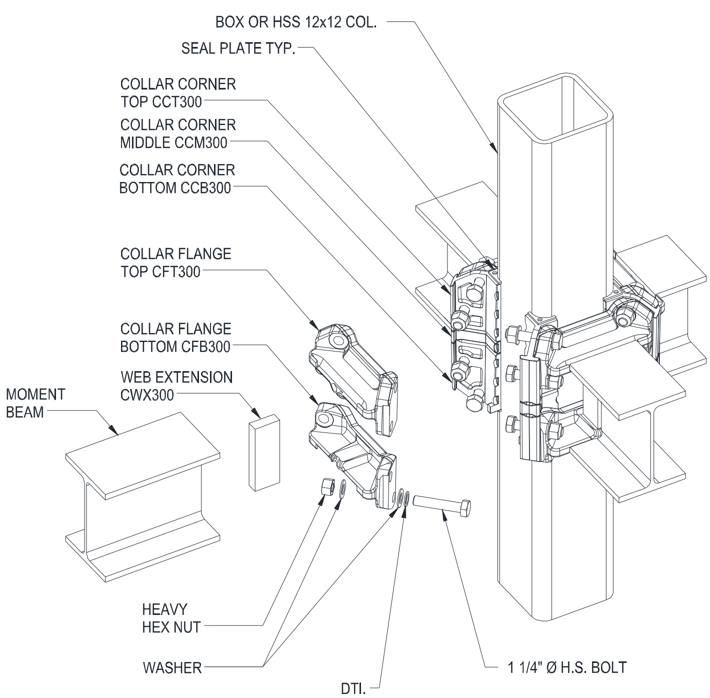


Figure 32: ConXL-300 Moment Connection Assembly.

The CCAs are drilled for through bolting and welded at the corners of the moment column at each floor level.

The CFA consists of a top and bottom Collar Flange, each welded to the top and bottom beam flange, and a Collar Web Extension (CWX), which is welded to the beam web and is welded to and spans between the top and bottom Collar Flanges.

The CWX comes in a variety of depths, while the top and bottom Collar Flanges are standardized modular units. The CFA can be assembled for a variety of beam sizes by adjusting the depth of the CWX.

The CFA is also drilled along its outside edges to receive Collar Bolts. Assembly in the field is accomplished by lowering the beam-CFA onto the column-CCA on each face of the column. The CCA has shear lugs at the bottom of the CCA (see Figure 32) that provide temporary vertical support for the beam and CFA until the bolts are pre-tensioned. Once the bolts are pretensioned, the friction due to the clamping force between the moment collar and the column provides vertical (upward or downward) carrying capacity for the beam.

Once beam-CFA assemblies have been located on each of the four faces of the column, high strength Collar Bolts are inserted into the holes in the adjoining CFAs and pretensioned, clamping the CFA to the CCA/column, forming a rigid collar assembly that surrounds the column.

The proceeding four pages illustrate the ConXL-300 connection in more detail. Standard details of the CFAs, CWX and CCAs are shown, as well as perspective images of beams fitted with the CFA.

2.2.2 Collar Flange Assembly

COLLAR FLANGE TOP (XL300-CFT)

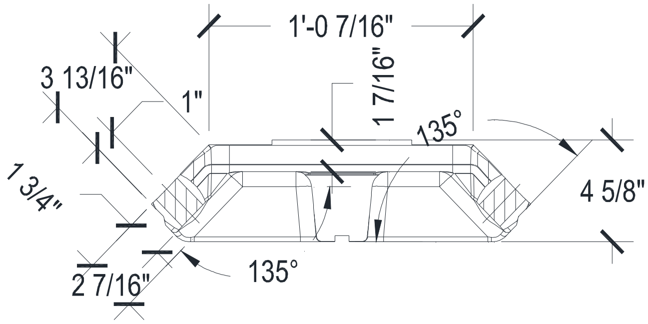


Figure 33: Top View of Collar Flange Top.

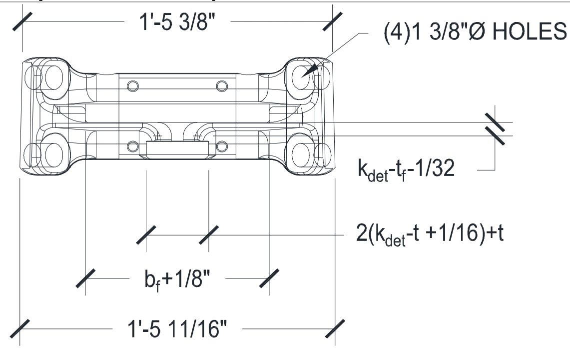


Figure 34: Front View of Collar Flange Top.

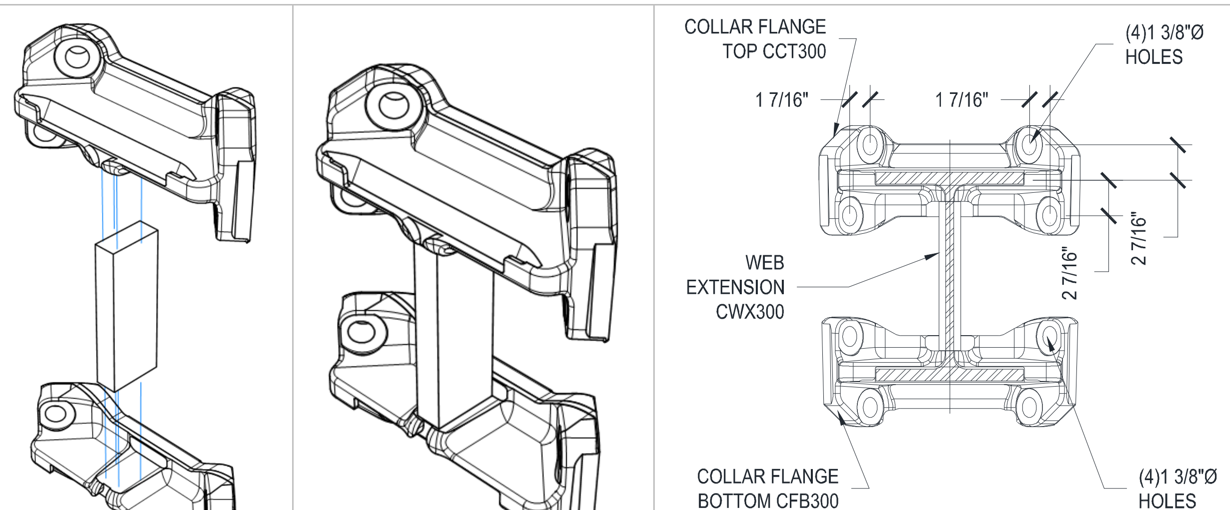


Figure 35: Collar Flange Assembly (XL300-CFA).

COLLAR FLANGE BOTTOM (XL300-CFB)

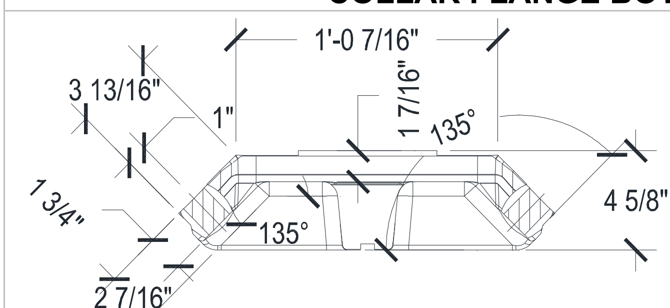


Figure 36: Top View of Collar Flange Bottom.

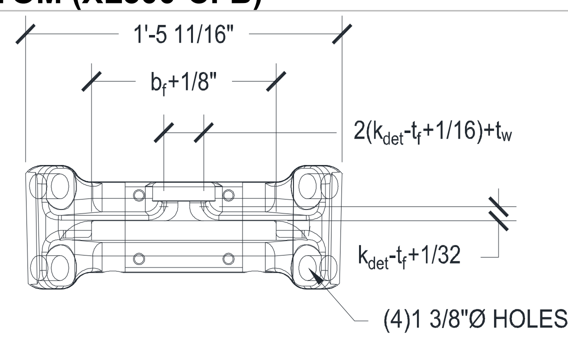


Figure 37: Front View of Collar Flange Bottom.

2.2.3 Beam-To-Collar Flange Assembly

The figures on this page illustrate the three types of beam configurations used in the ConXL-300 system.

The beam at the top is fitted with Collar Flanges on both ends. This beam configuration is referred to as a Beam-Moment-Moment (BMM).

The beam in the middle is fitted with a Collar Flange on one end. This beam is referred to as a Beam-Gravity-Moment (BGM) or a Beam-Moment-Gravity (BMG).

The beam on the bottom has no Collar Flanges on its ends and is referred to as a Beam-Gravity-Gravity (BGG).

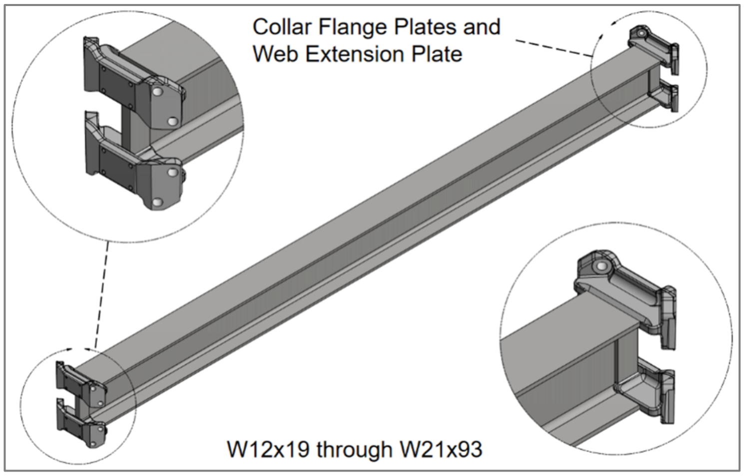


Figure 38: BMM.

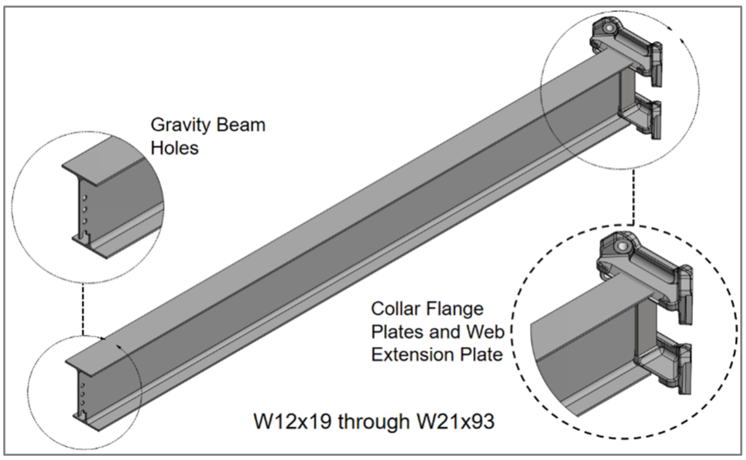


Figure 39: BGM.

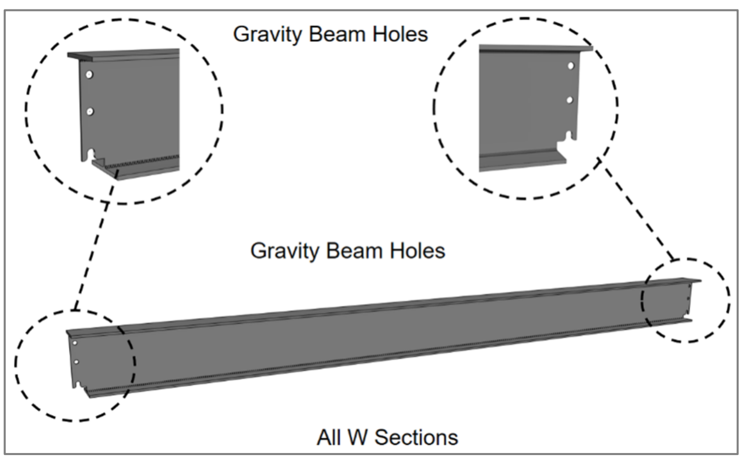


Figure 40: BGG.

2.2.4 Collar Corner Assembly

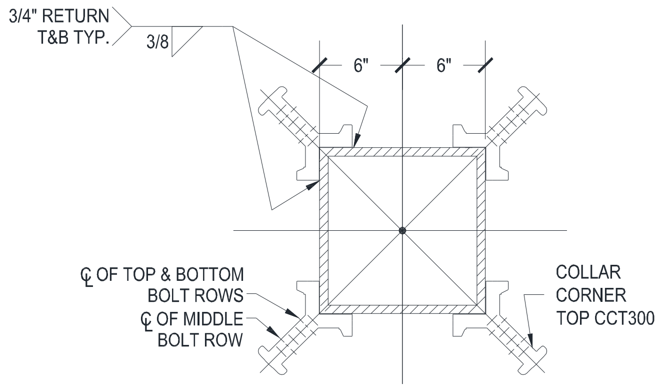


Figure 41: Plan View of ConXL-300 Collar Corners and Column.

The figures on this page show the plan, perspective and elevation views of the Collar Corner Assembly.

As can be seen, the Collar Corner Assemblies are attached to the moment column with fillet welds.

Note that the Collar Corner Middle is present only when the connected moment beam is deeper than a W18.

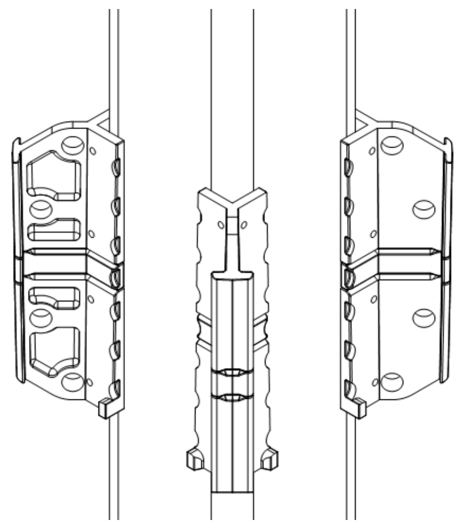


Figure 42: Perspective View of Collar Corner Assemblies welded onto a Column.

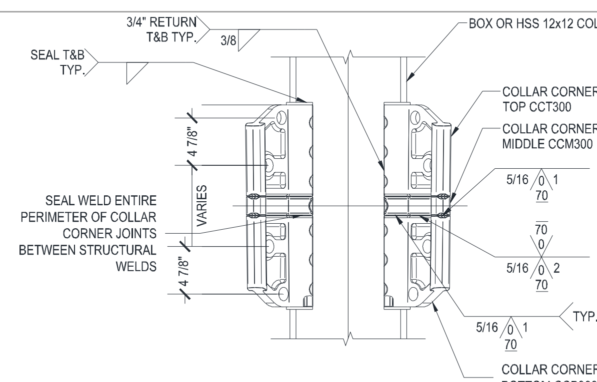


Figure 43: Standard Detail of Collar Corner Assemblies.

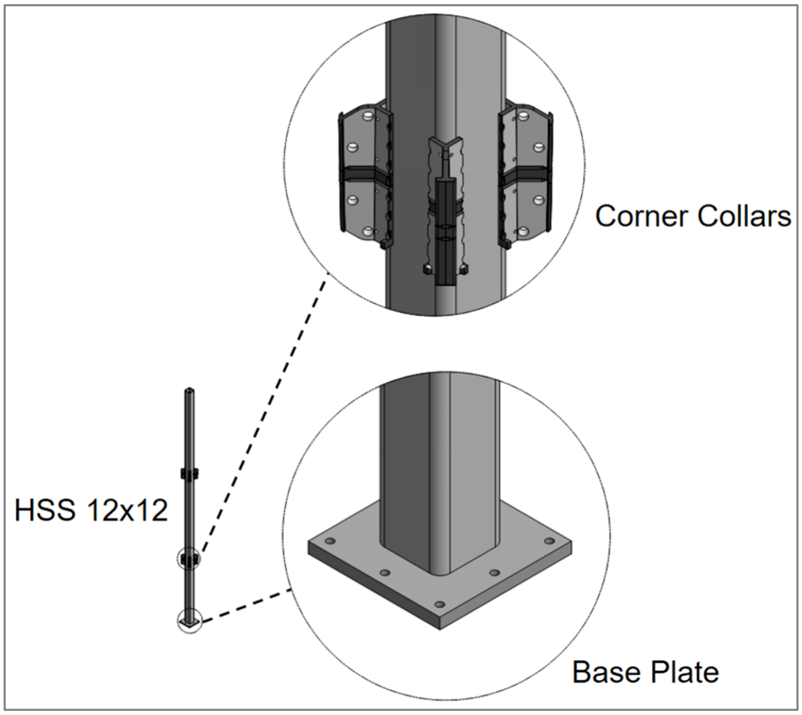


Figure 44: ConXL-300 Moment Column Assembly.

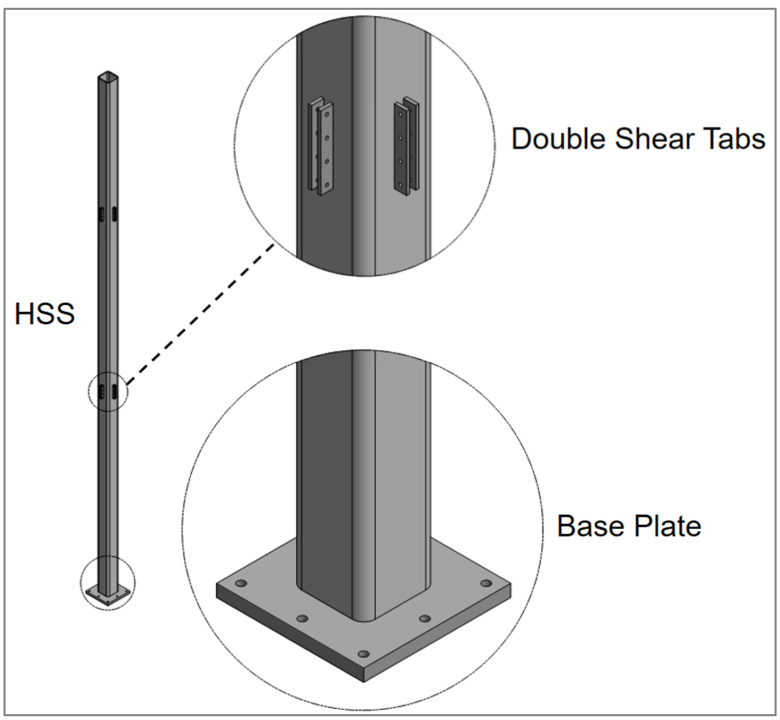


Figure 45: ConXL-300 Gravity Column Assembly.

2.2.5 Beam and Moment Collar Assembly

The figure below shows the plan and elevation standard details of the ConXL-300 moment connection.

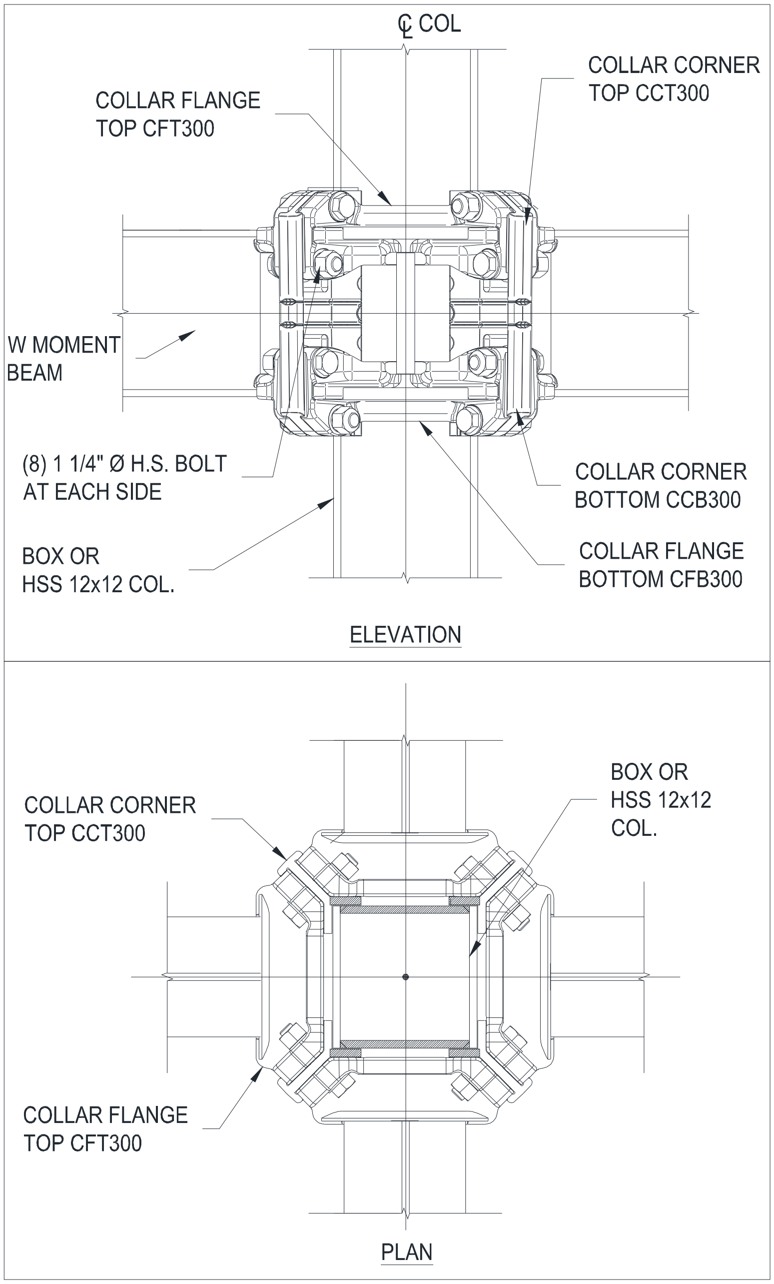


Figure 46: Elevation and Plan Views of the ConXL-300 Moment Connection.

2.2.6 Load Transfer Mechanism

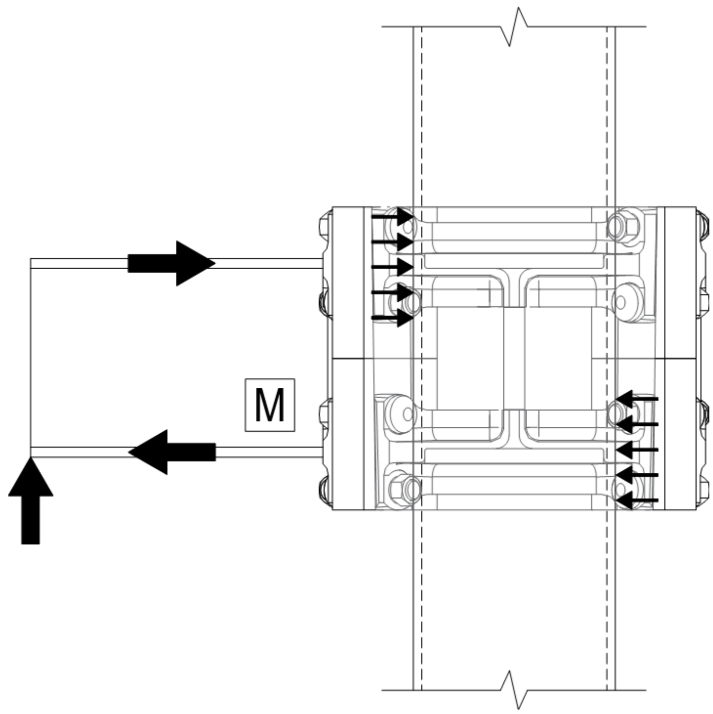


Figure 47: ConXL-300 Beam-To-Column Moment Transfer Mechanism.

Moment transfer between the beams and column is accomplished through direct compressive bearing as illustrated in Figure 47. Compressive flexural forces are transferred from the beam's flange to the Collar Flange through the CJP weld connecting them. The Collar Flange, acting as a beam spanning between the CCAs, delivers the compressive force through flexure to the CCAs via direct bearing. The CCAs delivers this bearing load to the column via fillet welds. Tensile flexural forces are transferred from the beam's flange to the Collar Flange through the CJP weld connecting them. The Collar Flange, acting as abeam spanning between Collar Bolts, delivers the tensile load force through flexure to Collar Bolts, which transfers this force in tension to the orthogonal CFAs, which transfers this tensile force to the far Collar Bolts, which transfers the tension force to the "rear" CFA. The "rear" CFA, acting as a beam spanning between CCAs, transfers the force in flexure to the CCAs via direct bearing. The CCAs deliver this bearing load to the column via fillet welds.

The CFA at the end of a moment beam is checked for required strengths per loading conditions. Through statics, these CFA loads are reduced to the loads subjected to the individual Collar Flange. These loads are then used to check the required strengths for the individual Collar Flange.

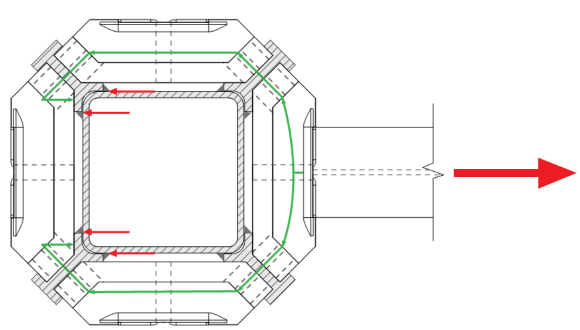


Figure 48: Load Path - Flange Tension Force to Column.

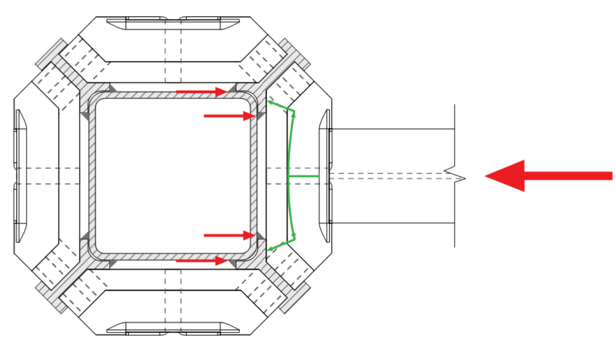


Figure 49: Load Path - Flange Compressive Force to Column.

2.2.7 Design Requirements

The design requirements detailed in this document for the ConXtech XL-300 moment connection pertain to the fundamental strength of the moment connection for non-seismic applications.

This document provides the design formulations for the following:

- Collar Flange Connection - Design and Allowable Flexural Strength.
- Bolted Connection - X-axis Design and Allowable Flexural Strength.
- Bolted Connection - Y-axis Design and Allowable Flexural Strength.
- Bolted Connection - Design and Allowable Strength for Combined Flexural, Shear, & Axial Load.
- Weld at Beam Web/Collar Web Extension - Design and Allowable Shear Strength.
- Weld at Collar Web Extension/Collar Flange - Design and Allowable Shear Strength.
- Weld at Collar Corner Assembly/Column - Design and Allowable Shear Strength.
- Collar Corner Assembly - Design and Allowable Shear Strength.
- Friction at Collar Flange/Collar Corner - Design and Allowable Slip Resistance Strength.
- Bearing at Collar Flange and Collar Corner - Design and Allowable Bearing Strength.
- Collar Corner Assembly - Design and Allowable Tensile Strength.
- Collar Corner Assembly - Design and Allowable Strength for Combined Shear Load.
- Panel Zone - Design and Allowable Shear Strength.
- Local Yielding of HSS/Box Column Sidewalls - Design and Allowable Strength.
- Local Crippling of HSS/Box Column Sidewalls - Design and Allowable Strength.

Connection Design Notes:
The connection design procedure presented below is based on the *Specification*.

Definitions:

CF = Collar Flange: one CF welded to each flange of beam.

CFC = Collar Flange Connection: Top and bottom Collar Flange working in conjunction with Collar Web Extension, connecting end of beam to column.

CWX = Collar Web Extension: welded to top and bottom Collar Flange; beam web is welded to CWX.

CC = Collar Corner: welded to column; top and bottom Collar Flanges are clamped into CC's via bolt pretension; surface of CC and CF in contact with each other are machined surfaces.

M_{ux} = LRFD Load Combinations Required Flexural Strength – Beam X-axis, *k-ft* (kN-m)

M_{uy} = LRFD Load Combinations Required Flexural Strength – Beam Y-axis, *k-ft* (kN-m)

P_{ub} = LRFD Load Combinations Required Axial Strength – Beam, *kips* (kN)

V_{ux} = LRFD Load Combinations Required Shear Strength – Beam X-axis, *kips* (kN)

V_{uy} = LRFD Load Combinations Required Shear Strength – Beam Y-axis, *kips* (kN)

P_{uc} = LRFD Load Combinations Required Axial Strength – Column, *kips* (kN)

V_{uPZ} = LRFD Load Combinations Required Shear Strength – Panel Zone, *kips* (kN)

w_u = LRFD Load Combinations Uniform Applied Load – Beam, *k/ft* (kN/m)

2.2.8 Flexural Strength of Collar Flange Connection

The design check treats the Collar Flange as a simply supported beam spanning between the bolts that connect it to the Collar Corners. The flanges of the moment beam are welded to the Collar Flange with a complete joint penetration weld; thus, the Collar Flange (spanning between the bolts) at the beam flange is considered restrained. Consequently, the location for critical shear/moment for the Collar Flange occurs just outside of the beam flange.

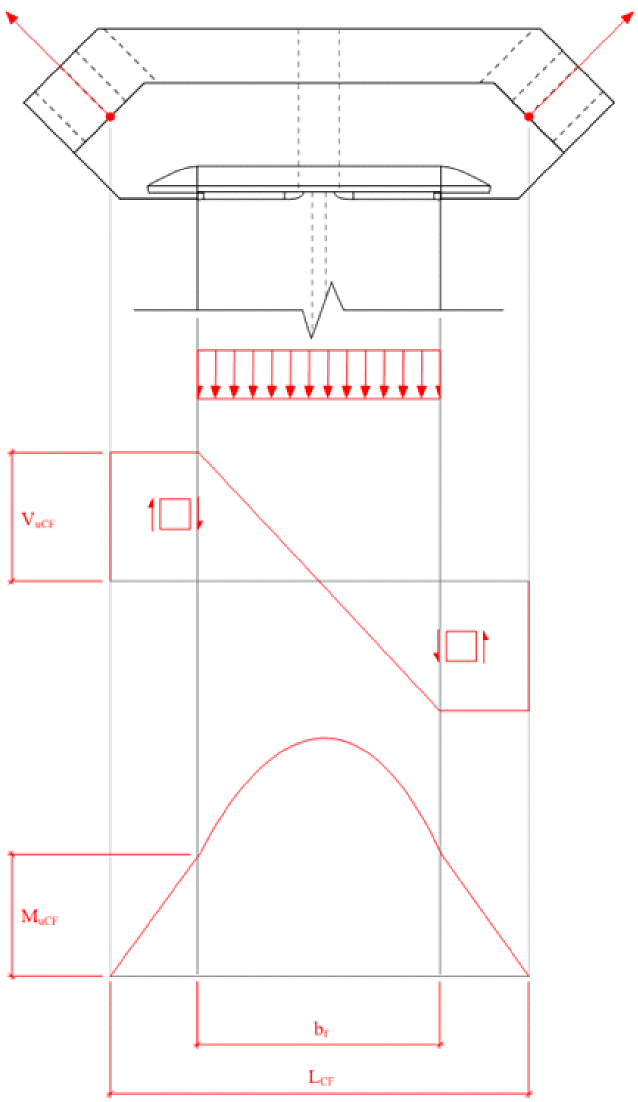


Figure 50: Shear and Moment Diagram of Collar Flange Under Loading.

V_{rCF} = Collar Flange req'd shear strength

M_{rCF} = Collar Flange req'd flex. strength

d = depth of beam

t_f = thickness of beam flange

L_{CF} = effective span of Collar Flange

Z_{CF} = plastic modulus of Collar Flange

A_{vCF} = shear area of Collar Flange

d_b = diameter of XL-300 bolt

d_b = 1.25 *in.* (31.8 mm)

F_{yCF} = yield stress of Collar steel (ASTM A572 Gr. 50)

F_{yCF} = 50 *ksi* (345 MPa)

As can be seen in the figure above, the flexural capacity of the Collar Flange is the smaller of either the flexural strength of the CF, $F_{yCF}Z_{CF}$, *or* the shear strength of the CF, V_{nCF}, multiplied by the distance $(L_{CF} - b_f)/2$.

The section properties of the Collar Flange are shown in Figure 51 below.

$$L_{CF300} = 12.4\ in. \quad (315\ \text{mm})$$

$$A_{vCF300} = 8.11\ in.^2 \quad (5,232\ \text{mm}^2)$$

$$Z_{CF300} = 16.42\ in.^3 \quad (269,076\ \text{mm}^3)$$

$$n_{b300} = \text{number of bolts per beam end} = 8$$

$$cb_{300} = \text{width of compression block} = 0.694\ in. \quad (17.6\ \text{mm})$$

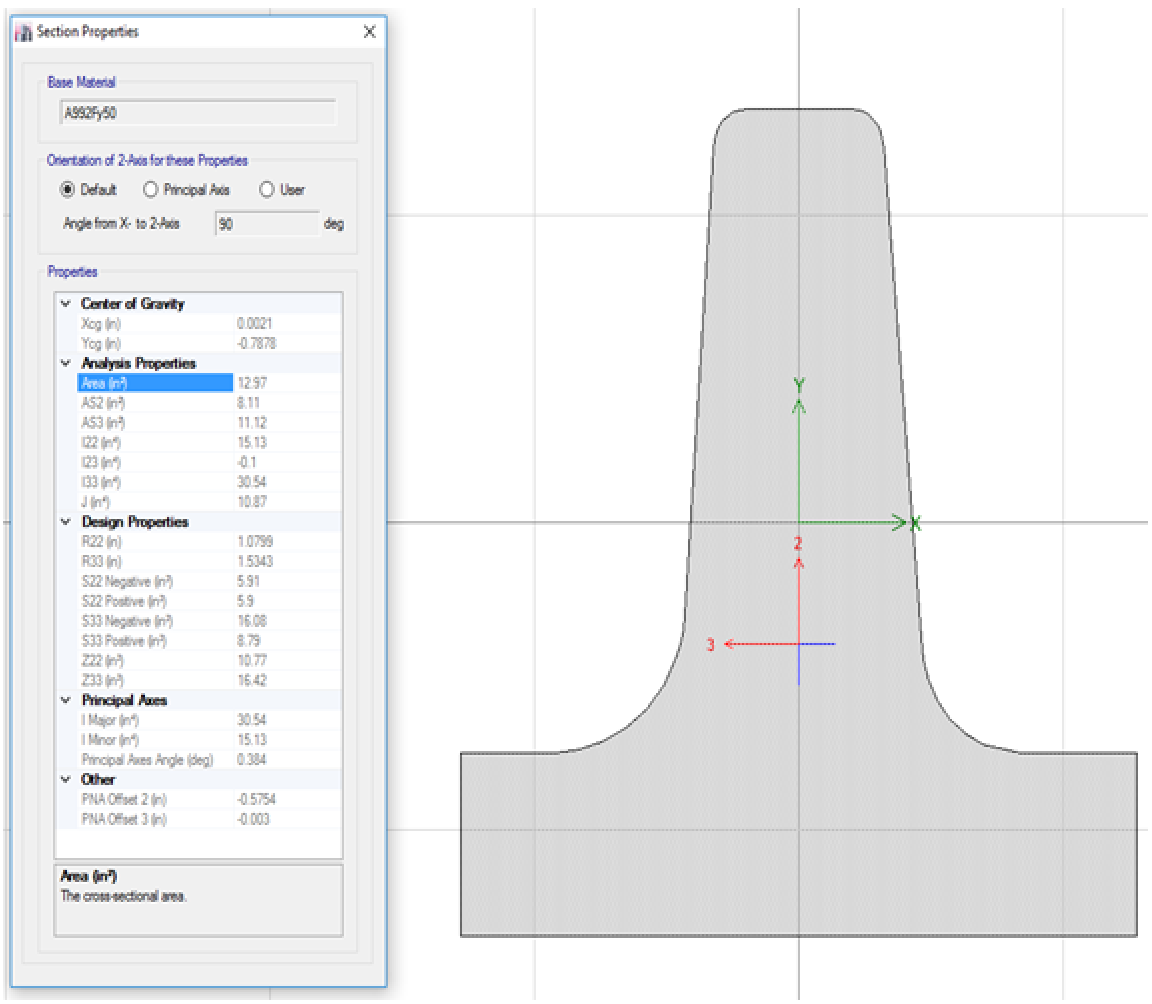


Figure 51: ConXL-300 Collar Flange Section Properties.

As explained previously on page 82, the flexural capacity of the Collar Flange is the smaller of either the flexural strength of the CF, $F_{yCF}Z_{CF}$, *or* the shear strength of the CF, V_{nCF}, multiplied by the distance $(L_{CF} - b_f)/2$.

The nominal flexural strength of the CF is $M_n = F_{yCF}Z_{CF}$. The *design* flexural strength (DFS) and the allowable flexural strength of the Collar Flange are calculated as shown below:

$$F_{yCF} = 50\ ksi \quad (345\text{ MPa})$$
$$Z_{CF} = 16.42\ in.^3 \quad (269{,}076\text{ mm}^3)$$

LRFD	ASD
$\phi M_{nCF} = \phi F_{yCF} Z_{CF}$ $= 0.90(50\ ksi)(16.42\ in.^3)$ $= 739\ kip-in.\ (83.5\text{ kN-m})$	$\frac{M_{nCF}}{\Omega} = \frac{Z_{CF}F_{yCF}}{\Omega}$ $= \frac{(50\ ksi)(16.42\ in.^3)}{1.67}$ $= 492\ kip-in.\quad (55.6\text{ kN-m})$

The nominal shear strength, V_{nCF}, of the Collar Flange is calculated below:

$$V_{nCF} = 0.60 F_{yCF} A_{vCF}$$

where

$$F_{yCF} = 50\ ksi \quad (345\text{ MPa})$$
$$A_{vCF} = 8.11\ in.^2 \quad (5{,}232\text{ mm}^2)$$

Therefore, the design and the allowable critical moment demand of the Collar Flange created by a shear load equal to the shear strength of the Collar Flange are as follows:

LRFD	ASD
$\phi M_{rCF} = \phi V_{nCF}\left(\frac{L_{CF} - b_f}{2}\right)$	$\frac{M_{rCF}}{\Omega} = \frac{V_{nCF}}{\Omega}\left(\frac{L_{CF} - b_f}{2}\right)$

where

$$L_{CF} = 12.4\ in. \quad (315\text{ mm})$$

The flexural capacity of the Collar Flange is determined by comparing the value of the CF's flexural strength to the value of the critical moment demand created by a shear load equal to the shear strength of the Collar Flange. If the critical moment demand due to the shear strength of the Collar Flange is smaller than the flexural strength of the Collar

Flange, then the Collar Flange shear strength governs; otherwise, the flexural strength of the Collar Flange governs.

The flexural capacity of the Collar Flange, can be expressed in a mathematical formula as shown below:

$$\text{Flexural Strength of Collar Flange} = \min\left(F_{yCF} Z_{CF}, V_{nCF}\left(\frac{L_{CF} - b_f}{2}\right)\right)$$

If the Collar Flange shear strength governs, the maximum beam moment that the connection can resist when the Collar Flange is the limiting component is as shown below:

LRFD	ASD
$\phi M_{nCFC} = \phi V_{nCF}(2)(d - t_f)$	$\frac{M_{nCFC}}{\Omega} = \frac{V_{nCF}}{\Omega}(2)(d - t_f)$

If the Collar Flange flexural strength governs, the maximum beam moment that the connection can resist when the Collar Flange is the limiting component is as shown below:

LRFD	ASD
$\phi M_{nCFC} = \frac{\phi M_{nCF}(2)(d - t_f)}{\left(\frac{L_{CF} - b_f}{2}\right)}$	$\frac{M_{nCFC}}{\Omega} = \frac{M_{nCF}(2)(d - t_f)}{\Omega\left(\frac{L_{CF} - b_f}{2}\right)}$

The Demand Capacity Ratio for the design and allowable Flexural Strength of the Collar Flange Connection is calculated as shown below:

	ASD
$DCR = \frac{M_{ux}}{\phi M_{nCFC}}$	$DCR = \frac{M_{ux}}{\left(\frac{M_{nCFC}}{\Omega}\right)}$

2.2.9 Bolted Connection: X-Axis Flexural Strength

As stated previously in this document, the ConXL-300 collar flanges transfer a beam's moment into the moment collar through pretensioned bolts. Because the size and number of bolts in the connection are a known constant, the maximum moment capacity of the connection can be determined in terms of the strength of the bolts.

A moment load in the strong-axis of the connected beam resolves into tension and compression forces within the beam's flange.

As can be seen in the figures below, the bolts are oriented 45 degrees relative to the longitudinal axis of the connected moment beam. Therefore, a tensile force from a beam's flange can be resolved into a shear force and a tension force in the bolt, as shown below.

$$f_r \cos\left(45^\circ\right) = f_v = f_t$$

$$f_r = \sqrt{f_t^2 + f_v^2} = \sqrt{f_t^2 + f_t^2} = \sqrt{2} f_t$$

The following section determines the maximum available bolt load, considering the effects of combined loading as stipulated in the *Specifications*.

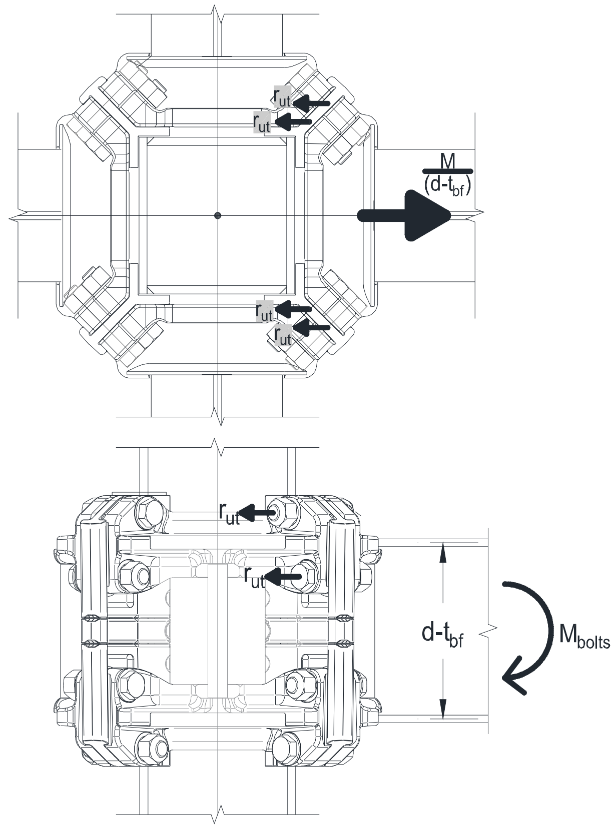


Figure 52: ConXL-300 Bolted Moment Connection Under Flexural Load.

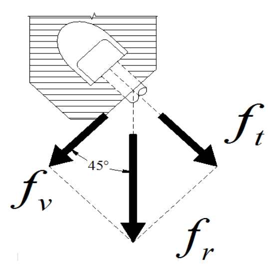


Figure 53: Bolt Load Resolved into a Tensile and Shear Force Components.

For the formulations that follow, the definitions below apply:

f_t = required tensile stress, *ksi* (MPa)
f_v = required shear stress, *ksi* (MPa)
f_t = f_v (bolts are 45° to applied load)
f_r = required resultant stress resisting applied load, *ksi* (MPa)
F_{nt} = nominal tensile stress = 0.75F_u, *ksi* (MPa) (*Specification* C-J3-2)
F_{nv} = nominal shear stress = 0.563F_u, *ksi* (MPa) (*Specification* C-J3-3)
F_u = specified minimum tensile strength of bolt, *ksi* (MPa)
ϕ_{bolts} = 0.75, resistance factor for bolts
Ω_{bolts} = 2.00, safety factor for bolts
R_n = nominal strength of bolt resisting applied load, *kips* (kN)
A_b = nominal unthreaded body area of bolt, *in.*^{2}

LRFD	ASD
Specification (C-J3-5a) $\left(\frac{f_t}{\phi_{bolts}F_{nt}}\right)^2+\left(\frac{f_v}{\phi_{bolts}F_{nv}}\right)^2=1$	*Specification* (C-J3-5b) $\left(\frac{\Omega_{bolts}f_t}{F_{nt}}\right)^2+\left(\frac{\Omega_{bolts}f_v}{F_{nv}}\right)^2=1$
$\left(\frac{f_t}{\phi_{bolts}0.75F_{nt}}\right)^2+\left(\frac{f_v}{\phi_{bolts}0.563F_{nv}}\right)^2=1$ $\left(\frac{f_t}{\phi_{bolts}0.75F_u}\right)^2+\left(\frac{f_t}{\phi_{bolts}0.563F_u}\right)^2=1$ $\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{1}{0.75^2}+\frac{1}{0.563^2}\right)=1$ $\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{0.563^2}{0.75^2\cdot 0.563^2}+\frac{0.75^2}{0.563^2\cdot 0.75^2}\right)=1$ $\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{0.563^2+0.75^2}{(0.563)^2(0.75)^2}\right)=1$ $f_t=\sqrt{\frac{(0.563)^2(0.75)^2}{\left(0.563^2+0.75^2\right)}\cdot\left(\phi_{bolts}F_u\right)^2}$ $f_t=0.45\left(\phi_{bolts}F_u\right)$	$\left(\frac{\Omega_{bolts}f_t}{0.75F_u}\right)^2+\left(\frac{\Omega_{bolts}f_v}{0.563F_u}\right)^2=1$ $\left(\frac{\Omega_{bolts}f_t}{0.75F_u}\right)^2+\left(\frac{\Omega_{bolts}f_t}{0.563F_u}\right)^2=1$ $\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{1}{0.75^2}+\frac{1}{0.563^2}\right)=1$ $\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{0.563^2}{0.75^2\cdot 0.563^2}+\frac{0.75^2}{0.563^2\cdot 0.75^2}\right)=1$ $\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{0.563^2+0.75^2}{(0.563)^2(0.75)^2}\right)=1$ $f_t=\sqrt{\frac{(0.563)^2(0.75)^2}{\left(0.563^2+0.75^2\right)}\cdot\left(\frac{F_u}{\Omega_{bolts}}\right)^2}$ $f_t=0.45\left(\frac{F_u}{\Omega_{bolts}}\right)$

The nominal tensile capacity of a bolt is shown below:

$$R_n = f_r A_b = \sqrt{2} f_t A_b = \sqrt{2}(0.45) F_u A_b = 0.636 F_u A_b$$

The design tension and the allowable tension of a bolt is determined as shown below:

LRFD	ASD
$\phi_{bolts} = 0.75$ $\phi_{bolts} R_n = \phi_{bolts} f_r A_b = \phi_{bolts} \sqrt{2} f_t A_b$ $\phi_{bolts} R_n = (0.75)\sqrt{2}(0.45) F_u A_b = 0.477 F_u A_b$	$\Omega_{bolts} = 2.00$ $\dfrac{R_n}{\Omega_{bolts}} = \dfrac{f_r A_b}{\Omega_{bolts}} = \dfrac{\sqrt{2} f_t A_b}{\Omega_{bolts}}$ $\dfrac{R_n}{\Omega_{bolts}} = \dfrac{\sqrt{2}(0.45) F_u A_b}{2.00} = 0.318 F_u A_b$

The nominal flexural strength of the bolted moment connection is:

$$M_{ntx} = 0.636 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$$

where

$F_u =$ specified minimum tensile strength of bolt, *ksi* (MPa)
$n_b =$ number of bolts in connection = 8
$A_b =$ nominal unthreaded body area of bolt = 1.227 *in*^{2} (791.6 mm^2); diameter of bolt is 1.25 *in* (31.8 mm)

Therefore, the design and allowable flexural strength of the bolted connection is as shown below:

LRFD	ASD
$\phi_{bolts} M_{ntx} = \phi_{bolts} 0.636 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$ $\phi_{bolts} M_{ntx} = 0.477 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$	$\dfrac{M_{ntx}}{\Omega_{bolts}} = \dfrac{0.636 F_u A_b}{\Omega_{bolts}} \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$ $\dfrac{M_{ntx}}{\Omega_{bolts}} = 0.318 F_u A_b \left(\frac{n_b}{2}\right)\left(d - t_{bf}\right)$

It follows that the Demand Capacity Ratio for the design and allowable flexural strength of the bolted connection is as shown below:

LRFD	ASD
$DCR = \dfrac{M_{ux}}{\phi_{bolts} M_{ntx}}$	$DCR = \dfrac{M_{ux}}{\left(M_{ntx} / \Omega_{bolts} \right)}$

2.2.10 Bolted Connection: Y-Axis Flexural Strength

This section discusses the weak-axis flexural mechanics of the ConXL-300 collar connection. The weak-axis flexural capacity of the bolted connection will be determined.

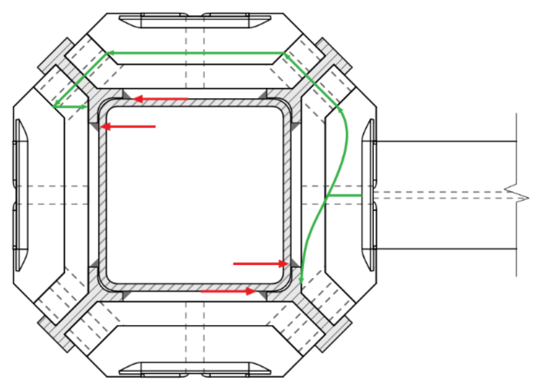


Figure 54: Y-Axis Bending Moment - Load Path Through Collars and Bolts to Column.

A moment load in a connected beam's weak-axis causes flexure in the two Collar Flanges attached to the beam, as depicted in Figure 54. The flexural load in the Collar Flanges causes tension on one side of the Collar Flanges and compression on the other side of the Collar Flanges. The tensile force in the Collar Flanges is transferred into the Collar Corners through the pretensioned bolts.

The compressive force causes bearing pressure between the Collar Flanges and the Collar Corners, as illustrated in Figure 54,

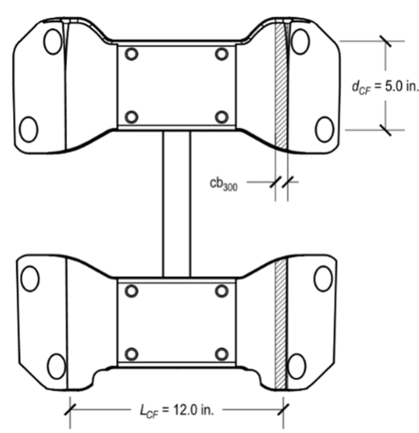


Figure 55: Cross-Section View of a ConXL-300 Collar Flange in Weak-Axis Flexure.

Where

d_{CF} = effective depth of Collar Flange for Y-Axis bolt moment
L_{CF} = effective length of Collar Flange for Y-Axis bolt moment
cb_{300} = width of compression block for Y-Axis bolt moment

The width of the compression block in the Collar Flange is calculated by setting the bolt strength equal to the collar bearing strength. The ConXL-300 system can accept High Strength or Super High Strength bolts. The depth of the compression block in the CF is a function of the bolt strength. The stronger the bolts are, the longer the depth of the compression block, and therefore, the smaller the moment arm. Super High Strength bolts have a specified minimum tensile strength of 200 *ksi*. Thus, in the calculations that follow, an ultimate strength F_u of 200 *ksi* (1,379 MPa) is assumed for the bolts because this is conservative in determining the Y-Axis flexural capacity of the bolted connection.

Calculations of the width of the collar compression block are shown below. The bolt tension load is as follows:

LRFD	ASD
$F_u = 200\ ksi\ \ (1{,}379\ \text{MPa})$ $A_b = 1.227\ in.^2\ \ (791.6\ \text{mm}^2)$ $n_b = 8$ $\phi_{bolts} R_n = 0.477 F_u A_b \left(\frac{n_b}{2}\right)$ $= 0.477(200\ ksi)(1.227\ in.^2)\left(\frac{8}{2}\right)$ $= 468.2\ kips\ \ (2{,}083\ \text{kN})$	$F_u = 200\ ksi\ \ (1{,}379\ \text{MPa})$ $A_b = 1.227\ in.^2\ \ (791.6\ \text{mm}^2)$ $n_b = 8$ $\frac{R_n}{\Omega_{bolts}} = 0.318 F_u A_b \left(\frac{n_b}{2}\right)$ $= 0.318(200\ ksi)(1.227\ in.^2)\left(\frac{8}{2}\right)$ $= 312.1\ kips\ \ (1{,}388\ \text{kN})$

In accordance with *Specifications* Chapter J7, the nominal bearing strength, R_n, for finished (milled) surfaces is as shown below:

$$R_n = 1.8 F_y A_{pb} \qquad \text{(J7-1)}$$

$$\phi = 0.75\ \ (\text{LRFD}) \qquad \Omega = 2.00\ \ (\text{ASD})$$

where

F_y = specified minimum yield stress, *ksi* (MPa)
A_{pb} = projected bearing area, *in.*^{2} (mm^2)

The width of the compression block in the collar is determined by setting the bolt tension load equal to the bearing strength of the collar, as shown on the next page.

The calculations below consider both the top and the bottom Collar Flanges, and thus, the projected bearing area is twice the bearing area of one Collar Flange.

LRFD	ASD
$F_y = 50\ ksi\ \left(345\text{ MPa}\right)$ $d_{CF} = 5.0\ in.\ \left(127\text{ mm}\right)$ $A_{pb} = 2d_{CF}cb_{300}$ $\phi R_n = \phi 1.8F_y A_{pb} = 468.2\ kips$ $0.75\left(1.8\left(50\ ksi\right)2\left(5.0\ in.\right)cb_{300}\right) = 468.2\ kips$ $cb_{300} = 0.694\ in.\ \left(17.6\text{ mm}\right)$	$F_y = 50\ ksi\ \left(345\text{ MPa}\right)$ $d_{CF} = 5.0\ in.\ \left(127\text{ mm}\right)$ $A_{pb} = 2d_{CF}cb_{300}$ $\dfrac{R_n}{\Omega} = \dfrac{1.8F_y A_{pb}}{\Omega} = 312.1\ kips$ $1.8\left(50\ ksi\right)2\left(5.0\ in.\right)cb_{300} = 2.00\left(312.1\ kips\right)$ $cb_{300} = 0.694\ in.\ \left(17.6\text{ mm}\right)$

Now that the depth of the compression stress block has been determined, the Y-Axis flexural strength of the bolted connection can be calculated. Using d_c to denote the depth of the column, the design or allowable Y-Axis flexural strength of the bolted connection is as shown below:

LRFD	ASD
$d_c = 12\ in.\ \left(304.8\text{ mm}\right)$ $cb_{300} = 0.694\ in.\ \left(17.6\text{ mm}\right)$ $\phi_{bolts}M_{nty} = 0.477F_u A_b\left(\dfrac{n_b}{2}\right)\left(\dfrac{L_{CF}}{2} + \dfrac{d_C}{2} - \dfrac{cb_{300}}{2}\right)$	$d_c = 12\ in.\ \left(304.8\text{ mm}\right)$ $cb_{300} = 0.694\ in.\ \left(17.6\text{ mm}\right)$ $\dfrac{M_{nty}}{\Omega_{bolts}} = 0.318F_u A_b\left(\dfrac{n_b}{2}\right)\left(\dfrac{L_{CF}}{2} + \dfrac{d_C}{2} - \dfrac{cb_{300}}{2}\right)$

The Demand Capacity Ratio for the design or allowable Y-Axis flexural strength of the bolted connection is as shown below:

LRFD	ASD
$DCR = \dfrac{M_{uy}}{\phi_{bolts}M_{nty}}$	$DCR = \dfrac{M_{uy}}{\left(M_{nty}/\Omega_{bolts}\right)}$

2.2.11 Bolted Connection: Combined Flexural, Shear and Axial Loading

Where there is a combined flexural (in both X and Y axes), shear, and axial load on the bolted moment connection, the Demand Capacity Ratio is determined as shown below:

LRFD
$DCR = \dfrac{M_{ux}}{\phi_{bolts} M_{ntx}} + \left(\dfrac{V_{ux}}{\phi R_{ncf}}\right)^2 + \dfrac{M_{uy}}{\phi_{bolts} M_{nty}} + \dfrac{P_{ub}}{n_b \phi_{bolts} R_n}$
ASD
$DCR = \dfrac{M_{ux}}{M_{ntx}/\Omega_{bolts}} + \left(\dfrac{V_{ux}}{R_{ncf}/\Omega}\right)^2 + \dfrac{M_{uy}}{M_{nty}/\Omega_{bolts}} + \dfrac{P_{ub}}{n_b R_n/\Omega_{bolts}}$

where

V_{ux} = required strong-axis beam shear load from load combinations.
P_{ub} = required beam axial load from load combinations.
R_{ncf} = available slip resistance from Section 2.2.17.

2.2.12 X-Axis Shear Transfer from Beam to Moment Collar

In the ConXL-300 moment collar connection, shear is transferred to the Collar Flange Assembly via fillet welds between the beam web and the Collar Web Extension (CWX). The Collar Web Extension transfers the shear load via fillet welds between the top and bottom Collar Flanges (CFs) and the Collar Web Extension. This shear load is transferred to the Collar Corners (CC) through friction via direct bearing between machined surfaces of the top and bottom Collar Flanges and the Collar Corners clamped together by pretensioned bolts. The Collar Corners transfers the shear load to the column via fillet welds between the Collar Corners and the column.

The strength of the aforementioned welds is calculated in accordance with Chapter J2.4 of the *Specifications*, as follows:

$$R_n = F_{nw} A_{we} \qquad \text{(J2-4)}$$

$$\phi = 0.75 \ \ (\text{LRFD}) \qquad \Omega = 2.00 \ \ (\text{ASD})$$

For fillet welds,

$$F_{nw} = 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5}(\theta)\right) \qquad \text{(J2-5)}$$

where

F_{EXX} = filler metal classification strength, *ksi* (MPa)

θ = angle of loading measured from the weld longitudinal axis, degrees

A_{we} = effective area of the weld, *in.*^{2} (mm^2)

The proceeding pages show calculations to determine the strength of the beam web to Collar Web Extension welds, Collar Web Extension to Collar Flange welds and Collar Corners to Column welds.

2.2.13 Weld at Beam Web to Collar Web Extension

The shear strength of the fillet welds between the Beam Web and the Collar Web Extension (CWX) is determined as follows:

The ConXL-300 *beam web* to Collar Web Extension *(*CWX*)* connection uses a 5/16 *in*. (7.94 mm) fillet weld on each side of the beam web, as shown in Figure 56.

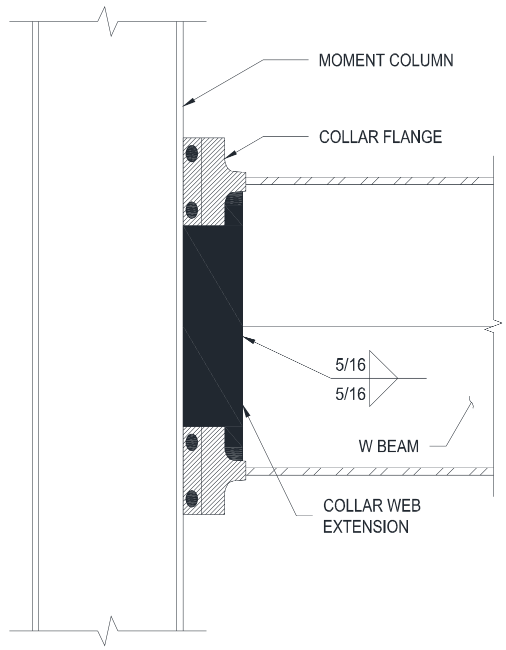


Figure 56: Beam Web-To-Collar Web Extension Welds.

The length of the fillet weld is:

$$l_{wb} = 2(d - 2k)\ in.\ (\mathrm{mm})$$

The size of the fillet weld is:

$$t_{we} = \frac{\sqrt{2}}{2} \cdot \frac{5}{16}\ in. = 0.221\ in.$$

The area of the fillet weld is:

$$A_{we} = t_{we} l_{wb}$$

The strength of the fillet weld is:

$$\theta = 0^\circ$$

$$F_{EXX} = 70\ ksi\ \left(483\ \text{MPa}\right)$$

$$R_n = 0.60 F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right)A_{we}$$

$$= 0.60(70\ ksi)\left(1.0 + 0.50\sin^{1.5}(0^\circ)\right)\left(\frac{\sqrt{2}}{2}\cdot\frac{5}{16}\,in.\right)2(d-2k)$$

$$= 0.60(70\ ksi)(1.0)\left(\frac{\sqrt{2}}{2}\cdot\frac{5}{16}\,in.\right)2(d-2k)$$

$$= \left(18.56\,\frac{kip}{in.}\right)(d-2k)$$

The design and allowable shear strength of the Beam Web to Collar Web Extension fillet welds is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(18.56\,\frac{kip}{in.}\right)(d-2k)$	$\frac{R_n}{\Omega} = \frac{\left(18.56\,\frac{kip}{in.}\right)(d-2k)}{2.00}$
ENGLISH	ENGLISH
$\phi R_n = \left(13.92\,\frac{kip}{in.}\right)(d-2k)$	$\frac{R_n}{\Omega} = \left(9.28\,\frac{kip}{in.}\right)(d-2k)$
METRIC	METRIC
$\phi R_n = \left(2.438\,\frac{\text{kN}}{\text{mm}}\right)(d-2k)$	$\frac{R_n}{\Omega} = \left(1.625\,\frac{\text{kN}}{\text{mm}}\right)(d-2k)$

The Demand Capacity Ratio (DCR) for the design and allowable shear strength of the Beam Web to Collar Web Extension (CWX) fillet welds is as shown below:

LRFD	ASD
$DCR = \frac{V_{ux}}{\phi R_n}$	$DCR = \frac{V_{ux}}{\left(R_n/\Omega\right)}$

2.2.14 Weld at Collar Web Extension to Collar Flanges

The ConXL-300 Collar Web Extension (CWX) to Collar Flange connections use 5/16 *in*. (7.94 mm) fillet welds. As can be seen in the figure on the right, the weld consists of both a parallel and a perpendicular part relative to the direction of the beam shear load.

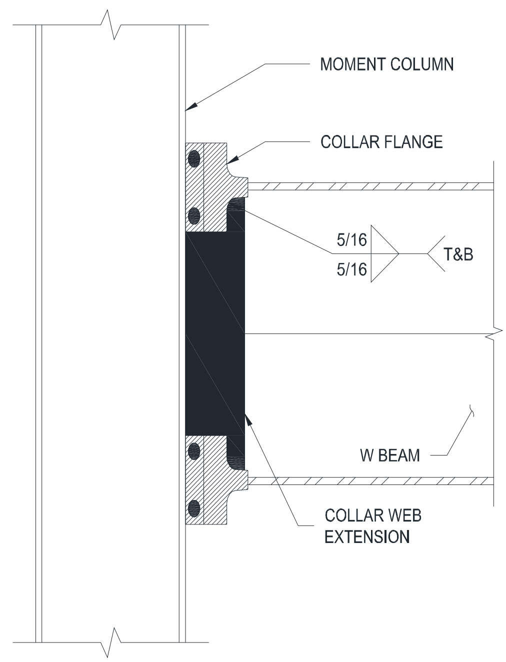


Figure 57: CWX-To-Collar Flange Welds.

The size of the fillet weld is:

$$t_{we} = \frac{\sqrt{2}}{2} \cdot \frac{5}{16}\, in. = 0.221\, in.$$

The total length and area of the perpendicular fillet welds is:

$$l_{wb_\perp} = 4(4.375\, in.) = 17.5\, in.$$

$$A_{we_\perp} = t_{we} l_{wb_\perp} = (0.221\, in.)(17.5\, in) = 3.87\, in.^2$$

The total length and area of the parallel fillet welds is:

$$l_{wb_\parallel} = 4(1.0\, in.) = 4.0\, in.$$

$$A_{we_\parallel} = t_{we} l_{wb_\parallel} = (0.221\, in.)(4.0\, in) = 0.884\, in.^2$$

The strength of the weld is the sum of the strengths of the parallel and the perpendicular welds. It is calculated below:

$$\theta_\parallel = 0^\circ$$

$$F_{EXX} = 70\, ksi \;\; (483\text{ MPa})$$

$$\theta_\perp = 90^\circ$$

$$R_n = 0.60 F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta_\parallel)\right) A_{we\parallel} + 0.60 F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta_\perp)\right) A_{we\perp}$$

$$R_n = 0.60(70\, ksi)\left(1.0 + 0.50\sin^{1.5}(0^\circ)\right)(0.884\, in.^2)\ldots$$
$$+\, 0.60(70\, ksi)\left(1.0 + 0.50\sin^{1.5}(90^\circ)\right)(3.87\, in.^2)$$

$$R_n = 0.60(70\, ksi)(0.884\, in.^2) + 0.60(70\, ksi)(1.5)(3.87\, in.^2)$$

$$R_n = 280.9\, kips$$

The design and allowable shear strength of the Collar Web Extension to Collar Flange fillet welds is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(280.9\ kips)$ $\phi R_n = 210.7\ kips\ \ (937.2\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{280.9\ kips}{2.00}$ $\dfrac{R_n}{\Omega} = 140.5\ kips\ \ (624.9\ \text{kN})$

The Demand Capacity Ratio for design and allowable shear strength of the Collar Web Extension (CWX) to Collar Flange fillet welds is:

LRFD	ASD
$\text{DCR} = \dfrac{V_{ux}}{\phi R_n}$ ENGLISH $\text{DCR} = \dfrac{V_{ux}}{210.7\ kips}$ METRIC $\text{DCR} = \dfrac{V_{ux}}{937.2\ \text{kN}}$	$\text{DCR} = \dfrac{V_{ux}}{\left(R_n / \Omega \right)}$ ENGLISH $\text{DCR} = \dfrac{V_{ux}}{140.5\ kips}$ METRIC $\text{DCR} = \dfrac{V_{ux}}{624.9\ \text{kN}}$

2.2.15 Weld at Collar Corner Assembly and Column (X-Axis Shear)

The ConXL-300 Collar Corner Assembly to column connection is a 3/8 *in*. (9.53 mm) fillet weld. As can be seen in the figure below, the weld is parallel to the direction of the beam shear load.

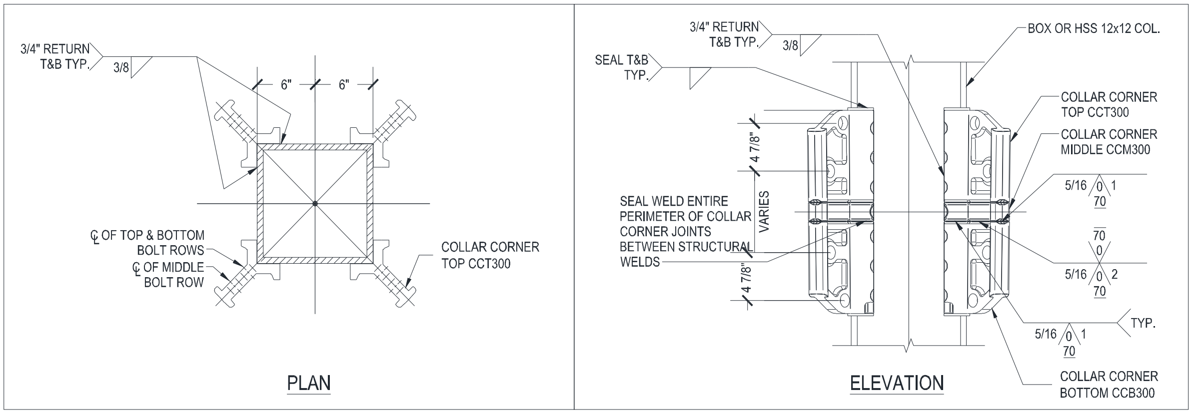


Figure 58: Collar Corner Assembly-To-Column Welds.

Using d_{nom} to denote the nominal depth of the connected beam, the total length of the fillet weld is:

$$l_{wc} = 2\left(d_{nom} + 4.5\,in.\right)$$

The size of the fillet weld is:

$$t_{wc} = \frac{\sqrt{2}}{2} \cdot \frac{3}{8}\,in. = 0.265\,in.$$

The area of the fillet weld is:

$$A_{wc} = t_{wc} l_{wc} = \left(0.265\,in.\right) 2\left(d_{nom} + 4.5\,in.\right)$$

The strength of the fillet weld is:

$$\theta = 0^{\circ}$$

$$\begin{aligned} R_n &= 0.60 F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right) A_{we} \\ &= 0.60\left(70\,ksi\right)\left(1.0 + 0.50\sin^{1.5}\left(0^{\circ}\right)\right)\left(0.53\,in.\right)\left(d_{nom} + 4.5\,in.\right) \\ &= 0.60\left(70\,ksi\right)\left(1.0 + 0\right)\left(0.53\,in.\right)\left(d_{nom} + 4.5\,in.\right) \end{aligned}$$

$$R_n = 22.3\,\frac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)$$

The design and allowable shear strength of the Collar Corner to column fillet welds is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(22.3\frac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)\right)$ ENGLISH $\phi R_n = 16.7\frac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)$ METRIC $\phi R_n = 2.92\frac{\text{kN}}{\text{mm}}\left(d_{nom} + 114.3\text{ mm}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{22.3\frac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)}{2.00}$ ENGLISH $\frac{R_n}{\Omega} = 11.1\frac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)$ METRIC $\phi R_n = 1.94\frac{\text{kN}}{\text{mm}}\left(d_{nom} + 114.3\text{ mm}\right)$

The Demand Capacity Ratio for design and allowable shear strength of the Collar Corner to column fillet welds is:

LRFD	ASD
ENGLISH $\text{DCR} = \frac{V_{ux}}{\phi R_n} = \frac{V_{ux}}{16.7\frac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)}$ METRIC $\text{DCR} = \frac{V_{ux}}{\phi R_n} = \frac{V_{ux}}{2.92\frac{\text{kN}}{\text{mm}}\left(d_{nom} + 114.3\text{ mm}\right)}$	ENGLISH $\text{DCR} = \frac{V_{ux}}{\left(R_n/\Omega\right)} = \frac{V_{ux}}{11.1\frac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)}$ METRIC $\text{DCR} = \frac{V_{ux}}{\phi R_n} = \frac{V_{ux}}{1.94\frac{\text{kN}}{\text{mm}}\left(d_{nom} + 114.3\text{ mm}\right)}$

2.2.16 Shear Yielding and Shear Rupture in Collar Corner Assemblies (X-Axis Shear)

In accordance with *Specification* Chapter J4.2, the available shear strength of the affected and connecting elements shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

a. For shear yielding of the element:

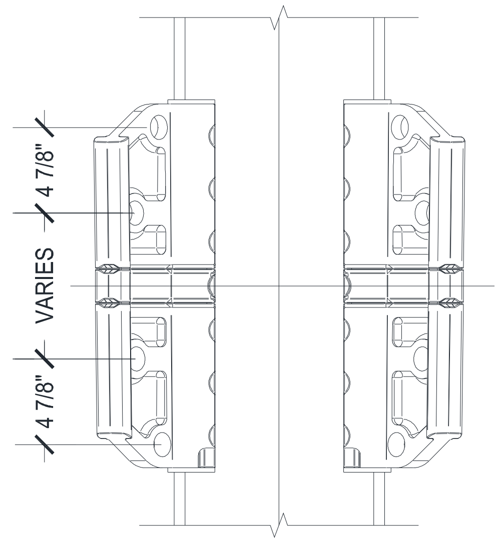
$$R_n = 0.60 F_y A_{gv} \quad \text{(J4-3)}$$

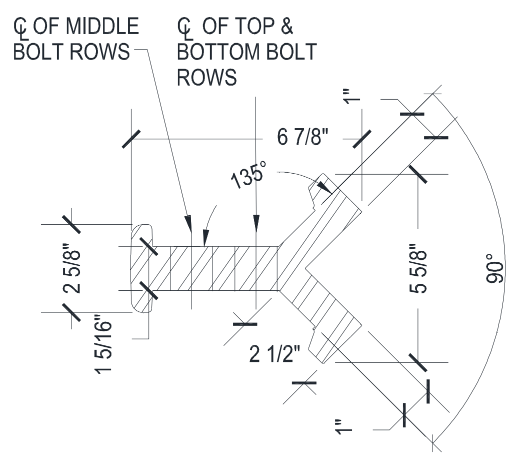
$\phi = 1.00$ (LRFD) $\qquad \Omega = 1.50$ (ASD)

b. For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \quad \text{(J4-4)}$$

$\phi = 0.75$ (LRFD) $\qquad \Omega = 2.00$ (ASD)



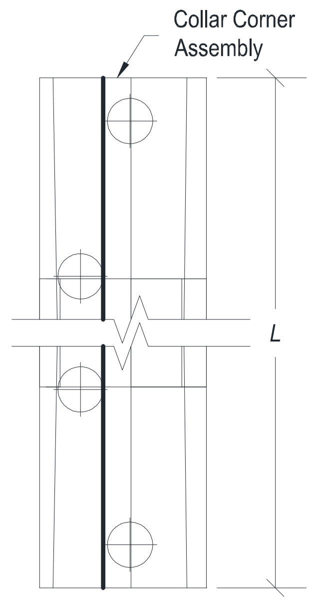


The Collar Corner Assemblies are made from ASTM A572 Grade 50 material. This material has the following specifications:

$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*

As can be seen in the figure directly on the right, the thickness of the CCA is 1-5/16 *in*. (33.3 mm).

The thickened vertical line in the figure on the right shows the shear plane of the gross section. The length of the shear plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism.



For shear yielding in the gross-section:

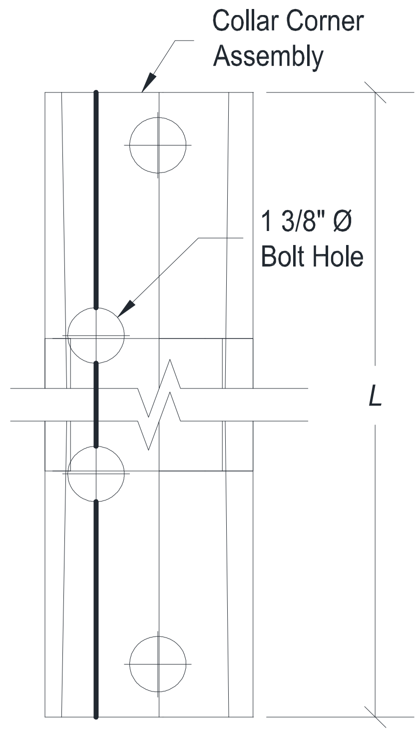
$$A_{gv} = \left(1\frac{5}{16}\,in.\right)L = \left(1\frac{5}{16}\,in.\right)(d_{nom})$$

$$\begin{aligned} R_n &= 0.60F_yA_{gv}\ x\ 2\ \text{CCAs} \\ &= 0.60(50\ ksi)\left(1\frac{5}{16}\,in.\right)(d_{nom})x2 \\ &= \left(78.8\,\frac{kip}{in.}\right)d_{nom} \end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00\left(78.8\,\frac{kip}{in.}\right)d_{nom}$ ENGLISH $\phi R_n = \left(78.8\,\frac{kip}{in.}\right)d_{nom}$ METRIC $\phi R_n = \left(13.8\,\frac{\text{kN}}{\text{mm}}\right)d_{nom}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{\left(78.8\,\frac{kip}{in.}\right)d_{nom}}{1.50}$ ENGLISH $\frac{R_n}{\Omega} = \left(52.5\,\frac{kip}{in.}\right)d_{nom}$ METRIC $\frac{R_n}{\Omega} = \left(9.19\,\frac{\text{kN}}{\text{mm}}\right)d_{nom}$

The thickened vertical lines in the figure on the right show the shear plane of the net section. In conformance with *Specification* Chapter B4.3b, in computing the net area for tension and shear, the width of a bolt hole shall be taken as 1/16 *in*. (2 mm) greater than the nominal dimension of the hole.



The length of the shear plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism.

For shear rupture in the net-section:

$$A_{nv} = \left(L - 2\left(d_h + \frac{1}{16}\,in. \right) \right)\left(1\frac{5}{16}\,in. \right)$$

$$= \left(d_{nom} - 2\left(1\frac{3}{8}\,in. + \frac{1}{16}\,in. \right) \right)\left(1\frac{5}{16}\,in. \right)$$

$$= \left(d_{nom} - 2.875\,in. \right) 1.3125\,in.$$

$$R_n = 0.60 F_u A_{nv}$$

$$= 0.60\left(65\,ksi\right)\left(\left(d_{nom} - 2.875\,in.\right)1.3125\,in.\right)\,x\,2\,\mathrm{CCAs}$$

$$= 102.4\,\frac{kips}{in.}\left(d_{nom} - 2.875\,in.\right)$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(102.4\,\frac{kips}{in.}\left(d_{nom} - 2.875\,in.\right)\right)$ ENGLISH $\phi R_n = \left(76.8\,\frac{kip}{in.}\right)\left(d_{nom} - 2.875\,in.\right)$ METRIC $\phi R_n = \left(13.4\,\frac{\mathrm{kN}}{\mathrm{mm}}\right)\left(d_{nom} - 73.0\,\mathrm{mm}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{102.4\,\frac{kips}{in.}\left(d_{nom} - 2.875\,in.\right)}{2.00}$ ENGLISH $\frac{R_n}{\Omega} = \left(51.2\,\frac{kip}{in.}\right)\left(d_{nom} - 2.875\,in.\right)$ METRIC $\frac{R_n}{\Omega} = \left(8.97\,\frac{\mathrm{kN}}{\mathrm{mm}}\right)\left(d_{nom} - 73.0\,\mathrm{mm}\right)$

Note: Based on the preceding calculations, shear rupture is the governing limit state of the two.

2.2.17 Friction at Collar Flange / Collar Corners and Column

The next step in the design process is to determine the slip resistance of the friction force between the CFA and CCA and the Column. In accordance with *Specification* Chapter J3.8, the design slip resistance, ϕR_n, and the allowable slip resistance, R_n/Ω, shall be determined for the limit state of slip as follows:

$$R_n = \left(\mu D_u h_f T_b n_s\right) n_b \qquad \text{(J3-4)}$$

For standard size and short-slotted holes perpendicular to the direction of the load,

$\phi = 1.00$ (LRFD) $\qquad$ $\Omega = 1.50$ (ASD)

For the limit state of slip resistance, the following definitions apply:

$\mu =$	mean slip coefficient for Class A or B surfaces. The CFA and CCA have Class A surfaces and therefore μ = 0.30.
$D_u =$	1.13, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension.
$T_b =$	minimum fastener tension given in *Specification* Table J3.1, *kips*, or Table J3.1M, kN
$h_f =$	factor for filler. There are no fillers present, thus, h_f =1.0
$n_s =$	number of slip planes. For this case, n_s = 1.0. Only one slip plane is used for each bolt because only the side pertaining to one beam is considered.
$n_b =$	number bolts. For this case, n_b = 8.

Using the information above, the slip resistance is determined as follows:

$$R_n = \left(\mu D_u h_f T_b n_s\right) n_b$$

$$R_n = 0.30(1.13)(1.0)(T_b)(1.0)(8)$$

$$R_{ncf} = 2.712T_b$$

The design slip resistance, ϕR_n, and the allowable slip resistance, R_n/Ω, is

LRFD	ASD
$\phi R_{ncf} = \phi(2.712T_b)$ $= 1.00(2.712T_b)$ $= 2.712T_b$	$\dfrac{R_{ncf}}{\Omega} = \dfrac{2.712T_b}{1.50}$ $= 1.808T_b$

The Demand Capacity Ratio for the shear strength of the friction force at the Collar Flange Collar Corner and column interface is shown below:

LRFD	ASD
$DCR = \dfrac{V_{ux}}{\phi R_{ncf}} = \dfrac{V_{ux}}{2.712T_b}$	$DCR = \dfrac{V_{ux}}{\left(R_{ncf}/\Omega\right)} = \dfrac{V_{ux}}{1.808T_b}$

2.2.18 Y-Axis Shear Transfer from Beam to Column

In the ConXL-300 moment collar connection, weak-axis shear is transferred to the Collar Flange Assembly via CJP welds between the beam flanges and the Collar Flanges (CFs). The Collar Flanges transfer the shear load via direct bearing against the Collar Corners (CCs); the weak-axis shear thus does not contribute to the interaction of forces resisted by the bolts. The Collar Corners transfer the shear load to the column via fillet welds between the Collar Corners and the column.

2.2.19 Bearing Strength at Collar Flange and Collar Corner

In accordance with *Specification* Section J7, the available bearing strength of surfaces in contact shall be determined for the limit state of bearing as follows:

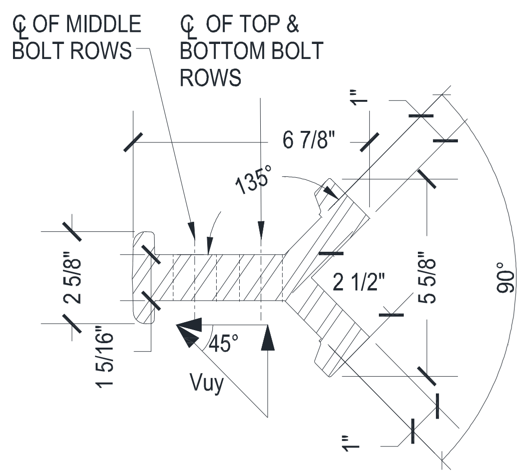
$$R_n = 1.8 F_y A_{pb} \qquad \text{(J7-1)}$$

$\phi = 0.75$ (LRFD) $\qquad \Omega = 2.00$ (ASD)

The Collar Corner Assemblies are made from ASTM A572 Grade 50 material. This material has the following specifications:

$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*

As shown in the figure on the right, the weak-axis shear is resisted by bearing of the Collar Flange against two surfaces of the Collar Corner. One component of the shear is resisted by the head of the Collar Corner and the other component is resisted by the neck of the Collar Corner.



Plan View of Collar Corner Section.

The dimensions of the tapered Collar Corner that result in the minimum bearing areas will be used,

where:

d_{CFb} = effective depth of Collar Flange for bearing
w_{cch} = width of Collar Corner head
w_{ccn} = width of Collar Corner neck
t_{ccn} = thickness of Collar Corner neck

For bearing at the head of the Collar Corner:

$$A_{pbh} = 2d_{CF}\left(\frac{w_{cch} - t_{ccn}}{2}\right) = 2\left(5.00\,in.\right)\left(\frac{2.625\,in. - 1.3125\,in.}{2}\right)$$
$$= 6.563\,in.^2$$

$$R_{nbh} = 1.8F_y A_{pbh}$$
$$= 1.8\left(50\,ksi\right)\left(6.563\,in.^2\right)$$
$$= 590.6\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{nbh} = 0.75\left(590.6\,kips\right)$ ENGLISH $\phi R_{nbh} = 442.9\,kips$ METRIC $\phi R_{nbh} = 1{,}970\,\text{kN}$	$\Omega = 2.00$ $\frac{R_{nbh}}{\Omega} = \frac{590.6\,kips}{2.00}$ ENGLISH $\frac{R_{nbh}}{\Omega} = 295.3\,kips$ METRIC $\frac{R_{nbh}}{\Omega} = 1{,}313\,\text{kN}$

For bearing at the neck of the Collar Corner:

$$A_{pbn} = 2d_{CF}w_{ccn} = 2(5.00\,in.)(3.75\,in.)$$
$$= 37.50\,in.^2$$

$$R_{nbn} = 1.8F_y A_{pbn}$$
$$= 1.8(50\,ksi)(37.50\,in.^2)$$
$$= 3,375\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{nbn} = 0.75(3375\,kips)$ ENGLISH $\phi R_{nbn} = 2,531\,kips$ METRIC $\phi R_{nbn} = 11,260\,\text{kN}$	$\Omega = 2.00$ $\frac{R_{nbn}}{\Omega} = \frac{3375\,kips}{2.00}$ ENGLISH $\frac{R_{nbn}}{\Omega} = 1,688\,kips$ METRIC $\frac{R_{nbh}}{\Omega} = 7,506\,\text{kN}$

The Demand Capacity Ratio for the bearing strength of the Collar Corner Assembly is shown below:

LRFD	ASD
$DCR = \frac{V_{uy}\cos(45°)}{\phi R_{nbh}} + \frac{V_{uy}\sin(45°)}{\phi R_{nbn}}$	$DCR = \frac{V_{uy}\cos(45°)}{R_{nbh}/\Omega} + \frac{V_{uy}\sin(45°)}{R_{nbn}/\Omega}$

where

V_{uy} = required weak-axis beam shear load from load combinations.

2.2.20 Tensile Yielding and Tensile Rupture in Collar Corner Assembly

In accordance with *Specification* Section J4.1, the available tensile strength of the affected and connecting elements shall be the lower value obtained according to the limit states of tensile yielding and tensile rupture:

c. For tensile yielding of the element:

$$R_n = F_y A_g \quad \text{(J4-1)}$$

$\phi = 0.90$ (LRFD) $\quad \Omega = 1.67$ (ASD)

d. For tensile rupture of the element:

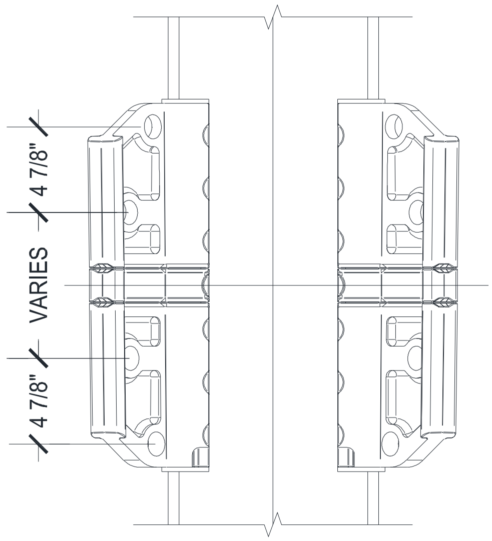
$$R_n = F_u A_e \quad \text{(J4-4)}$$

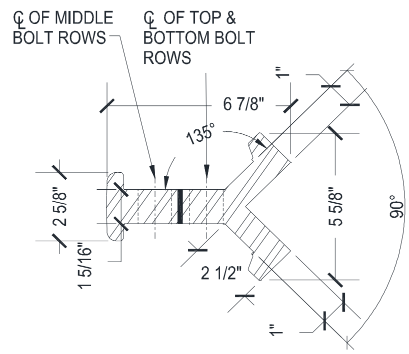
$\phi = 0.75$ (LRFD) $\quad \Omega = 2.00$ (ASD)

The Collar Corner Assemblies are made from ASTM A572 Grade 50 material. This material has the following specifications:

$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*

As can be seen in the figure directly on the right, the thickness of the CCA is 1-5/16 *in*. (33.3 mm).





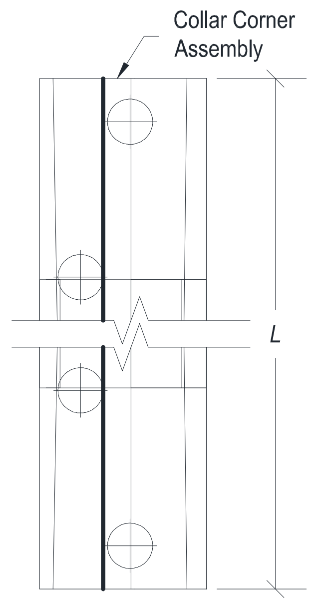
Plan View of Collar Corner Section; tension plane indicated by thickened vertical line.

The thickened vertical line in the figure on the right shows the tension plane of the gross section. The length of the tension plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism.

For tensile yielding in the gross-section:

$$A_g = \left(1\frac{5}{16}in.\right)L = (1.3125\,in.)\,d_{nom}$$

$$\begin{aligned} R_n &= F_y A_g \\ &= (50\,ksi)(1.3125\,in.)\,d_{nom} \\ &= \left(65.6\frac{kip}{in.}\right)d_{nom} \end{aligned}$$



Tensile Yielding in the CCA.

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90\left(65.6\frac{kip}{in.}\right)d_{nom}$ ENGLISH $\phi R_n = \left(59.0\frac{kip}{in.}\right)d_{nom}$ METRIC $\phi R_n = \left(10.3\frac{\text{kN}}{\text{mm}}\right)d_{nom}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{\left(65.6\frac{kip}{in.}\right)d_{nom}}{1.67}$ ENGLISH $\frac{R_n}{\Omega} = \left(39.3\frac{kip}{in.}\right)d_{nom}$ METRIC $\frac{R_n}{\Omega} = \left(6.88\frac{\text{kN}}{\text{mm}}\right)d_{nom}$

The thickened vertical lines in the figure on the right show the tension plane of the net section. In conformance with *Specification* Section B4.3b, in computing the net area for tension and shear, the width of a bolt hole shall be taken as 1/16 *in*. (2 mm) greater than the nominal dimension of the hole.

The length of the tension plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism. The thickness of the CCA was previously determined to be 1-5/16 *in*. (33.3 mm).

For tensile rupture on the effective net area:

$$A_e \approx A_{nt} = \left(L - 2\left(d_h + \frac{1}{16} in.\right)\right)\left(1\frac{5}{16} in.\right)$$

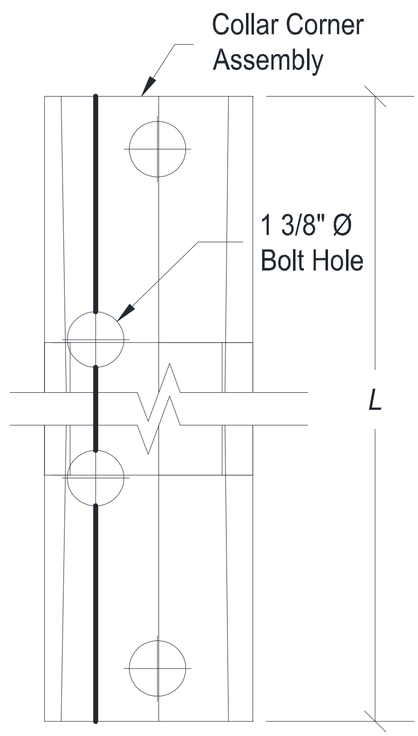
$$= \left(d_{nom} - 2\left(1\frac{3}{8} in. + \frac{1}{16} in.\right)\right)(1.3125 in.)$$

$$= (d_{nom} - 2.875 in.)(1.3125 in.)$$

$$R_n = F_u A_e$$

$$= (65 ksi)(d_{nom} - 2.875 in.)(1.3125 in.)$$

$$= 85.3 \frac{kip}{in.}(d_{nom} - 2.875 in.)$$



Tensile Rupture in the CCA.

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(85.3 \frac{kip}{in.}(d_{nom} - 2.875 in.)\right)$	$\frac{R_n}{\Omega} = \frac{85.3 \frac{kip}{in.}(d_{nom} - 2.875 in.)}{2.00}$
ENGLISH	ENGLISH
$\phi R_n = \left(64.0 \frac{kip}{in.}\right)(d_{nom} - 2.875 in.)$	$\frac{R_n}{\Omega} = \left(42.7 \frac{kip}{in.}\right)(d_{nom} - 2.875 in.)$
METRIC	METRIC
$\phi R_n = \left(11.2 \frac{\text{kN}}{\text{mm}}\right)(d_{nom} - 73.0 \text{ mm})$	$\frac{R_n}{\Omega} = \left(7.47 \frac{\text{kN}}{\text{mm}}\right)(d_{nom} - 73.0 \text{ mm})$

Note: based on the preceding calculations, tensile rupture is the governing limit state of the two.

The Demand Capacity Ratio for the tensile strength of the Collar Corner Assembly is shown below:

LRFD	ASD
$DCR = \dfrac{V_{uy}\cos(45°)}{\phi R_n}$	$DCR = \dfrac{V_{uy}\cos(45°)}{R_n/\Omega}$

2.2.21 Shear Yielding and Shear Rupture in Collar Corner Assembly (at Neck, Y-Axis Shear)

The shear planes for shear yielding and shear rupture in the Collar Corner are identical to those described in Section 2.2.16. Since the Collar Flanges bear against one Collar Corner Assembly, only one Collar Corner Assembly resists the weak-axis shear (instead of two as is the case for strong-axis shear). Therefore, the available design strengths and allowable strengths will be one-half of those computed in Section 2.2.16.

For shear yielding in the gross-section, the available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00\left(78.8\,\dfrac{kip}{in.}\right)d_{nom} \times \frac{1}{2}$ ENGLISH $\phi R_n = \left(39.4\,\dfrac{kip}{in.}\right)d_{nom}$ METRIC $\phi R_n = \left(6.90\,\dfrac{\text{kN}}{\text{mm}}\right)d_{nom}$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{\left(78.8\,\dfrac{kip}{in.}\right)d_{nom}}{1.50} \times \frac{1}{2}$ ENGLISH $\dfrac{R_n}{\Omega} = \left(26.3\,\dfrac{kip}{in.}\right)d_{nom}$ METRIC $\dfrac{R_n}{\Omega} = \left(4.60\,\dfrac{\text{kN}}{\text{mm}}\right)d_{nom}$

For shear rupture in the net-section, the available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(102.4\,\dfrac{kips}{in.}\left(d_{nom} - 2.875\,in.\right)\right) \times \frac{1}{2}$	$\dfrac{R_n}{\Omega} = \dfrac{102.4\,\dfrac{kips}{in.}\left(d_{nom} - 2.875\,in.\right)}{2.00} \times \frac{1}{2}$
ENGLISH	ENGLISH
$\phi R_n = \left(38.4\,\dfrac{kip}{in.}\right)\left(d_{nom} - 2.875\,in.\right)$	$\dfrac{R_n}{\Omega} = \left(25.6\,\dfrac{kip}{in.}\right)\left(d_{nom} - 2.875\,in.\right)$
METRIC	METRIC
$\phi R_n = \left(6.72\,\dfrac{\text{kN}}{\text{mm}}\right)\left(d_{nom} - 73.0\text{ mm}\right)$	$\dfrac{R_n}{\Omega} = \left(4.48\,\dfrac{\text{kN}}{\text{mm}}\right)\left(d_{nom} - 73.0\text{ mm}\right)$

Note: Based on the preceding calculations, shear rupture is the governing limit state of the two.

The Demand Capacity Ratio for the shear strength of the Collar Corner Assembly at the neck is shown below:

LRFD	ASD
$DCR = \dfrac{V_{uy}\sin(45°)}{\phi R_n}$	$DCR = \dfrac{V_{uy}\sin(45°)}{R_n/\Omega}$

2.2.22 Shear Yielding in Collar Corner Assembly (at Head, Y-Axis Shear)

In accordance with *Specification* Section J4.2, the available shear strength of the affected and connecting elements shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

c. For shear yielding of the element:

$$R_n = 0.60 F_y A_{gv} \quad \text{(J4-3)}$$

$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$

d. For shear rupture of the element:

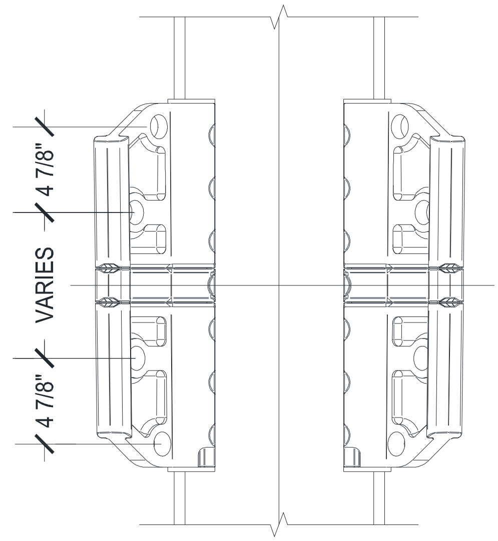
$$R_n = 0.60 F_u A_{nv} \quad \text{(J4-4)}$$

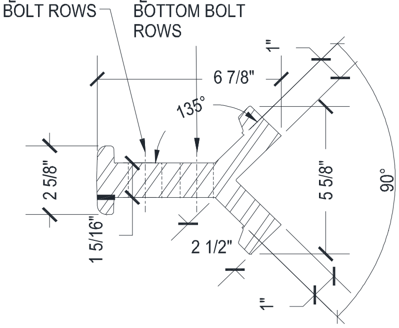
$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$

The Collar Corner Assemblies are made from ASTM A572 Grade 50 material. This material has the following specifications:

$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*

The limit state of shear rupture on the net-section will not be evaluated as there are no bolt holes interrupting the shear plane.





Plan View of Collar Corner Section; shear plane indicated by thickened horizontal line.

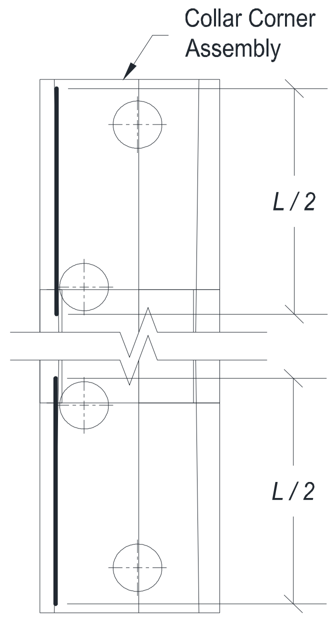
The thickened vertical lines in the figure on the right show the shear plane of the gross section. The length of the shear plane, L, is taken as twice the depth of the Collar Flange, d_{CF}, for conservatism.

The thickness of the Collar Corner head, t_{cch}, will be taken as the minimum thickness at the tip for conservatism

For shear yielding in the gross-section:

$$A_{gv} = 2d_{CF}t_{cch} = 2(5.00\,in.)(0.625\,in.)$$
$$= 6.250\,in.^2$$

$$R_n = 0.60F_yA_{gv}$$
$$= 0.60(50\,ksi)(6.250\,in.^2)$$
$$= 187.5\,kips$$



Shear Yielding in the CCA.

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(187.5\,kips)$ ENGLISH $\phi R_n = 187.5\,kips$ METRIC $\phi R_n = 834.0\,\text{kN}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{(187.5\,kips)}{1.50}$ ENGLISH $\frac{R_n}{\Omega} = 125.0\,kips$ METRIC $\frac{R_n}{\Omega} = 556.0\,\text{kN}$

The Demand Capacity Ratio for the shear strength of the Collar Corner Assembly at the head is shown below:

LRFD	ASD
$$DCR = \frac{V_{uy}\cos(45°)}{\phi R_n}$$	$$DCR = \frac{V_{uy}\cos(45°)}{R_n/\Omega}$$

2.2.23 Combined Shear and Tension in Collar Corner Assembly (Y-Axis Shear)

For the interaction of shear and tension in the Collar Corner Assembly, the Demand Capacity Ratio is determined as shown below:

LRFD	ASD
$$DCR = \sqrt{\left(\frac{V_{uy}\cos(45°)}{\phi R_{ntr}}\right)^2 + \left(\frac{V_{uy}\sin(45°)}{\phi R_{nsr}}\right)^2}$$	$$DCR = \sqrt{\left(\frac{V_{uy}\cos(45°)}{R_{ntr}/\Omega}\right)^2 + \left(\frac{V_{uy}\sin(45°)}{R_{nsr}/\Omega}\right)^2}$$

where:

R_{ntr} = tensile rupture strength at the neck from Section 2.2.20

R_{nsr} = shear rupture strength at the neck from Section 2.2.21

2.2.24 Weld at Collar Corner Assembly and Column (Y-Axis Shear)

The ConXL-300 Collar Corner Assembly to column connection is a 3/8 *in*. (9.53 mm) fillet weld. The weld is perpendicular to the direction of the weak-axis beam shear load. The load is oriented 90° to the longitudinal axis of the weld.

Using d_{nom} to denote the nominal depth of the connected beam, the total length of the fillet weld is:

$$l_{wc} = 2\left(d_{nom} + 4.5\, in.\right)$$

The size of the fillet weld is:

$$t_{wc} = \frac{\sqrt{2}}{2} \cdot \frac{3}{8}\, in. = 0.265\, in.$$

The area of the fillet weld is:

$$A_{wc} = t_{wc} l_{wc} = \left(0.265\, in.\right) \times 2\left(d_{nom} + 4.5\, in.\right)$$

The strength of the fillet weld is:

$$\begin{aligned} \theta &= 90^{\circ} \\ R_n &= 0.60 F_{EXX}\left(1.0 + 0.50 \sin^{1.5}(\theta)\right) A_{we} \\ &= 0.60\left(70\, ksi\right)\left(1.0 + 0.50 \sin^{1.5}\left(90^{\circ}\right)\right)\left(0.53\, in.\right)\left(d_{nom} + 4.5\, in.\right) \\ &= 0.60\left(70\, ksi\right)\left(1.0 + 0.50\right)\left(0.53\, in.\right)\left(d_{nom} + 4.5\, in.\right) \end{aligned}$$

$$R_n = 33.4 \frac{kips}{in.}\left(d_{nom} + 4.5\, in.\right)$$

The design and allowable shear strength of the Collar Corner to column fillet welds is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(33.4\,\dfrac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)\right)$ ENGLISH $\phi R_n = 25.0\,\dfrac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)$ METRIC $\phi R_n = 4.38\,\dfrac{\text{kN}}{\text{mm}}\left(d_{nom} + 114.3\text{ mm}\right)$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{33.4\,\dfrac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)}{2.00}$ ENGLISH $\dfrac{R_n}{\Omega} = 16.7\,\dfrac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)$ METRIC $\phi R_n = 2.92\,\dfrac{\text{kN}}{\text{mm}}\left(d_{nom} + 114.3\text{ mm}\right)$

The Demand Capacity Ratio for design and allowable shear strength of the Collar Corner to column fillet welds is:

LRFD	ASD
ENGLISH $DCR = \dfrac{V_{uy}}{\phi R_n} = \dfrac{V_{uy}}{25.0\,\dfrac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)}$ METRIC $DCR = \dfrac{V_{uy}}{\phi R_n} = \dfrac{V_{uy}}{4.38\,\dfrac{\text{kN}}{\text{mm}}\left(d_{nom} + 114.3\text{ mm}\right)}$	ENGLISH $DCR = \dfrac{V_{uy}}{\left(R_n/\Omega\right)} = \dfrac{V_{uy}}{16.7\,\dfrac{kips}{in.}\left(d_{nom} + 4.5\,in.\right)}$ METRIC $DCR = \dfrac{V_{uy}}{\phi R_n} = \dfrac{V_{uy}}{2.92\,\dfrac{\text{kN}}{\text{mm}}\left(d_{nom} + 114.3\text{ mm}\right)}$

2.2.25 Collar Corner Assembly: Combined Shear Loading

Where there is a combined shear load in both X and Y axes, the Demand Capacity Ratio is determined as shown below:

LRFD
$DCR = \sqrt{\left(\frac{V_{ux}}{\phi R_{nsr}}\right)^2 + \left(\frac{V_{uy}\cos(45°)}{\phi R_{ntr}}\right)^2 + \left(\frac{V_{uy}\sin(45°)}{\phi R_{nsr}}\right)^2}$
ASD
$DCR = \sqrt{\left(\frac{V_{ux}}{R_{nsr}/\Omega}\right)^2 + \left(\frac{V_{uy}\cos(45°)}{R_{ntr}/\Omega}\right)^2 + \left(\frac{V_{uy}\sin(45°)}{R_{nsr}/\Omega}\right)^2}$

2.2.26 Weld at Collar Corner Assembly: Combined Shear Loading

Where there is a combined shear load in both X and Y axes, the Demand Capacity Ratio is determined as shown below:

LRFD	ASD
$DCR = \sqrt{\left(\frac{V_{ux}}{\phi R_{nwx}}\right)^2 + \left(\frac{V_{uy}}{\phi R_{nwy}}\right)^2}$	$DCR = \sqrt{\left(\frac{V_{ux}}{R_{nwx}/\Omega}\right)^2 + \left(\frac{V_{uy}}{R_{nwy}/\Omega}\right)^2}$

where

R_{nwx} = available strong-axis shear strength from Section 2.2.15.

R_{nwy} = available weak-axis shear strength from Section 2.2.24.

2.2.27 Panel Zone Shear Strength

The final step of the ConXL-300 moment connection design process is determining the Shear Strength of the Panel Zone.

Panel zone shear is resisted by the column walls and the Collar Corner Assemblies. The Collar Corner Assemblies act as reinforcing plates for the column web.

The Collar Corner Assemblies are welded along their entire depth to the column, with their depth greater than the depth of the moment beam.

In accordance with *Specification* Section J10.6, the available strength of the web panel zone for the limit state of shear yielding shall be determined as follows:

$$\phi = 0.90 \ \ (\text{LRFD}) \qquad \Omega = 1.67 \ \ (\text{ASD})$$

In the figure on the right, the shaded regions represent the area effective in resisting panel zone shear.

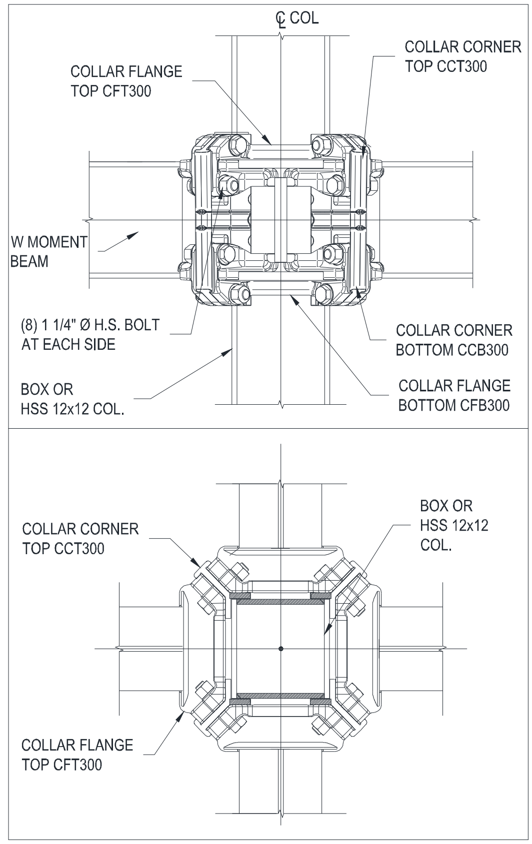


Figure 59: ConXL-300 Elevation and Plan View at Panel Zone.

The nominal strength, R_n, shall be determined as follows: when the effect of panel-zone deformation on frame stability is not considered in the analysis:

$$\text{For } P_r \le 0.4P_c \qquad R_n = 0.60F_y d_c t_w \qquad \text{(J10-9)}$$

$$\text{For } P_r > 0.4P_c \qquad R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \qquad \text{(J10-10)}$$

where

F_y = specified minimum yield stress of the column web, *ksi* (MPa)

d_c = depth of column, *in*. (mm)

t_w = thickness of column web, *in*. (mm)

P_r = required axial strength using LRFD or ASD load combinations, *kips* (*kN*)

P_c = P_y (LRFD) or 0.60P_y (ASD), *kips* (kN)

$P_y =$ F_yA_g, axial yield strength of the column, *kips* (kN)
$A_g =$ gross cross-sectional area of column, $in.^2$ (mm^2)

The Collar Corner Assemblies contribute to the steel area resisting the panel zone shear. The properties of the column and CCAs pertaining to panel zone shear are shown below:

$A_{pz} =$ $2d_ct_c + 4d_{cc}t_{cc}$
$t_c =$ thickness of column wall, *in*. (mm)
$d_c =$ depth of column, 12 *in*. (304.8 mm)
$d_{cc} =$ depth of Collar Corner = 2.625 *in.* (66.7 mm)
$t_{cc} =$ effective thickness of Outer Collar = 0.375 *in.* (9.525 mm)

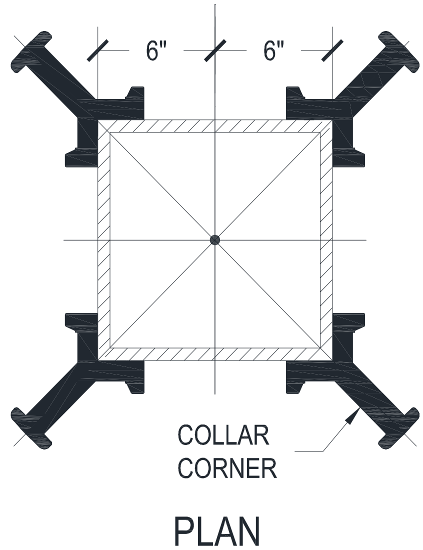


Figure 60: Effective Shear Areas of Column and CCA.

The shear strength of the panel zone is as follows:

For $P_r \le 0.4P_c$ $$R_{nPZ} = 0.60F_y\left(2d_ct_c + 4d_{cc}t_{cc}\right)$$

For $P_r > 0.4P_c$ $$R_{nPZ} = 0.60F_y\left(2d_ct_c + 4d_{cc}t_{cc}\right)\left(1.4 - \frac{P_r}{P_c}\right)$$

The design panel zone shear strength, ϕR_n, and the allowable panel zone shear strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi R_{nPZ} = 0.90R_{nPZ}$	$\frac{R_{nPZ}}{\Omega} = \frac{R_{nPZ}}{1.67}$

The Demand Capacity Ratio for the panel zone shear strength is shown below:

LRFD	ASD
$\mathrm{DCR} = \frac{V_{uPZ}}{\phi R_{nPZ}}$	$\mathrm{DCR} = \frac{V_{uPZ}}{\left(R_{nPZ}/\Omega\right)}$

2.2.28 Local Yielding of HSS Sidewalls

In accordance with Section J10.2 of the *Specification*, the limit state of Web Local Yielding (i.e., Local Yielding of HSS Sidewalls) is determined as follows:

$$R_n = F_{yw} t_w \left(5k + l_b\right) \qquad \text{(J10-2)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

F_{yw} = specified minimum yield stress of HSS material, *ksi* (MPa)

l_b = bearing length, which equals the depth of the Collar Flange, d_{CF}.

k= outside HSS corner radius (0 for Box)

$$F_y = 50\, ksi \quad \left(345\, MPa\right)$$

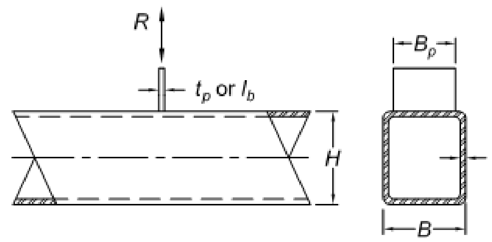
$$t_w = t_{des}$$

$$l_b = d_{CF} = 5.0\, in.$$

$$k = 1.5 t_{des}$$

Connection Type

Transverse Plate T- and Cross-Connections, Under Plate Axial Load



where $\beta = \dfrac{B_p}{B}$

Note: since HSS members have two webs, the available strength resulting from Eq. J10-2 will be multiplied by 2.

Note: Eq. J10-2 only applies when the concentrated load is applied at a distance from the member end that is greater than the nominal depth of the HSS (i.e., 12 *in.*). If the concentrated load is applied at a distance from the member end that is less than or equal to the nominal depth of the HSS (e.g., if the collar is near the top of the column), then Eq. J10-3 is used instead as demonstrated on the following pages.

The available strength is therefore calculated as follows:

$$\begin{aligned} R_n &= F_{yw} t_w \left(5k + l_b\right) \times 2\, walls \\ &= \left(50\, ksi\right)\left(t_{des}\right)\left(5\left(1.5 t_{des}\right) + 5.0\, in.\right) \times 2\, walls \\ &= \left(100\, ksi\right) t_{des} \left(7.5 t_{des} + 5.0\, in.\right) \end{aligned}$$

The design wall local yielding strength, ϕR_n, and the allowable wall local yielding strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(100\ ksi)t_{des}(7.5t_{des} + 5.0\ in.)$ ENGLISH $\phi R_n = (100\ ksi)t_{des}(7.5t_{des} + 5.0\ in.)$ METRIC $\phi R_n = (689\ \text{MPa})t_{des}(7.5t_{des} + 127\ \text{mm})$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{(100\ ksi)t_{des}(7.5t_{des} + 5.0\ in.)}{1.50}$ ENGLISH $\frac{R_n}{\Omega} = (66.7\ ksi)t_{des}(7.5t_{des} + 5.0\ in.)$ METRIC $\frac{R_n}{\Omega} = (460\ \text{MPa})t_{des}(7.5t_{des} + 127\ \text{mm})$

When the concentrated force is applied at a distance from the member end that is less than or equal to the nominal depth of the HSS (i.e., 12 *in.*), the coefficient on *k* decreases to 2.5 (Eq. J10-3); for example, this condition should be checked when the collar is located near the top of the column. The design wall local yielding strength, ϕR_n, and the allowable wall local yielding strength, R_n/Ω, are then calculated as follows:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(100\ ksi)t_{des}(2.5(1.5t_{des}) + 5.0\ in.)$ ENGLISH $\phi R_n = (100\ ksi)t_{des}(3.75t_{des} + 5.0\ in.)$ METRIC $\phi R_n = (689\ \text{MPa})t_{des}(3.75t_{des} + 127\ \text{mm})$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{(100\ ksi)t_{des}(2.5(1.5t_{des}) + 5.0\ in.)}{1.50}$ ENGLISH $\frac{R_n}{\Omega} = (66.7\ ksi)t_{des}(3.75t_{des} + 5.0\ in.)$ METRIC $\frac{R_n}{\Omega} = (460\ \text{MPa})t_{des}(3.75t_{des} + 127\ \text{mm})$

The Demand Capacity Ratio for wall local yielding is shown below:

LRFD	ASD
$DCR = \dfrac{P_{rf}}{\phi R_n}$	$DCR = \dfrac{P_{rf}}{\left(R_n/\Omega \right)}$

where

$$P_{rf} = \left(\frac{M_{ux}}{d - t_f} \right) + \frac{P_{ub}}{2}$$

with M_{ux} and the P_{ub} are the required beam moment (X-axis) and the required beam axial demand, respectively. The term d denotes the depth of the beam and the term t_f is the thickness of the beam flange.

2.2.29 Local Crippling of HSS Sidewalls

In accordance with Section J10.3 of the *Specification*, the limit state of Web Local Yielding (i.e., Local Crippling of HSS Sidewalls) is determined as follows:

$$R_n = 0.80 t_w^{\,2}\left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{E F_{yw} t_f}{t_w}}\,Q_f$$

(J10-4)

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$

where

E = modulus of elasticity of steel = 29,000 *ksi* (200,000 MPa)

F_{yw} = specified minimum yield stress of HSS main member material, *ksi* (MPa)

l_b = thickness of transverse plate, which equals the depth of the Collar Flange, d_{CF}.

t_w = design web thickness of HSS or Box column
= t_{des}

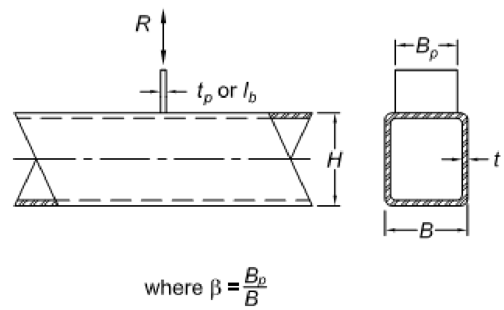
t_f = design flange thickness of HSS or Box column
= t_{des}

d = depth of web, *in.* (mm)
= $H - 3t_{des}$

Q_f = chord-stress interaction parameter

Connection Type

Transverse Plate T- and Cross-Connections, Under Plate Axial Load



Note: since HSS members have two webs, the available strength resulting from Eq. J10-4 will be multiplied by 2.

Note: Eq. J10-4 only applies when the concentrated compressive force is applied at a distance from the member end that is greater than or equal to $d/2$. If the concentrated compressive force is applied at a distance from the member end that is less than $d/2$ (e.g., if the collar is near the top of the column), then Eq. J10-5a or J10-5b should be used instead as demonstrated on the following pages.

From *Specification* Table K3.2, Q_f is determined as follows:

$Q_f = 1$ for connecting surface in tension,

$Q_f = 1.3 - 0.4\dfrac{U}{\beta}$ for connecting surface in compression.

Per *Specification* Section K3.1, the width ratio, β, is defined as the ratio of overall branch width (i.e., Collar Flange length) to chord width (i.e., column width) $= B_b / B$ for rectangular HSS. Since the Collar Flange length is slightly larger than the column width, β = 1.0.

The utilization ratio, U, is determined as follows using Eq. K2-4:

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right|$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS.

$P_{ro} = P_u$ for LRFD; P_a for ASD.
$M_{ro} = M_u$ for LRFD; M_a for ASD.

The available strength based on the limit state of local crippling is calculated as follows:

$$F_y = 50\,ksi \quad (345\text{ MPa})$$
$$H = 12\,in. \quad (304.8\text{ mm})$$
$$l_b = d_{CF} = 5.0\,in.$$

$$Q_f = 1.3 - 0.4\left(\frac{U}{1.0}\right) \le 1.0$$

$$R_n = 0.80 t_w^{\,2}\left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f \times 2\,walls$$
$$= 1.6 t_{des}^{\,2}\left(1 + 3\left(\frac{d_{CF}}{H - 3t_{des}}\right)\left(\frac{t_{des}}{t_{des}}\right)^{1.5}\right)\sqrt{\frac{EF_y t_{des}}{t_{des}}}Q_f$$
$$= 1.6 t_{des}^{\,2}\left(1 + \frac{3d_{CF}}{H - 3t_{des}}\right)\sqrt{EF_y}Q_f$$
$$= 1.6 t_{des}^{\,2}\left(\frac{H - 3t_{des} + 3d_{CF}}{H - 3t_{des}}\right)\sqrt{EF_y}Q_f$$
$$= 1.6 t_{des}^{\,2}\left(\frac{12\,in. - 3t_{des} + 3(5.0\,in.)}{12\,in. - 3t_{des}}\right)\sqrt{(29000\,ksi)(50\,ksi)}Q_f$$

$$R_n = (1{,}926\,ksi)\,t_{des}^{\,2}\left(\frac{27\,in. - 3t_{des}}{12\,in. - 3t_{des}}\right)Q_f$$

The design local wall crippling strength, ϕR_n, and the allowable local wall crippling strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(1{,}926\ ksi)t_{des}{}^2\left(\frac{27\ in. - 3t_{des}}{12\ in. - 3t_{des}}\right)Q_f$ ENGLISH $\phi R_n = (1{,}445\ ksi)t_{des}{}^2\left(\frac{27\ in. - 3t_{des}}{12\ in. - 3t_{des}}\right)Q_f$ METRIC $\phi R_n = (9{,}962\ \text{MPa})t_{des}{}^2\left(\frac{685.8\ \text{mm} - 3t_{des}}{304.8\ \text{mm} - 3t_{des}}\right)Q_f$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{(1{,}926\ ksi)t_{des}{}^2\left(\frac{27\ in. - 3t_{des}}{12\ in. - 3t_{des}}\right)Q_f}{2.00}$ ENGLISH $\frac{R_n}{\Omega} = (963\ ksi)t_{des}{}^2\left(\frac{27\ in. - 3t_{des}}{12\ in. - 3t_{des}}\right)Q_f$ METRIC $\frac{R_n}{\Omega} = (6{,}639\ \text{MPa})t_{des}{}^2\left(\frac{685.8\ \text{mm} - 3t_{des}}{304.8\ \text{mm} - 3t_{des}}\right)Q_f$

If the concentrated compressive force is applied at a distance from the member end that is less than $d/2$, then either Eq. J10-5a or J10-5b shall be used instead depending on the quantity l_b/d.

$$\frac{l_b}{d} = \frac{d_{CF}}{H - 3t_{des}} = \frac{5.0\,in.}{12.0\,in. - 3t_{des}}$$

The quantity l_b/d will always be greater than or equal to 0.2 regardless of t_{des}; therefore, Eq. J10-5b applies and the local wall crippling strength shall be determined as shown on the following page.

$$R_n = 0.40 t_w{}^2 \left(1 + \left(\frac{4l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f \times 2\, walls$$

$$= 0.80 t_{des}{}^2 \left(1 + \left(\frac{4d_{CF}}{H - 3t_{des}} - 0.2\right)\left(\frac{t_{des}}{t_{des}}\right)^{1.5}\right)\sqrt{\frac{EF_y t_{des}}{t_{des}}} Q_f$$

$$= 0.80 t_{des}{}^2 \left(1 + \frac{4d_{CF}}{H - 3t_{des}} - 0.2\right)\sqrt{EF_y} Q_f$$

$$= 0.80 t_{des}{}^2 \left(\frac{4d_{CF} + 0.8(H - 3t_{des})}{H - 3t_{des}}\right)\sqrt{EF_y} Q_f$$

$$= 0.80 t_{des}{}^2 \left(\frac{4(5.0\,in.) + 0.8(12\,in.) - 2.4t_{des}}{12\,in. - 3t_{des}}\right)\sqrt{(29{,}000\,ksi)(50\,ksi)} Q_f$$

$$R_n = (963\,ksi)\, t_{des}{}^2 \left(\frac{29.6\,in. - 2.4t_{des}}{12\,in. - 3t_{des}}\right) Q_f$$

The design local wall crippling strength, ϕR_n, and the allowable local wall crippling strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(963\ ksi)\, t_{des}{}^2 \left(\frac{29.6\ in. - 2.4t_{des}}{12\ in. - 3t_{des}}\right) Q_f$	$\frac{R_n}{\Omega} = \frac{(963\ ksi)\, t_{des}{}^2 \left(\frac{29.6\ in. - 2.4t_{des}}{12\ in. - 3t_{des}}\right) Q_f}{2.00}$
ENGLISH	ENGLISH
$\phi R_n = (722\ ksi)\, t_{des}{}^2 \left(\frac{29.6\ in. - 2.4t_{des}}{12\ in. - 3t_{des}}\right) Q_f$	$\frac{R_n}{\Omega} = (481\ ksi)\, t_{des}{}^2 \left(\frac{29.6\ in. - 2.4t_{des}}{12\ in. - 3t_{des}}\right) Q_f$
METRIC	METRIC
$\phi R_n = (4{,}980\ \mathrm{MPa})\, t_{des}{}^2 \left(\frac{751.8\ \mathrm{mm} - 2.4t_{des}}{304.8\ \mathrm{mm} - 3t_{des}}\right) Q_f$	$\frac{R_n}{\Omega} = (3{,}320\ \mathrm{MPa})\, t_{des}{}^2 \left(\frac{751.8\ \mathrm{mm} - 2.4t_{des}}{304.8\ \mathrm{mm} - 3t_{des}}\right) Q_f$

The Demand Capacity Ratio for wall local crippling is shown below:

LRFD	ASD
$DCR = \frac{P_{rf}}{\phi R_n}$	$DCR = \frac{P_{rf}}{\left(R_n / \Omega\right)}$

2.3 ConXR200

2.3.1 Moment Connection Description

The ConXtech ConXR-200 moment connection is a steel moment-resisting beam to column connection for resisting gravity, wind and seismic forces imposed on a building/non-building structure. As can be seen in Figure 61, the collar connection is designed to join hollow structural section columns (or built-up box columns) and wide flange steel beams at all four sides of the column, although the connection can be used for as few as one beam.

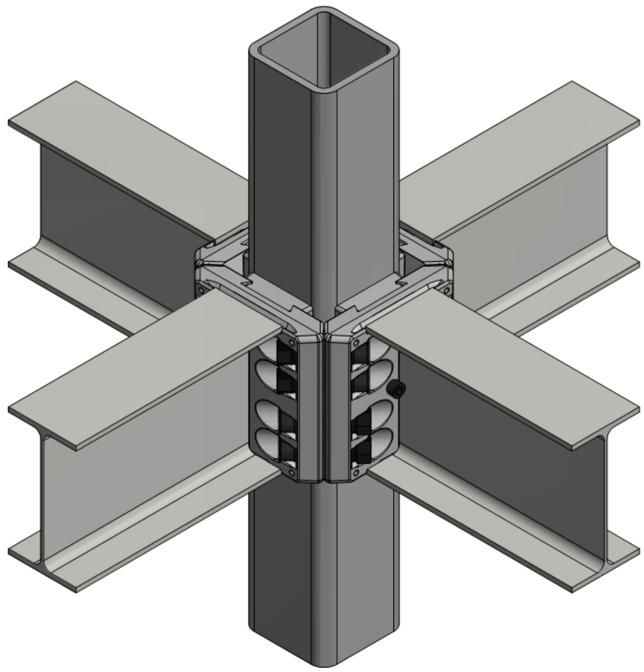


Figure 61: ConXR-200 Moment Connection Assembly.

Figure 62 shows an isolated view of a fully assembled moment collar. The moment collar is comprised of three main components. These components are the Inner Collar, the Outer Collar, and High Strength bolts. Four Outer Collars are joined to four Inner Collars, and the entire collar assembly is held together by sixteen pretensioned high strength bolts.

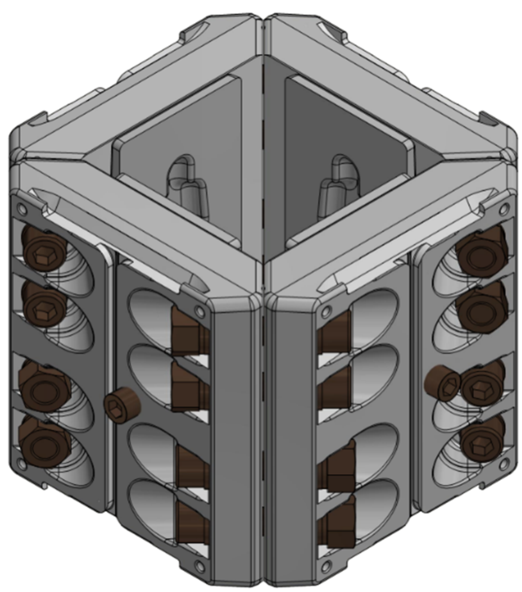


Figure 62: 3D-Perspective View of a ConXR-200 Moment Collar Assembly.

The ConXR-200 connection is only used on columns that are 8 *in*. (203.2 mm) square combined with wide flange beams of the sizes shown in Table 3. Note that the connection can only be used with moment beams from the same nominal depth family.

Table 3: List of Compatible Beam Sizes for the ConXR-200 System.

W12
W12x19
W12x22
W12x26
W12x30
W12x35
W12x40
W12x45
W12x50

The collar connection assembly comprises of four Inner Collar plates (IC) and four Outer Collar plates (OC). Each of the ICs is attached to the column with vertical fillet welds and is machined to include a large tapered boss. Each of the OCs is machined to include a tapered slot contoured to the same shape as the boss on the IC. The OC is also drilled along the vertical exterior edges to receive Collar Bolts.

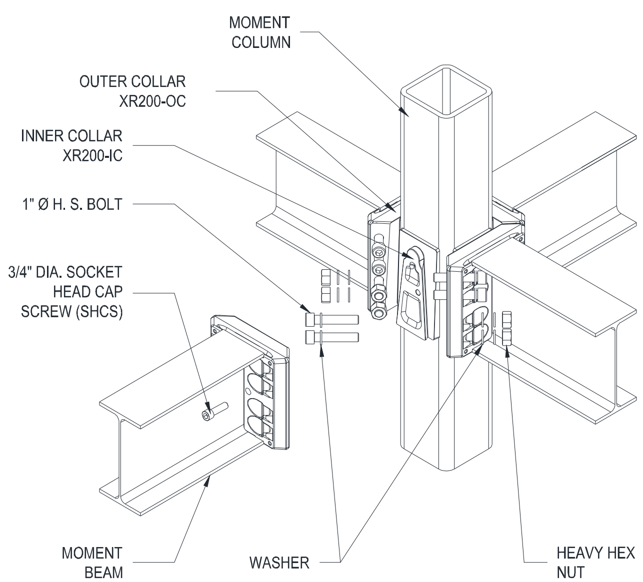


Figure 63: ConXR-200 Moment Connection Assembly.

Each OC is shop welded to a wide flange beam. Assembly in the field is accomplished by slipping one beam-OC assembly over the boss on the IC assembly on each face of the column.

A single horizontal bolt is also inserted on one side of the beam web, through a pre-drilled hole in the OC into a tapped hole in the IC.

Once beam-OC (or just OCs) assemblies have been located on each of the four faces of the column, high strength Collar Bolts are inserted into the holes in the adjoining OCs and pretensioned.

The pretensioning in the bolts clamps the OCs to the ICs/column, forming a rigid collar assembly that surrounds the column.

The proceeding four pages illustrate the ConXR-200 connection in more detail. Standard details of the Inner Collar, Outer Collar, and perspective images of beams fitted with Outer Collars are shown.

2.3.2 Inner Collar Assembly

Figure 64 below shows the Inner Collar assembly. The Inner Collar assembly comprises of four Inner Collar plates (IC). As can be seen in the Elevation and 3D-Perspective Views below, each IC is machined to include a large tapered shear lug and is attached to the column with vertical fillet welds. The ICs also consist of a bolt that is inserted through a pre-drilled hole in the Outer Collar into a tapped hole in the IC. This bolt provides a positive connection for uplift shear forces in the beam.

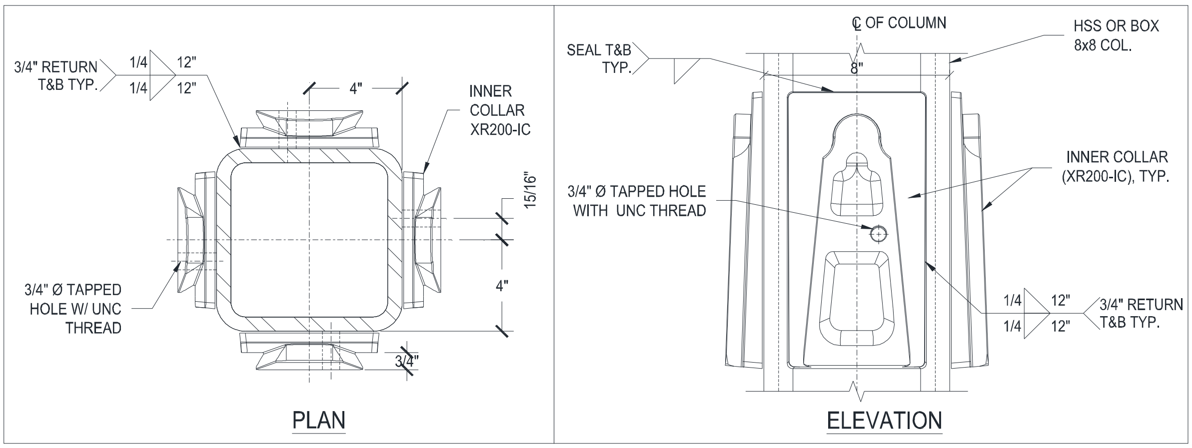


Figure 64: Plan and Elevation Views of the ConXR-200 Inner Collar.

Figure 65 on the right shows a 3D-Perspective View of the ConXR-200 Inner Collar assembly. Four Inner Collars are attached to the moment column via fillet welds (welds not shown for clarity).

Each Inner Collar contains a machined shear lug that is tapered: The tapered shear lugs allow the Outer Collars to be lowered into the Inner Collars, forming Inner and Outer Collar Assemblies.

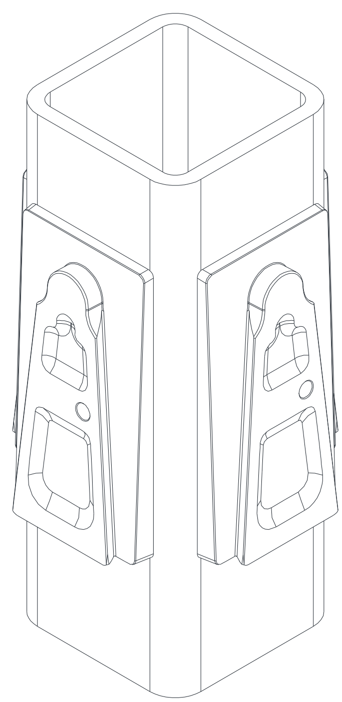


Figure 65: 3D-Perspective View of ConXR-200 Inner Collars welded onto a Column.

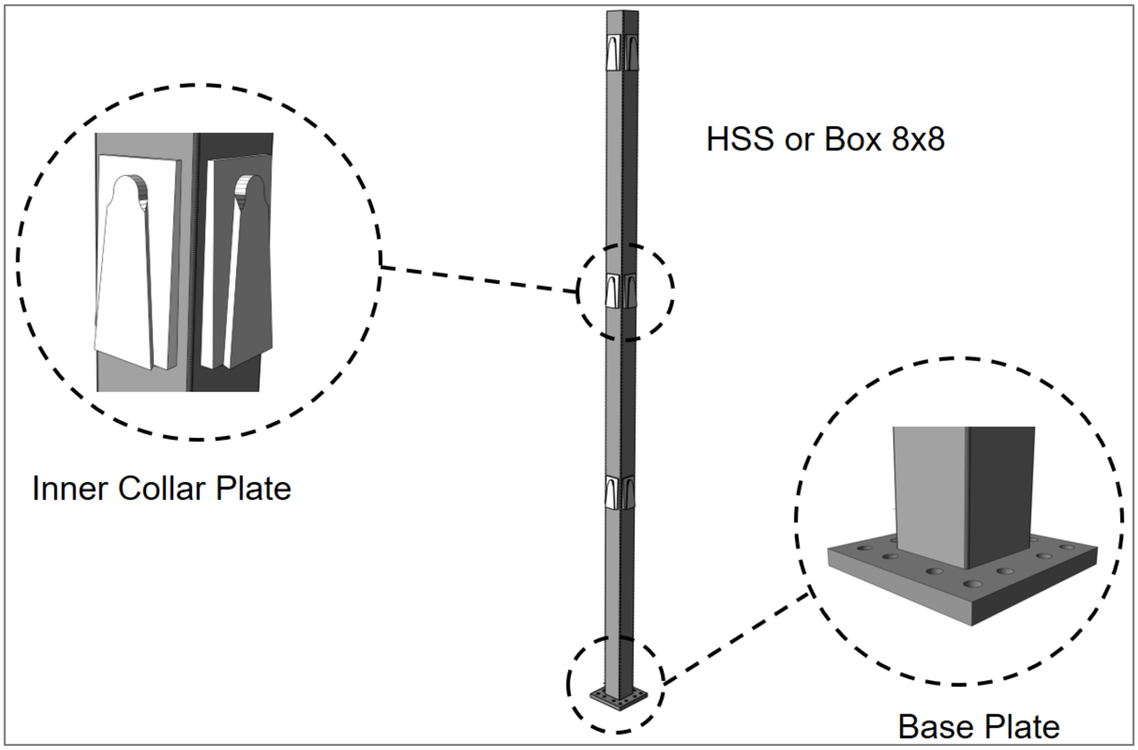


Figure 66: ConXR-200 Moment Column Assembly.

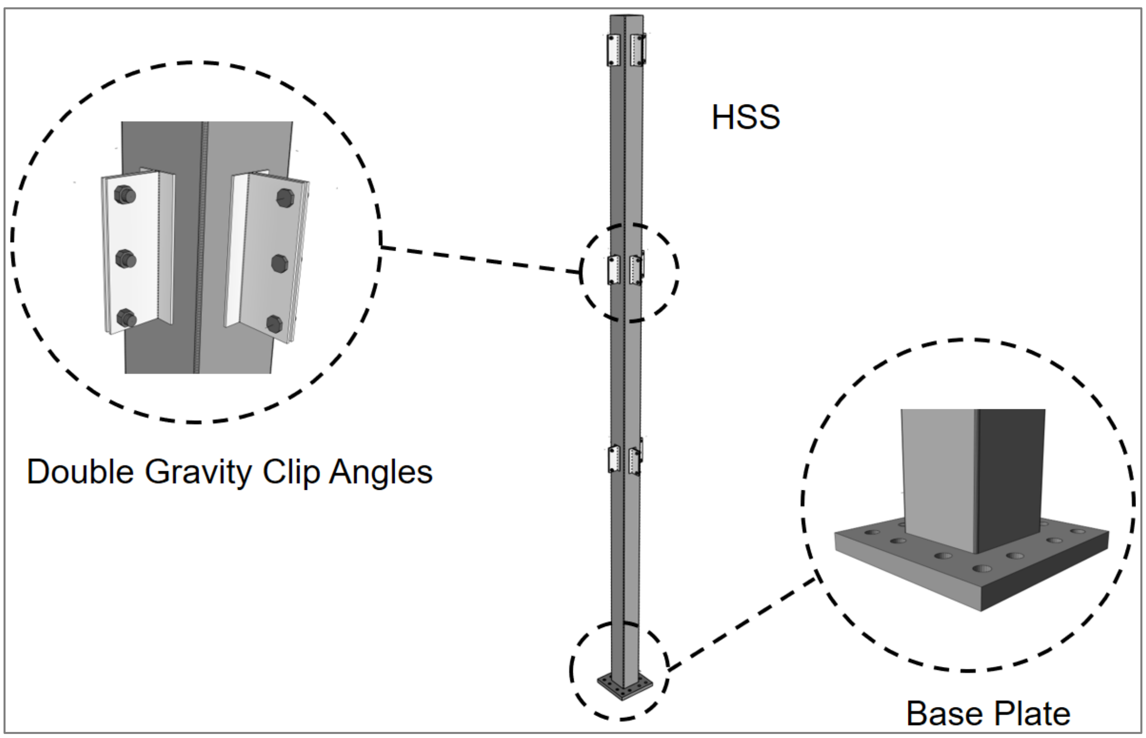


Figure 67: ConXR-200 Gravity Column Assembly.

2.3.3 Outer Collar Assembly

OUTER COLLAR PLAN (TOP)

Figures on this page show the complete Outer Collar detail plans.

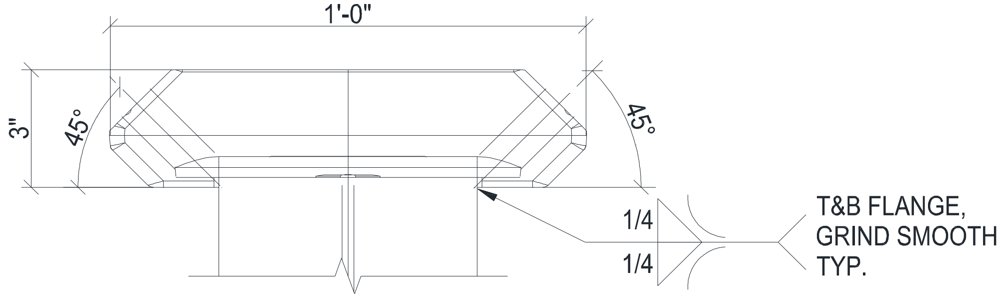


Figure 68: Top View of Collar Flange Top.

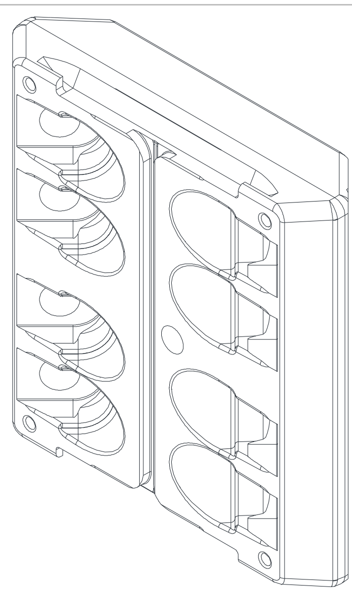


Figure 69: 3D-Perspective View of the ConXR-200 Outer Collar.

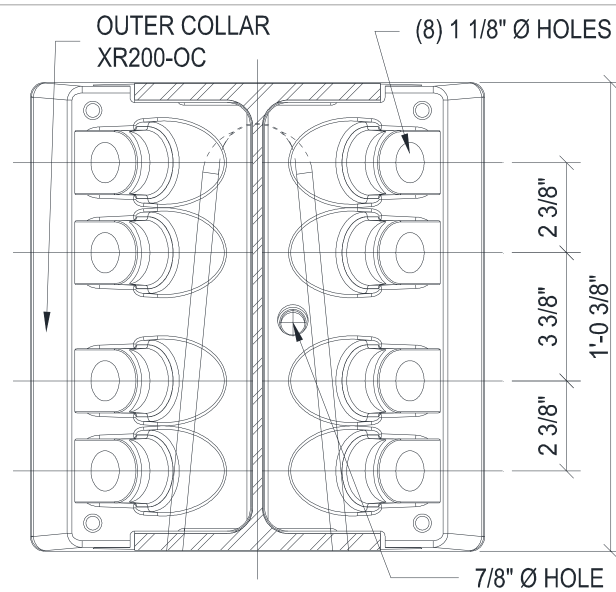


Figure 70: Outer Collar (XR200-OC).

OUTER COLLAR PLAN (BOTTOM)

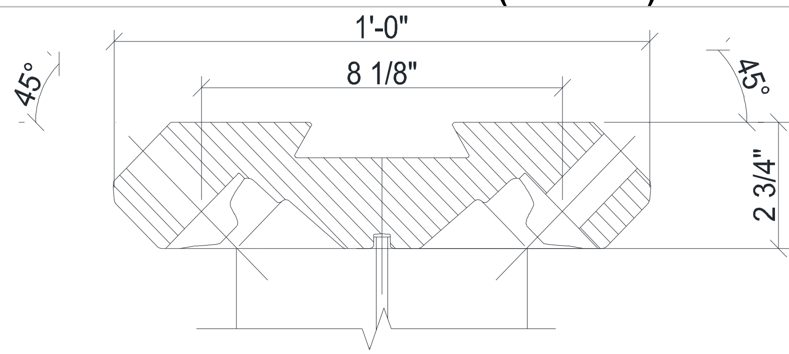


Figure 71: Top View of Outer Collar.

2.3.4 Beam-To-Outer Collar Assembly

The ConXR-200 Beam-to-Outer Collar assembly consists of a wide flange beam attached to the Outer Collar. The beam is attached to the Outer Collar via welds, as illustrated in the figures on this page.

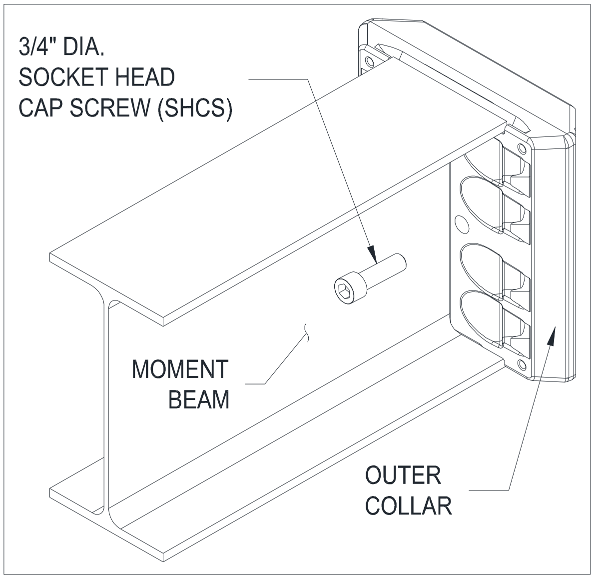


Figure 72: 3D-Perspective View of ConXR-200 Beam-To-Outer Collar Assembly.

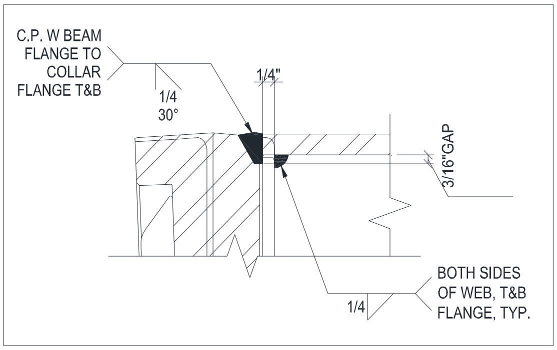


Figure 73: Beam Flange-To-Outer Collar Connection Detail.

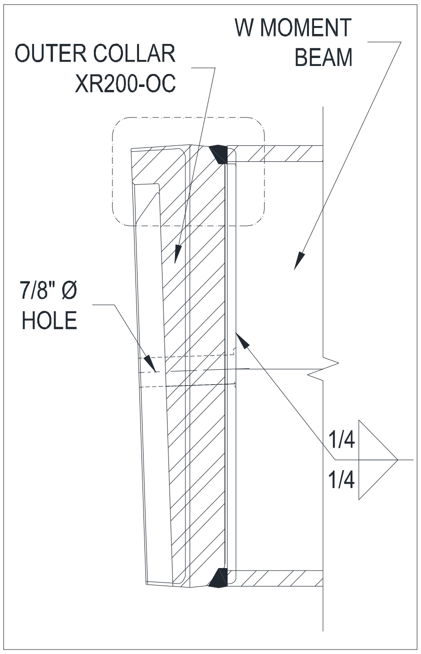


Figure 74: Beam-To-Outer Collar Connection Detail.

The figures on this page illustrate the three types of beam configurations used in the ConXR-200 system.

Going from top to bottom, Beam-Moment-Moment (BMM), Beam-Moment-Gravity (BMG), and Beam-Gravity-Gravity (BGG).

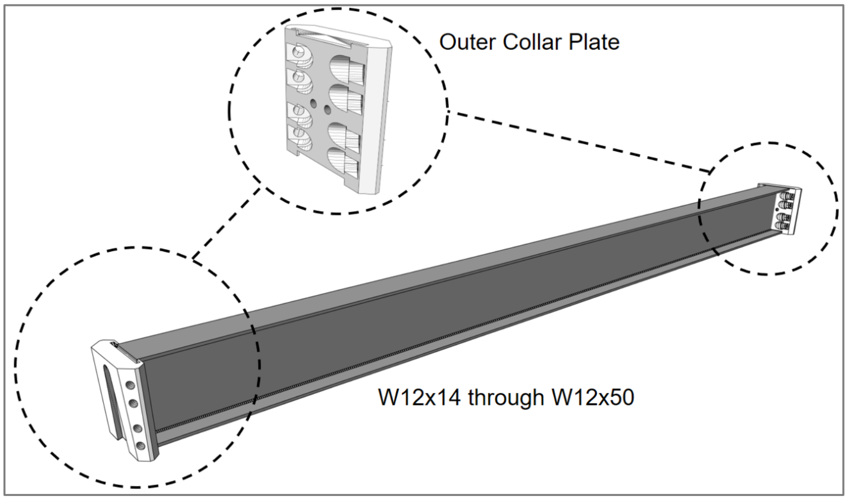


Figure 75: BMM.

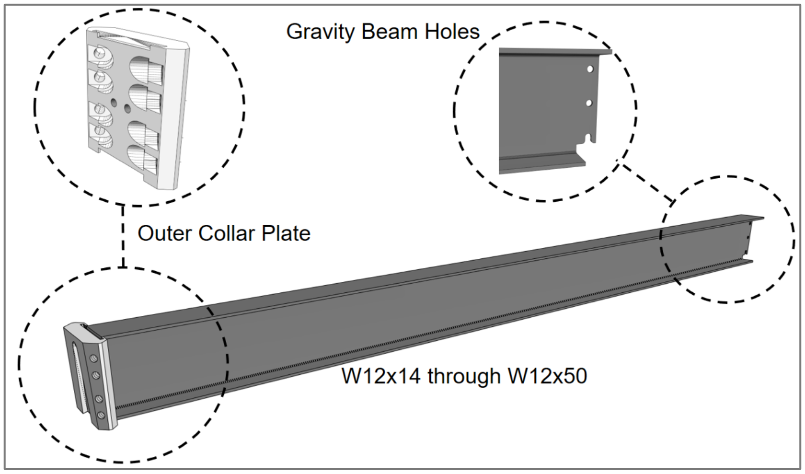


Figure 76: BMG.

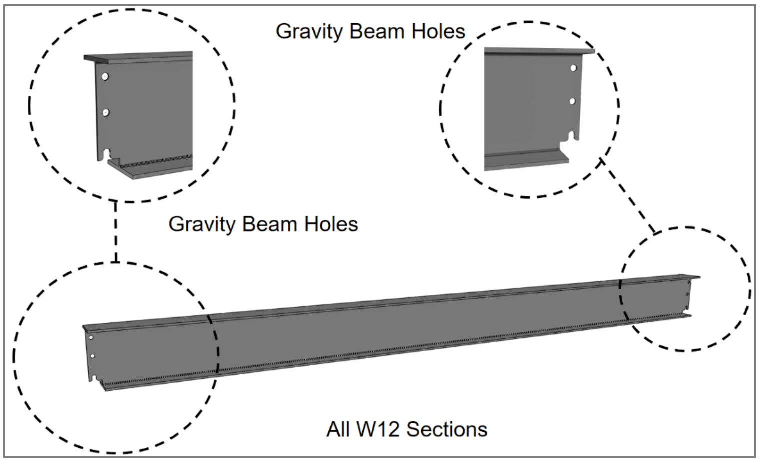


Figure 77: BGG.

2.3.5 Beam and Moment Collar Assembly

The figure below shows the plan and elevation standard details of the ConXR-200 moment connection.

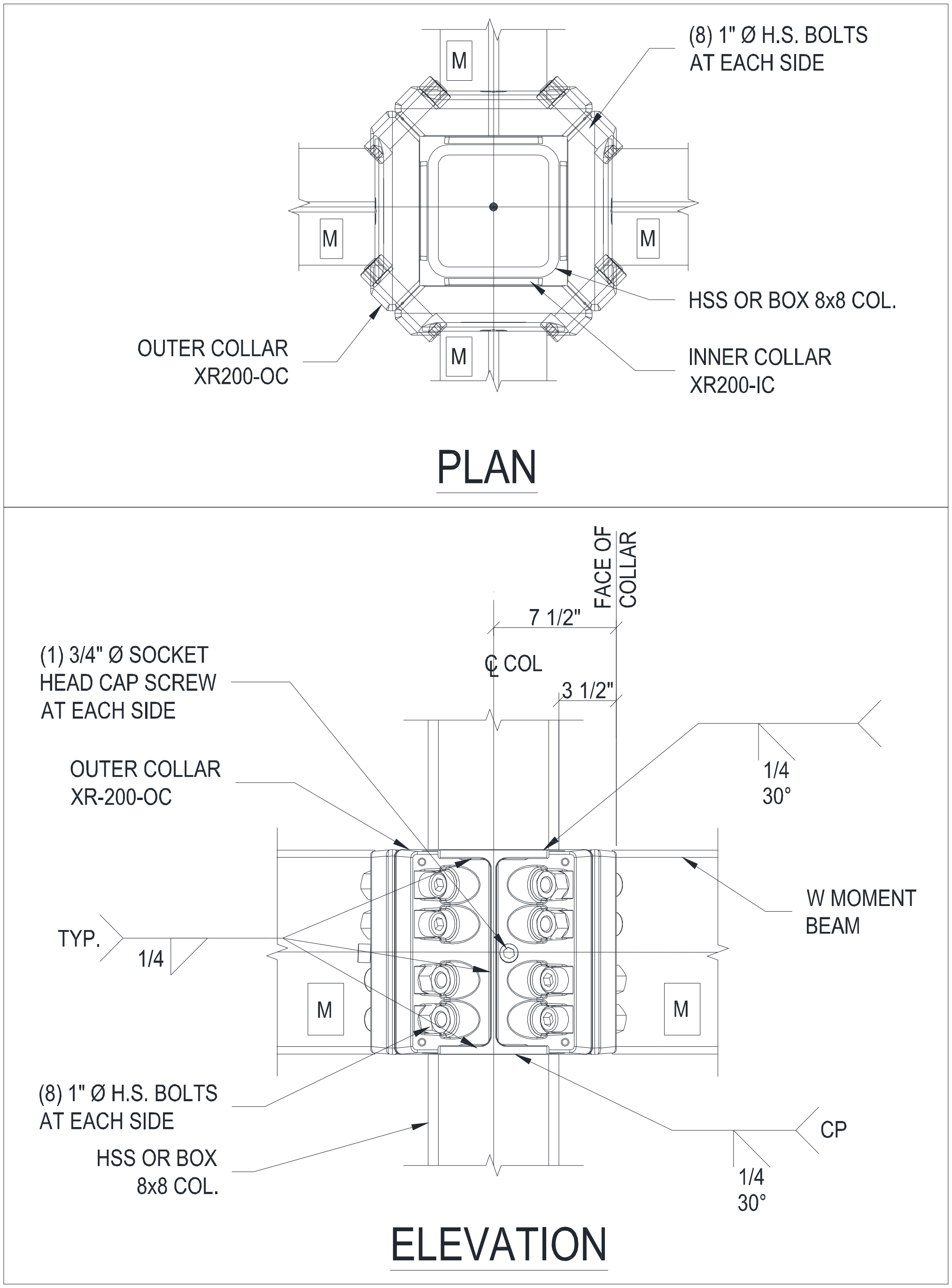


Figure 78: Elevation and Plan Views of the ConXR-200 Moment Connection.

2.3.6 Load Transfer Mechanism

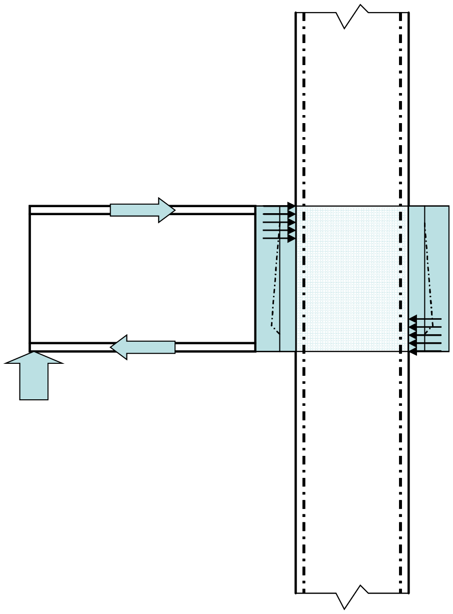


Figure 79: ConXR-200 Beam-To-Column Moment Transfer Mechanism.

Moment transfer between the beams and columns is accomplished through compressive bearing as illustrated in Figure 79.

Compressive flexural forces in the beam flange bear directly against the collar plates, which then transfer the force to the column through bearing. Tensile forces in the beam flanges are transferred to the OC through complete joint penetration groove welds to these plates.

The OC undergoes flexure from the tensile forces and transfers the forces to the Collar Bolts. The Collar Bolts (in tension) transfer these forces through the orthogonal OCs, which then transfer the load through the far set of Collar Bolts to the OC on the opposite face of the column. The OC on the opposite face of the column then bears against far side of the column.

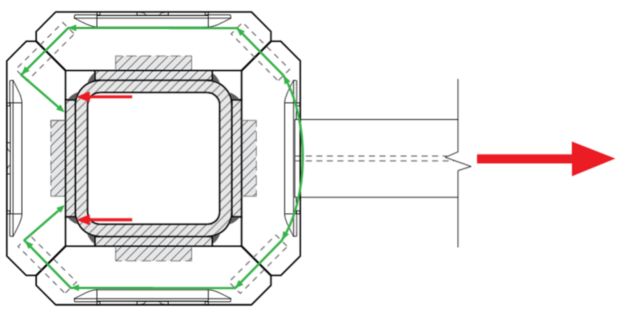


Figure 80: Load Path of Flange Tension Force to Column.

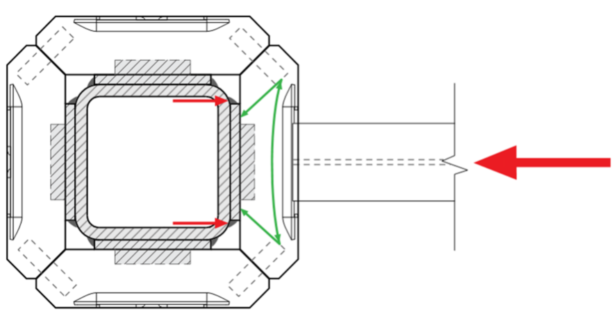


Figure 81: Load Path of Flange Compressive Force to Column.

2.3.7 Design Requirements

The design requirements detailed in this document for the ConXtech XR-200 moment connection pertain to the fundamental strength of the moment connection for non-seismic applications.

This document provides the design formulations for the following:

- Collar Flange Connection - Design and Allowable Flexural Strength.
- Bolted Connection - X-axis Design and Allowable Flexural Strength.
- Bolted Connection - Y-axis Design and Allowable Flexural Strength.
- Bolted Connection - Design and Allowable Strength for Combined Flexural, Shear, & Axial Load.
- Weld at Beam Web/Collar Web Extension - Design and Allowable Shear Strength.
- Weld at Collar Web Extension/Collar Flange - Design and Allowable Shear Strength.
- Weld at Collar Corner Assembly/Column - Design and Allowable Shear Strength.
- Friction at Collar Flange/Collar Corner - Design and Allowable Slip Resistance Strength.
- Panel Zone - Design and Allowable Shear Strength.
- Local Yielding of HSS/Box Column Sidewalls - Design and Allowable Strength.
- Local Crippling of HSS/Box Column Sidewalls - Design and Allowable Strength.

Connection Design Notes:
The connection design procedure presented below is based on *Specification*.

The following definitions are used in this procedure:

OC = Outer Collar: The machined plate that is welded to the end of a moment beam.

IC = Inner Collar: An Inner Collar is a machined plate that is welded to a face of the square tube column. Four Inner Collars for the four faces of a tube or box column form a collar joint.

Node = A complete assembly of a ConXR-200 Moment Collar: Four Inner Collars attached to a moment column and surrounded by Four Outer Collars.

Collar Joint = The intersection of moment beams and moment column occurs at a node/joint. A node will have four Outer Collars and four Inner Collars. A joint can have one to four moment beams, but it always has four Outer Collars and four Inner Collars. If a face of a moment column does not have a moment beam, there will be an Outer Collar without beam, called a blank.

M_{ux} = LRFD Load Combinations Required Flexural Strength – Beam X-axis, *k-ft* (kN-m)

M_{uy} = LRFD Load Combinations Required Flexural Strength – Beam Y-axis, *k-ft* (kN-m)

P_{ub} = LRFD Load Combinations Required Axial Strength – Beam, *kips* (kN)

V_{ux} = LRFD Load Combinations Required Shear Strength – Beam X-axis, kips (kN)

V_{uy} = LRFD Load Combinations Required Shear Strength – Beam Y-axis, kips (kN)

P_{uc} = LRFD Load Combinations Required Axial Strength – Column, *kips* (kN)

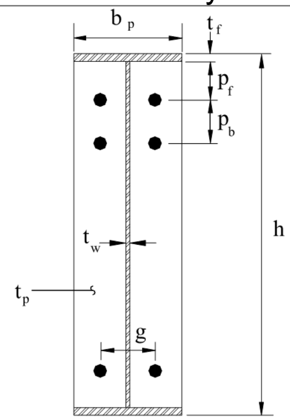
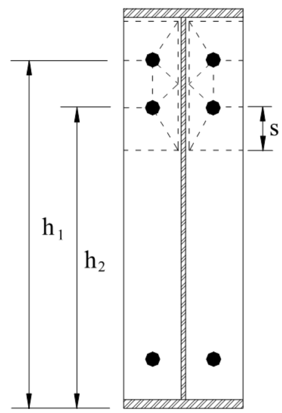
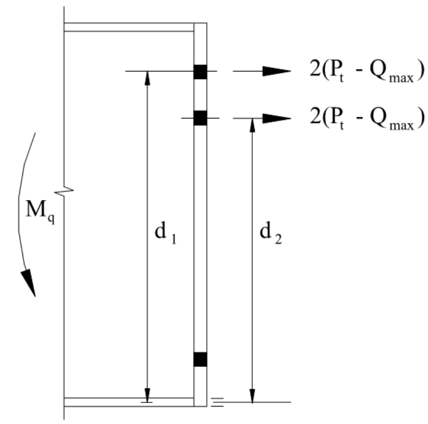
V_{uPZ} = LRFD Load Combinations Required Shear Strength – Panel Zone, *kips* (kN)

w_u = LRFD Load Combinations Uniform Applied Load – Beam, *k/ft* (kN/m)

2.3.8 Flexural Strength of Outer Collar

The ConXR-200 moment connection is designed similar to the Four-Bolt Flush Unstiffened Moment End-Plate Connection presented in the *AISC Steel Design Guide 16*. For reference, the table below is a summary of Four-Bolt Flush Unstiffened Moment End-Plate Analysis, per *AISC DG16*. The summary table is taken from Table 3-3 of the aforementioned Design Guide.

Table 4: Summary of Four-Bolt Flush Unstiffened Moment End-Plate Analysis

Geometry	Yield-Line Mechanism	Bolt Force Model
		

where

- b_p = width of end plate, *in*. (mm)
- t_f = thickness of beam flange, *in*. (mm)
- p_f = distance from the bolt centerline adjacent the beam tension flange to the near face of the beam tension flange, *in*. (mm)
- p_b = distance from bolt centerline to bolt centerline, *in*. (mm)
- t_w = thickness of beam web, *in*. (mm)
- g = bolt gage, *in*. (mm)
- t_p = thickness of plate, *in*. (mm)
- h = height of plate, *in*. (mm)
- h_1 = distance from the compression side of the beam to the farthest inner load-carrying bolt line, *in*. (mm)
- h_2 = distance from the compression side of the beam to the second farthest inner load-carrying bolt line, *in*. (mm)
- s = distance from the innermost bolt centerline to the innermost yield line, *in*. (mm)
- d_1 = distance from the center of the beam compression flange to the farthest inner load-carrying bolt line, *in*. (mm)
- d_2 = distance from the center of the beam compression flange to the second farthest inner load-carrying bolt line, *in*. (mm)
- P_t = bolt material ultimate tensile load capacity, proof load = $A_b \times F_t$, *kips* (kN)
- Q_{max} = maximum possible prying force, *kips* (kN)
- M_q = connection strength for the limit state of bolt fracture with prying action, *kip-in*. (kN-m)

The figure below shows the ConXR-200 Outer Collar idealized as a flush end-plate. For calculations, the effective thickness of the Outer Collar is conservatively taken as 1.25 *in*.

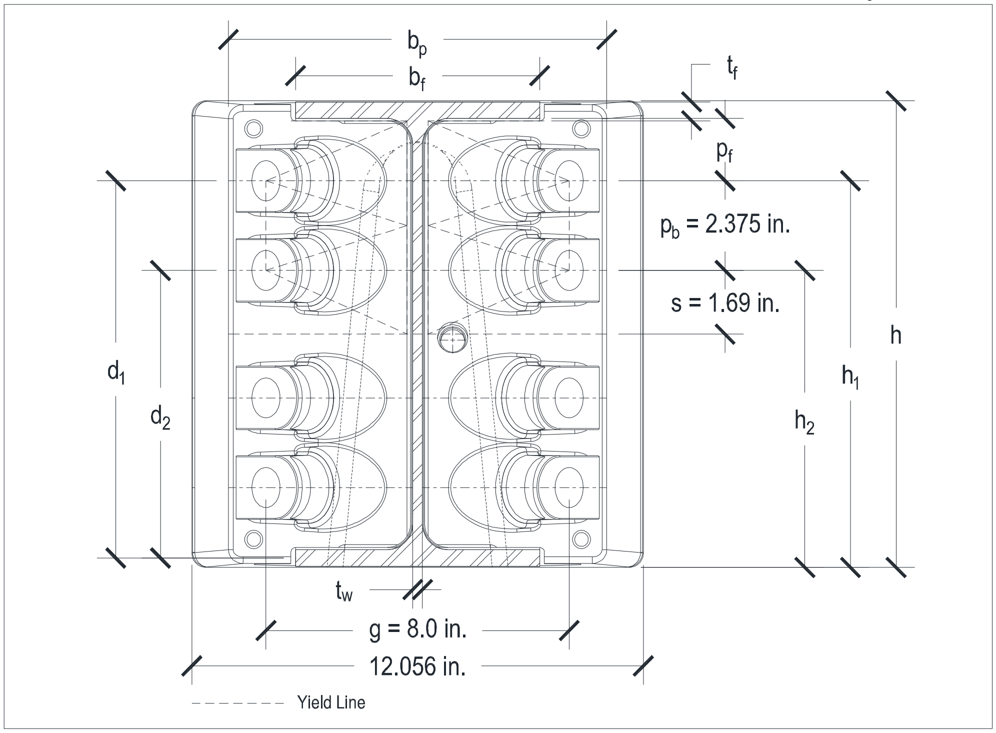


Figure 82: ConXR-200 Outer Collar Under Strong-Axis Flexural Loading.

Definition of Parameters:

b_p = width of end plate, *in*. (mm)

p_f = distance from the bolt centerline adjacent the beam tension flange to the near face of the beam tension flange, *in*. (mm)

p_b = distance from bolt centerline to bolt centerline, *in*. (mm)

b_f = width of flange, *in*. (mm)

t_f = thickness of beam flange, *in*. (mm)

t_w = thickness of beam web, *in*. (mm)

h = height of plate, *in*. (mm)

h_1 = distance from the compression side of the beam to the farthest inner load-carrying bolt line, *in*. (mm)

h_2 = distance from the compression side of the beam to the second farthest inner load-carrying bolt line, *in*. (mm)

g = bolt gage, *in*. (mm)

s = distance from the innermost bolt centerline to the innermost yield line, *in*. (mm)

d_1 = distance from the center of the beam compression flange to the farthest inner load-carrying bolt line, *in*. (mm)

d_2 = distance from the center of the beam compression flange to the second farthest inner load-carrying bolt line, *in*. (mm)

d_b = diameter of bolts = 1.0 *in*. (25.4 mm)

From Figure 82 on the previous page,	
$h = d =$ beam depth $d_b =$ diameter of bolts $= 1.0\ in.$ $g =$ distance between bolts $= 8.0\ in.$ $b_p = 10.10\ in.$ $p_b = 2.375\ in.$ $s = 1.69\ in.$	$p_f = \dfrac{d - 2t_f - 2(p_b + s)}{2}$ Note: Use $p_f = s$ if $p_f > s$. $h_1 = d - t_f - p_f$ $h_2 = h_1 - p_b = h_1 - 2.38\ in.$ $d_1 = d - 1.5t_f - p_f$ $d_2 = d_1 - p_b$ $y_r = 1.25$

Chapter 2.5 of the *AISC Design Guide 16* states that the threshold when prying action begins to take place in the bolts is at 90% of the full strength of the plate, or 0.90M_{pl}. This means that if the applied load is less than 0.90M_{pl}, the end-plate behaves like a thick plate and prying action can be neglected in the bolts.

In accordance with Chapter 3 of the *AISC Design Guide 16*, "for either ASD Type 1 or LRFD rigid frame construction, the required factored moment, M_u, must be increased 25% to limit the connection rotation at ultimate moment to 10% of the simple span beam rotation. Therefore, the factor y_r = 1.25 is used in the procedure for the flush connection plate design.

Per *AISC Design Guide 16*, the required end-plate thickness, $t_{p,reqd}$, is calculated based on Equation (2-7) of the Design Guide, shown below:

LRFD	ASD
$t_{p,reqd} = \sqrt{\dfrac{(1.11)\,y_r\,(\phi M_{np})}{\phi_b F_{py} Y}}$	$t_{p,reqd} = \sqrt{\dfrac{(1.67)\,y_r}{\Omega_b F_{py} Y}\left(\dfrac{M_{np}}{\Omega}\right)}$

where

$$Y = \frac{b_p}{2}\left(h_1\left(\frac{1}{p_f}\right) + h_2\left(\frac{1}{s}\right)\right) + \frac{2}{g}\left(h_1\left(p_f + 0.75p_b\right) + h_2\left(s + 0.25p_b\right)\right) + \frac{g}{2}$$

For the ConXR-200 Moment Connection, the effective thickness of the Outer Collar is known and therefore the above equation is re-arranged to obtain the design and the allowable flexural strength of the plate (Outer Collar), as shown below:

LRFD	ASD
$\phi M_{np} = \dfrac{\left(t_{p,reqd}\right)^2 \phi_b F_{py} Y}{(1.11)\,y_r}$	$\dfrac{M_{np}}{\Omega} = \dfrac{\left(t_{p,reqd}\right)^2 \Omega_b F_{py} Y}{(1.67)\,y_r \Omega}$

The Demand Capacity Ratio for the design and allowable flexural strength of the Outer Collar is calculated as shown below:

LRFD	ASD
$DCR = \dfrac{M_{ux}}{\phi M_{np}}$	$DCR = \dfrac{M_{ux}}{\left(\dfrac{M_{np}}{\Omega}\right)}$

2.3.9 Bolted Connection X-Axis Flexural Strength

As stated previously in this document, the ConXR-200 collar flanges transfer a beam's moment into the moment collar through pretensioned bolts. Because the size and number of bolts in the connection are known, the maximum moment capacity of the connection can be determined in terms of the strength of the bolts.

A moment load in the strong-axis of the connected beam resolves into tension and compression forces within the beam's flange.

As can be seen in the figures below, the bolts are oriented 45 degrees relative to the longitudinal axis of the connected moment beam. Therefore, a tensile force from a beam's flange can be resolved into a shear force and a tension force in the bolt, as shown below.

$$f_r \cos\left(45^\circ\right) = f_v = f_t$$

$$f_r = \sqrt{f_t^2 + f_v^2} = \sqrt{f_t^2 + f_t^2} = \sqrt{2} f_t$$

The following section determines the maximum available bolt load, considering the effects of combined loading as stipulated in the *Specification*.

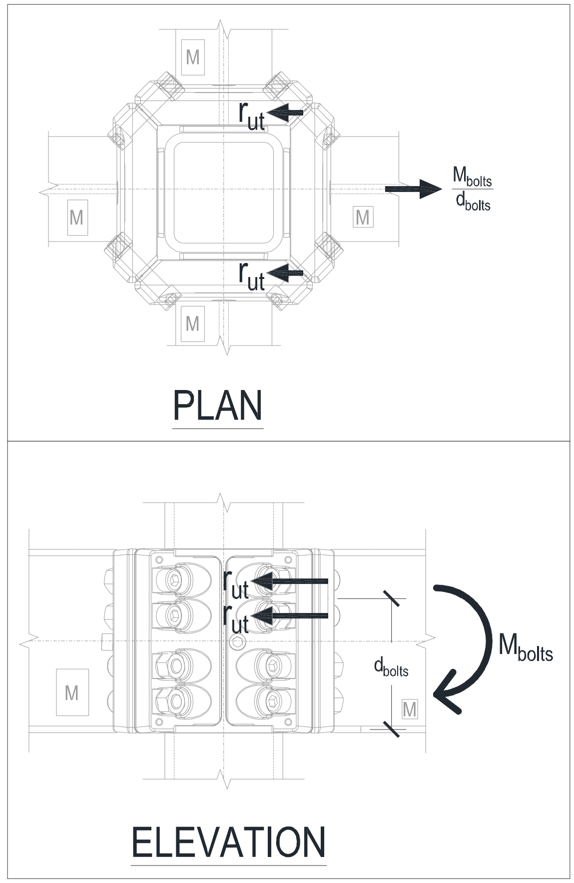


Figure 83: ConXR-200 Bolted Moment Connection Under Flexural Load.

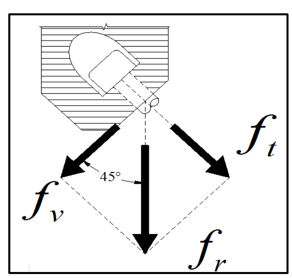


Figure 84: Bolt Load Resolved into a Tensile and Shear Force Components.

For the formulations that follow, the definitions below apply:

f_t = required tensile stress, *ksi* (MPa)
f_v = required shear stress, *ksi* (MPa)
f_t = f_v (bolts are 45° to applied load)
f_r = required resultant stress resisting applied load, *ksi* (MPa)
F_{nt} = nominal tensile stress = 0.75F_u, *ksi* (MPa) (*Specification* C-J3-2)
F_{nv} = nominal shear stress = 0.563F_u, *ksi* (MPa) (*Specification* C-J3-3)
F_u = specified minimum tensile strength of bolt, *ksi* (MPa)
ϕ_{bolts} = 0.75, resistance factor for bolts
Ω_{bolts} = 2.00, safety factor for bolts
R_n = nominal strength of bolt resisting applied load, *kips* (kN)
A_b = nominal unthreaded body area of bolt, *in.*^{2}

Continuing from the previous page,

LRFD	ASD
Specification (C-J3-5a) $\left(\frac{f_t}{\phi_{bolts}F_{nt}}\right)^2+\left(\frac{f_v}{\phi_{bolts}F_{nv}}\right)^2=1$	*Specification* (C-J3-5b) $\left(\frac{\Omega_{bolts}f_t}{F_{nt}}\right)^2+\left(\frac{\Omega_{bolts}f_v}{F_{nv}}\right)^2=1$
$\left(\frac{f_t}{\phi_{bolts}0.75F_{nt}}\right)^2+\left(\frac{f_v}{\phi_{bolts}0.563F_{nv}}\right)^2=1$ $\left(\frac{f_t}{\phi_{bolts}0.75F_u}\right)^2+\left(\frac{f_t}{\phi_{bolts}0.563F_u}\right)^2=1$ $\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{1}{0.75^2}+\frac{1}{0.563^2}\right)=1$ $\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{0.563^2}{0.75^2\cdot 0.563^2}+\frac{0.75^2}{0.563^2\cdot 0.75^2}\right)=1$ $\left(\frac{f_t}{\phi_{bolts}F_u}\right)^2\left(\frac{0.563^2+0.75^2}{(0.563)^2(0.75)^2}\right)=1$ $f_t=\sqrt{\frac{(0.563)^2(0.75)^2}{\left(0.563^2+0.75^2\right)}\cdot\left(\phi_{bolts}F_u\right)^2}$ $f_t=0.45\left(\phi_{bolts}F_u\right)$	$\left(\frac{\Omega_{bolts}f_t}{0.75F_u}\right)^2+\left(\frac{\Omega_{bolts}f_v}{0.563F_u}\right)^2=1$ $\left(\frac{\Omega_{bolts}f_t}{0.75F_u}\right)^2+\left(\frac{\Omega_{bolts}f_t}{0.563F_u}\right)^2=1$ $\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{1}{0.75^2}+\frac{1}{0.563^2}\right)=1$ $\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{0.563^2}{0.75^2\cdot 0.563^2}+\frac{0.75^2}{0.563^2\cdot 0.75^2}\right)=1$ $\left(\frac{\Omega_{bolts}f_t}{F_u}\right)^2\left(\frac{0.563^2+0.75^2}{(0.563)^2(0.75)^2}\right)=1$ $f_t=\sqrt{\frac{(0.563)^2(0.75)^2}{\left(0.563^2+0.75^2\right)}\cdot\left(\frac{F_u}{\Omega_{bolts}}\right)^2}$ $f_t=0.45\left(\frac{F_u}{\Omega_{bolts}}\right)$

The nominal tensile capacity of a bolt is shown below:

$$R_n = f_r A_b = \sqrt{2} f_t A_b = \sqrt{2}\left(0.45\right) F_u A_b = 0.636 F_u A_b$$

The design tension and the allowable tension of a bolt is determined as shown below:

$\phi_{bolts} = 0.75$ $\phi_{bolts} R_n = \phi_{bolts} f_r A_b = \phi_{bolts} \sqrt{2} f_t A_b$ $\phi_{bolts} R_n = \left(0.75\right)\sqrt{2}\left(0.45\right) F_u A_b = 0.477 F_u A_b$	$\Omega_{bolts} = 2.00$ $\dfrac{R_n}{\Omega_{bolts}} = \dfrac{f_r A_b}{\Omega_{bolts}} = \dfrac{\sqrt{2} f_t A_b}{\Omega_{bolts}}$ $\dfrac{R_n}{\Omega_{bolts}} = \dfrac{\sqrt{2}\left(0.45\right) F_u A_b}{2.00} = 0.318 F_u A_b$

It follows that the nominal flexural strength of the bolted moment connection is:

$$M_{ntx} = 0.636 F_u A_b \left(\frac{n_b}{2}\right) d_{bolts}$$

where

$F_u =$ specified minimum tensile strength of bolt, *ksi* (MPa)

$n_b =$ number of bolts in connection = 8

$A_b =$ nominal unthreaded body area of bolt = 0.785 *in.*^{2} (506.7 mm^2); diameter of bolt is 1.0 *in* (25.4 mm)

The average distance between the center of the compression flange and the bolts under tension is as shown below. Therefore, the design and allowable flexural strength of the bolted connection is as shown below:

LRFD	ASD
$\phi_{bolts} M_{ntx} = \phi_{bolts} 0.636 F_u A_b \left(\dfrac{n_b}{2}\right) d_{bolts}$ $\phi_{bolts} M_{ntx} = 0.477 F_u A_b \left(\dfrac{n_b}{2}\right) d_{bolts}$	$\dfrac{M_{ntx}}{\Omega_{bolts}} = \dfrac{0.636 F_u A_b}{\Omega_{bolts}} \left(\dfrac{n_b}{2}\right) d_{bolts}$ $\dfrac{M_{ntx}}{\Omega_{bolts}} = 0.318 F_u A_b \left(\dfrac{n_b}{2}\right) d_{bolts}$

where $\quad d_{bolts} = \dfrac{d_1 + d_2}{2}$

and where (from Section 2.3.8) $\quad d_1 = d - 1.5 t_f - p_f$

$$d_2 = d_1 - p_b$$

It follows that the Demand Capacity Ratio for the design and allowable flexural strength of the bolted connection is as shown below:

LRFD	ASD
$DCR = \dfrac{M_{ux}}{\phi_{bolts} M_{ntx}}$	$DCR = \dfrac{M_{ux}}{\left(M_{ntx} / \Omega_{bolts} \right)}$

2.3.10 Bolted Connection Y-Axis Flexural Strength

This section discusses the weak-axis flexural mechanics of the ConXR-200 collar connection. The weak-axis flexural capacity of the bolted connection will be determined.

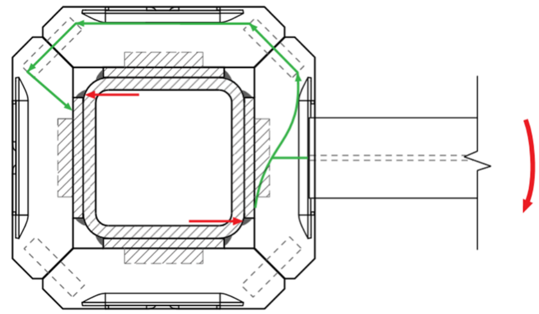


Figure 85: Load Path of Y-Axis Bending Moment Through Collars and Bolts to Column.

A moment load in a connected beam's weak-axis causes flexure in the Outer Collar attached to the beam, as depicted in Figure 85. The flexural load in the Outer Collar causes tension on one side of the Outer Collar and compression on the other side of the Outer Collar. The tensile force in the Outer Collar is transferred into the Collar Assembly through the pretensioned bolts.

The compression force causes bearing between the back of the Outer Collar and the front of the Inner Collar, as illustrated in Figure 85

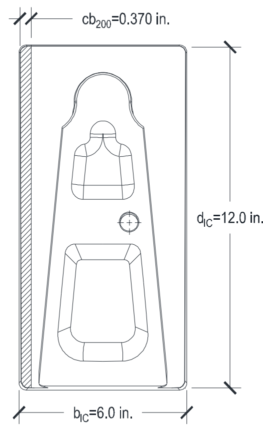


Figure 86: Cross-Section View of a ConXR-200 Collar Flange in Weak-Axis Flexure.

Where

d_{IC} = effective depth of Inner Collar for Y-Axis bolt moment
b_{IC} = effective width of Inner Collar for Y-Axis bolt moment
cb_{200} = width of compression block for Y-Axis bolt moment,

The width of the compression block in the Collar Flange is calculated by setting the bolt strength equal to the bearing strength of the Inner Collar. The ConXR-200 system can accept High Strength or Super High Strength bolts. The depth of the compression block in the Inner Collar is a function of the bolt strength. The stronger the bolts are, the deeper the depth of the compression block, and therefore, the smaller the moment arm. Super High Strength bolts have a specified minimum tensile strength of 200 *ksi*. Thus, in the calculations that follow, an ultimate strength F_u of 200 *ksi* (1,379 MPa) is assumed for the bolts because this is conservative in determining the Y-Axis flexural capacity of the bolted connection.

Calculations of the width of the collar compression block are shown below. The bolt tension load is as follows:

LRFD	ASD
$F_u = 200\ ksi\ \ (1{,}379\text{ MPa})$ $A_b = 0.785\ in.^2\ \ (506.7\text{ mm}^2)$ $n_b = 8$ $\phi_{bolts} R_n = 0.477 F_u A_b \left(\frac{n_b}{2}\right)$ $= 0.477(200\ ksi)(0.785\ in.^2)\left(\frac{8}{2}\right)$ $= 299.6\ kips\ \ (1{,}333\text{ kN})$	$F_u = 200\ ksi\ \ (1{,}379\text{ MPa})$ $A_b = 0.785\ in.^2\ \ (506.7\text{ mm}^2)$ $n_b = 8$ $\frac{R_n}{\Omega_{bolts}} = 0.318 F_u A_b \left(\frac{n_b}{2}\right)$ $= 0.318(200\ ksi)(0.785\ in.^2)\left(\frac{8}{2}\right)$ $= 199.7\ kips\ \ (888.3\text{ kN})$

In accordance with *Specification* Chapter J7, the nominal bearing strength, R_n, for finished (milled) surfaces is as shown below:

$$R_n = 1.8 F_y A_{pb} \qquad \text{(J7-1)}$$

$$\phi = 0.75\ \ (\text{LRFD}) \qquad \Omega = 2.00\ \ (\text{ASD})$$

where

F_y = specified minimum yield stress, *ksi* (MPa)
A_{pb} = projected bearing area, *in.*^{2} (mm^2)

The width of the compression block in the collar is determined by setting the bolt tension load equal to the bearing strength of the Inner Collar, as shown below.

LRFD	ASD
$F_y = 50\ ksi\ \left(345\ \text{MPa}\right)$ $d_{CF} = 12.0\ in.\ \left(305\ \text{mm}\right)$ $A_{pb} = d_{IC} cb_{200}$ $\phi R_n = \phi 1.8 F_y A_{pb} = 299.6\ kips$ $0.751.8\left(50\ ksi\right)\left(12.0\ in.\right) cb_{200} = 299.6\ kips$ $cb_{200} = 0.370\ in.\ \left(9.40\ \text{mm}\right)$	$F_y = 50\ ksi\ \left(345\ \text{MPa}\right)$ $d_{CF} = 12.0\ in.\ \left(305\ \text{mm}\right)$ $A_{pb} = d_{IC} cb_{200}$ $\dfrac{R_n}{\Omega} = \dfrac{1.8 F_y A_{pb}}{\Omega} = 199.7\ kips$ $1.8\left(50\ ksi\right)\left(12.0\ in.\right) cb_{200} = 2.00\left(199.7\ kips\right)$ $cb_{200} = 0.370\ in.\ \left(9.40\ \text{mm}\right)$

Now that the depth of the compression stress block has been determined, the Y-Axis flexural strength of the bolted connection can be calculated. The design or allowable Y-Axis flexural strength of the bolted connection is as shown below:

LRFD	ASD
$b_{IF} = 6\ in.\ \left(152.4\ \text{mm}\right)$ $g = 8.0\ in.\ \left(203.2\ \text{mm}\right)$ $cb_{200} = 0.370\ in.\ \left(9.40\ \text{mm}\right)$ $\phi_{bolts} M_{nty} = 0.477 F_u A_b \left(\dfrac{n_b}{2}\right)\left(\dfrac{b_{IF}}{2} + \dfrac{g}{2} - \dfrac{cb_{200}}{2}\right)$	$b_{IF} = 6\ in.\ \left(152.4\ \text{mm}\right)$ $g = 8.0\ in.\ \left(203.2\ \text{mm}\right)$ $cb_{200} = 0.370\ in.\ \left(9.40\ \text{mm}\right)$ $\dfrac{M_{nty}}{\Omega_{bolts}} = 0.318 F_u A_b \left(\dfrac{n_b}{2}\right)\left(\dfrac{b_{IF}}{2} + \dfrac{g}{2} - \dfrac{cb_{200}}{2}\right)$

The Demand Capacity Ratio for the design or allowable Y-Axis flexural strength of the bolted connection is as shown below:

LRFD	ASD
$\text{DCR} = \dfrac{M_{uy}}{\phi_{bolts} M_{nty}}$	$\text{DCR} = \dfrac{M_{uy}}{\left(M_{nty} / \Omega_{bolts} \right)}$

2.3.11 Bolted Connection: Combined Flexural, Shear (X-Axis), and Axial Loading

Where there is a combined flexural (in both X and Y axes), shear (X-axis only), and axial load on the bolted moment connection, the Demand Capacity Ratio is determined as shown below:

LRFD
$DCR = \dfrac{M_{ux}}{\phi_{bolts} M_{ntx}} + \left(\dfrac{V_{ux}}{\phi R_{ncf}} \right)^2 + \dfrac{M_{uy}}{\phi_{bolts} M_{nty}} + \dfrac{P_{ub}}{n_b \phi_{bolts} R_n}$
ASD
$DCR = \dfrac{M_{ux}}{M_{ntx} / \Omega_{bolts}} + \left(\dfrac{V_{ux}}{R_{ncf} / \Omega} \right)^2 + \dfrac{M_{uy}}{M_{nty} / \Omega_{bolts}} + \dfrac{P_{ub}}{n_b R_n / \Omega_{bolts}}$

where

V_{ux} = required strong-axis beam shear load from load combinations.

P_{ub} = required beam axial load from load combinations.

R_{ncf} = available slip resistance from Section 2.3.17.

2.3.12 X-Axis Shear Transfer from Beam to Moment Collar

In the ConXR-200 moment collar connection, the shear load path involves the beam, Outer Collar, Inner Collar, and the moment column. Shear force from the moment beam is transferred into the Outer Collar via fillet welds between the beam web and the Outer Collar. When the moment collar is fully assembled but before the High Strength bolts have been pretensioned, downward shear forces from the beam are transferred from the Outer Collar to the Inner Collar. Upward shear forces are resisted by the 3/4 *in*. diameter horizontal bolt that connects the Outer Collar to the Inner Collar. When the moment collar is fully assembled and all the High Strength bolts have been pretensioned, both upward and downward shear forces from the beam are resisted by the clamping force between the Outer Collar and the Inner Collar caused the pretensioning in the bolts.

The strength of the aforementioned welds is calculated in accordance with *Specification* Chapter J2.4, as follows:

$$R_n = F_{nw} A_{we} \qquad \text{(J2-4)}$$

$$\phi = 0.75 \ \ (\text{LRFD}) \qquad \Omega = 2.00 \ \ (\text{ASD})$$

For fillet welds,

$$F_{nw} = 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5}(\theta)\right) \qquad \text{(J2-5)}$$

where

F_{EXX} = filler metal classification strength, *ksi* (MPa)
θ = angle of loading measured from the weld longitudinal axis, degrees
A_{we} = effective area of the weld, *in*.^{2} (mm^2)

The proceeding pages show calculations to determine the strength of the beam web to Collar Web Extension welds, Collar Web Extension to Collar Flange welds and Collar Corners to Column welds.

2.3.13 Weld at Beam Web to Outer Collar

The shear strength of the fillet welds between the Beam Web and the Collar Web Extension (CWX) is determined as follows:

The ConXR-200 beam web to Outer Collar connection uses a 1/4 *in*. (6.35 mm) fillet weld on each side of the beam web, as shown in Figure 87.

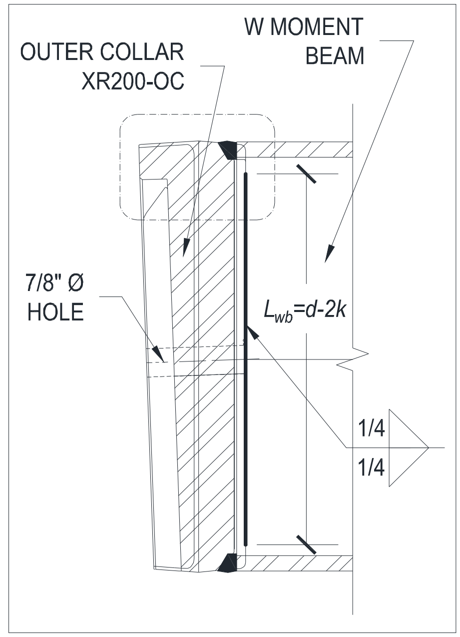


Figure 87: Beam Web-To-Outer Collar Welds.

The length of the fillet weld is:

$$l_{wb} = 2(d - 2k)\ in.\ (\mathrm{mm})$$

The size of the fillet weld is:

$$t_{we} = \frac{\sqrt{2}}{2} \cdot \frac{1}{4}\ in. = 0.177\ in.$$

The area of the fillet weld is:

$$A_{we} = t_{we} l_{wb}$$

The strength of the fillet weld is:

$$\theta = 0^\circ$$
$$F_{EXX} = 70\,ksi \;\; (483\text{ MPa})$$

$$R_n = 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right)A_{we}$$
$$= 0.60(70\,ksi)\left(1.0 + 0.50\sin^{1.5}(0^\circ)\right)(0.177\,in.)2(d - 2k)$$
$$= 0.60(70\,ksi)(0.354\,in.)(d - 2k)$$

$$R_n = \left(14.87\frac{kip}{in.}\right)(d - 2k)$$

The design and allowable shear strength of the Beam Web to Outer Collar fillet welds is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(14.87\frac{kip}{in.}\right)(d-2k)$ ENGLISH $\phi R_n = \left(11.1\frac{kip}{in.}\right)(d-2k)$ METRIC $\phi R_n = \left(1.94\frac{\text{kN}}{\text{mm}}\right)(d-2k)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{\left(14.87\frac{kip}{in.}\right)(d-2k)}{2.00}$ ENGLISH $\frac{R_n}{\Omega} = \left(7.42\frac{kip}{in.}\right)(d-2k)$ METRIC $\frac{R_n}{\Omega} = \left(1.30\frac{\text{kN}}{\text{mm}}\right)(d-2k)$

The Demand Capacity Ratio (DCR) for the design and allowable shear strength of the Beam Web to Outer Collar fillet welds is calculated as shown below:

LRFD	ASD
$\text{DCR} = \frac{V_{ux}}{\phi R_n}$	$\text{DCR} = \frac{V_{ux}}{\left(R_n/\Omega\right)}$

2.3.14 Weld at Beam Flange to Outer Collar

The ConXR-200 Beam Flange to Outer Collar connection is a Complete Joint penetration (CJP) weld from the beam flange to the Outer Collar, as can be seen in Figure 88.

A flexural load from the beam resolves into a tension and compression force couple within the beam flanges.

The CJP weld transfers the tensile force from the beam flange to the Outer Collar. The CJP weld provides the full strength of the connected parts.

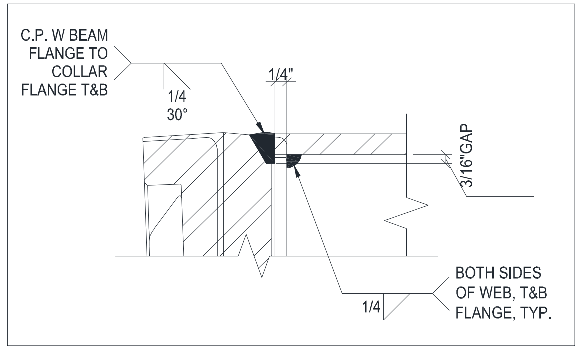


Figure 88: Beam Flange - To - Outer Collar Welds.

2.3.15 Weld at Inner Collar and Column (X-Axis Shear)

The ConXR-200 Inner Collars are attached to the moment column via a 1/4 *in*. (6.35 mm) fillet weld on two sides of the Inner Collar, as illustrated in the two figures below.

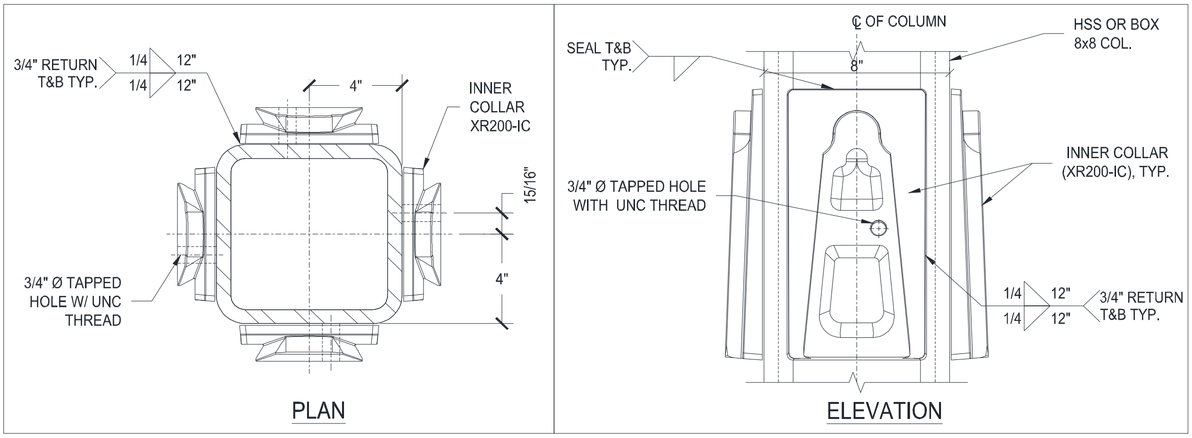


Figure 89: Inner Collar-To-Column Welds.

As can be seen in the details above, the total length of each fillet weld is:

$$l_{we} = 12\,in.$$

The size of each fillet weld is:

$$t_{we} = \frac{\sqrt{2}}{2} \cdot \frac{1}{4}\,in.$$

The area of the fillet welds is:

$$A_{we} = t_{we} l_{we}$$

The strength of the fillet weld is:

$$\theta = 0^{\circ}$$

$$R_{nwx} = 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5}(\theta)\right) A_{we}$$

$$= 0.60(70\,ksi)\left(1.0 + 0.50 \sin^{1.5}(0^{\circ})\right) 2\left(\frac{\sqrt{2}}{2} \cdot \frac{1}{4}\,in.\right)(12\,in.)$$

$$= 178.2\,kips$$

$$R_{nwx} = 178.2\,kips$$

The design and allowable shear strength of the fillet welds that join the Inner Collar to the column is as shown below:

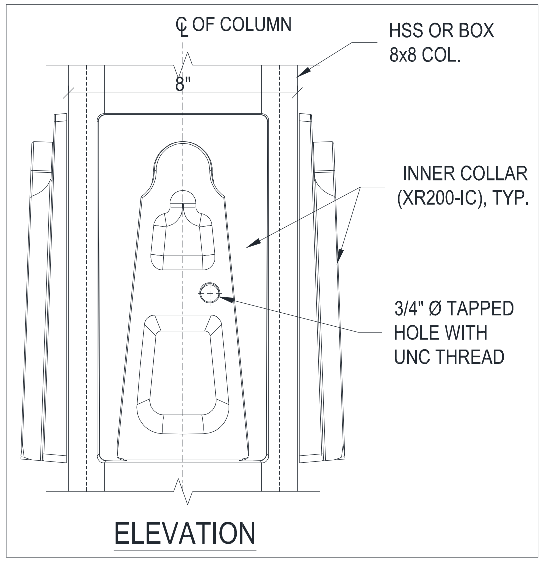
LRFD	ASD
$\phi = 0.75$ $\phi R_{nwx} = 0.75(178.2\ kips)$ $\phi R_{nwx} = 133.7\ kips\ \ (594.7\ \mathrm{kN})$	$\Omega = 2.00$ $\dfrac{R_{nwx}}{\Omega} = \dfrac{178.2\ kips}{2.00}$ $\dfrac{R_{nwx}}{\Omega} = 89.10\ kips\ \ (396.3\ \mathrm{kN})$

The Demand Capacity Ratio (DCR) for the design and allowable shear strength of the fillet welds that join the Inner Collar to the column is as shown below:

LRFD	ASD
ENGLISH $\mathrm{DCR} = \dfrac{V_{ux}}{\phi R_{nwx}} = \dfrac{V_{ux}}{133.7\ kips}$ METRIC $\mathrm{DCR} = \dfrac{V_{ux}}{\phi R_{nwx}} = \dfrac{V_{ux}}{594.7\ \mathrm{kN}}$	ENGLISH $\mathrm{DCR} = \dfrac{V_{ux}}{\left(R_{nwx}/\Omega\right)} = \dfrac{V_{ux}}{89.10\ kips}$ METRIC $\mathrm{DCR} = \dfrac{V_{ux}}{\phi R_{nwx}} = \dfrac{V_{ux}}{396.3\ \mathrm{kN}}$

2.3.16 Shear Yielding and Shear Rupture in Inner Collar

In accordance with *Specification* Chapter J4.2, the available shear strength of the affected and connecting elements shall be the lower value obtained according to the limit states of shear yielding and shear rupture:



c. For shear yielding of the element:

$$R_n = 0.60 F_y A_{gv} \quad \text{(J4-3)}$$

$\phi = 1.00$ (LRFD) $\quad \Omega = 1.50$ (ASD)

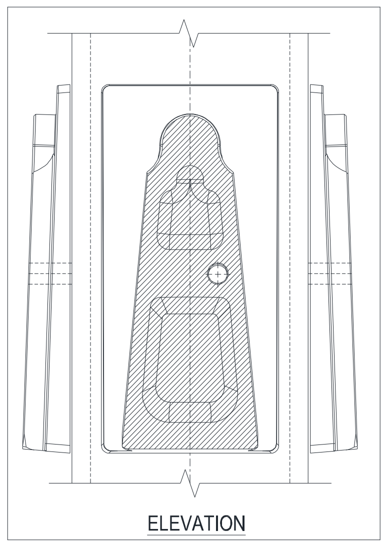
d. For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \quad \text{(J4-4)}$$

$\phi = 0.75$ (LRFD) $\quad \Omega = 2.00$ (ASD)

In the ConXR-200 connection, a shear load goes from the moment beam into the Outer Collar and finally into the Inner Collar. The Inner Collar has a smaller shear area than the Outer Collar, and therefore is the governing element between the Collars. The Inner and Outer Collars are made from ASTM A572 Grade 50 material. This material has the following specifications:

$F_y =$ 50 ksi
$F_u =$ 65 ksi



Shear Yielding in the Inner Collar.

The hatched area in the figure on the right shows the shear area of the Inner Collar. The hatched region shown in the Inner Collar has an area of 37 $in.^2$

For shear yielding in the gross-section:

$$d_h = d_b + \frac{1}{16}\ in. = 0.69\ in.$$

$$A_h = \pi\frac{(0.69\ in.)^2}{4} = 0.37\ in.^2$$

$$A_{gv} = 37\ in.^2 - 0.37\ in.^2 = 36.6\ in.^2$$

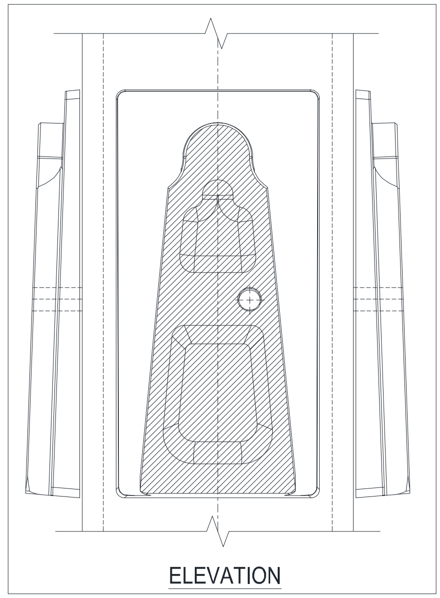
$$R_n = 0.60F_yA_{gv}$$
$$= 0.60(50\ ksi)\left(36.6\ in.^2\right) = 1{,}098\ kips$$

The available design or allowable shear yielding strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(1{,}098\ kips)$ ENGLISH $\phi R_n = 1{,}098\ kips$ METRIC $\phi R_n = 4{,}884\ \mathrm{kN}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{1{,}098\ kips}{1.50} = 732\ kips$ ENGLISH $\frac{R_n}{\Omega} = 732\ kips$ METRIC $\frac{R_n}{\Omega} = 3{,}256\ \mathrm{kN}$

The hatched area in the figure on the right shows the shear plane of the net section. In conformance with *Specification* Chapter B4.3b, in computing the net area for tension and shear, the width of a bolt hole shall be taken as 1/16 *in*. (2 mm) greater than the nominal dimension of the hole.

The length of the shear plane, L, is taken as the nominal beam depth, d_{nom}, for conservatism. The average thickness of the CCA was previously calculated to be 1.41 *in*. (35.8 mm).



Shear Rupture in the Inner Collar.

For shear rupture in the *net*-section:

$$d_{h\_net} = d_h + \frac{1}{16}\,in. = \frac{3}{4}\,in. + \frac{1}{16}\,in. = 0.81\,in.$$

$$A_{h\_net} = \pi\frac{(0.81\,in.)^2}{4} = 0.52\,in.^2$$

$$A_{nv} = 37\,in.^2 - 0.52\,in.^2 = 36.5\,in.^2$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(65\,ksi)(36.5\,in.^2) = 1{,}424\,kips \end{aligned}$$

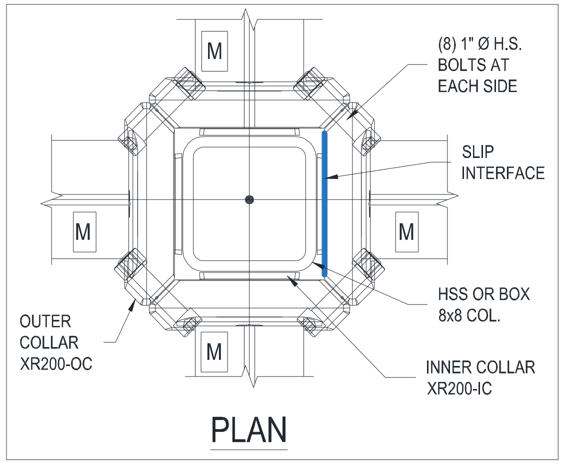
The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(1{,}424\,kips) = 1{,}068\,kips$ ENGLISH $\phi R_n = 1{,}068\,kips$ METRIC $\phi R_n = 4{,}750\text{ kN}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{1{,}424\,kips}{2.00} = 712\,kips$ ENGLISH $\frac{R_n}{\Omega} = 712\,kips$ METRIC $\frac{R_n}{\Omega} = 3{,}167\text{ kN}$

Note: Based on the preceding calculations, shear rupture is the governing limit state of the two.

2.3.17 Friction between Outer Collar and Inner Corner (X-Axis Shear)

The next step in the design process is to determine the slip resistance of the friction force at the interface between the Outer Collar and the Inner Collar. In accordance with *Specification* Chapter J3.8, the design slip resistance, ϕR_n, and the allowable slip resistance, R_n/Ω, shall be determined for the limit state of slip as follows:



For standard size and short-slotted holes perpendicular to the direction of the load,

$$\phi = 1.00 \ \text{(LRFD)} \qquad \Omega = 1.50 \ \text{(ASD)}$$

For the limit state of slip resistance, the following definitions apply:

μ = mean slip coefficient for Class A or B surfaces. The Inner Collar and Outer Collar have Class A surfaces and therefore μ = 0.30.

D_u = 1.13, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension.

T_b = minimum fastener tension given in *Specification* Table J3.1, *kips*, or Table J3.1M, kN. As specified in the ConXR-200 Standard Details, the 1.0 *in*. ASTM A574 bolts are pretensioned to the min. bolt pretension of ASTM A490 bolts. Per *Specification* Table J3.1, T_b = 64 *kips* (285 kN).

h_f = factor for filler. There are no fillers present, thus, h_f =1.0

n_s = number of slip planes. For this case, n_s = 1.0. Only one slip plane is used for each bolt because only the side pertaining to one beam is considered.

n_b = number bolts. For this case, n_b = 8.

Using the information above, the slip resistance is determined as follows:

$$T_b = 64 \ kips$$

$$R_n = \left(\mu D_u h_f T_b n_s\right) n_b$$

$$R_{ncf} = 0.30(1.13)(1.0)(64 \ kips)(1.0)(8) = 173.6 \ kips$$

The design slip resistance, ϕR_n, and the allowable slip resistance, R_n/Ω, is as shown below:

LRFD	ASD
$\phi = 1.00$ $\phi R_{ncf} = 1.00(173.6\ kips)$ $\phi R_{ncf} = 173.6\ kips\ \ (772.2\ \text{kN})$	$\Omega = 1.50$ $R_{ncf} / \Omega = 173.6\ kips / 1.50$ $R_{ncf} / \Omega = 115.7\ kips\ \ (514.6\ \text{kN})$

The Demand Capacity Ratio (DCR) for the shear strength of the friction force at the interface between the Outer Collar and the Inner Collar is as shown below:

LRFD	ASD
$DCR = \dfrac{V_{ux}}{\phi R_{ncf}}$ ENGLISH $DCR = \dfrac{V_{ux}}{173.6\ kips}$ METRIC $DCR = \dfrac{V_{ux}}{772.2\ \text{kN}}$	$DCR = \dfrac{V_{ux}}{\left(R_{ncf} / \Omega \right)}$ ENGLISH $\text{DCR} = \dfrac{V_{ux}}{115.7\ kips}$ METRIC $\text{DCR} = \dfrac{V_{ux}}{514.6\ \text{kN}}$

2.3.18 Y-Axis Shear Transfer from Beam to Column

In the ConXR-200 moment collar connection, weak-axis shear is transferred to the Outer Collar via CJP welds between the beam flanges and the Outer Collar. The Outer Collar transfers the shear load via direct bearing against the Inner Collar. Due to the inclined surface of the shear lug on the Inner Collar, the weak-axis shear will also induce an uplift force on the connection, which is resisted by clamping force between the Outer Collar and the Inner Collar. The Inner Collar transfers the shear load to the column via fillet welds between the Inner Collar and the column.

2.3.19 Bearing Strength at Inner Collar and Outer Collar

In accordance with *Specification* Section J7, the available bearing strength of surfaces in contact shall be determined for the limit state of bearing as follows:

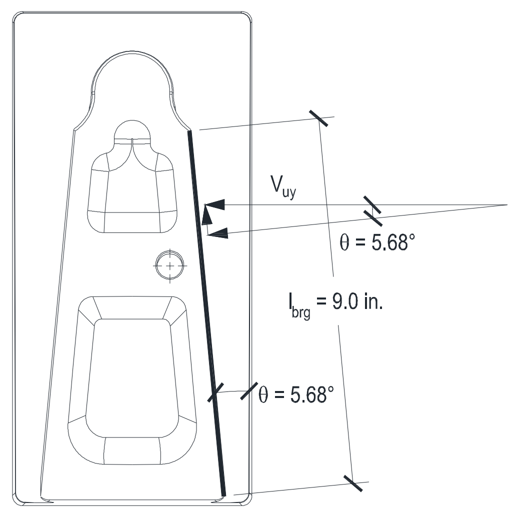
$$R_n = 1.8 F_y A_{pb} \quad \text{(J7-1)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

The Inner Collar and Outer Collar are made from ASTM A572 Grade 50 material. This material has the following specifications:

$F_y =$ 50 *ksi*
$F_u =$ 65 *ksi*

As shown in the figure on the right, one component of the weak-axis shear is resisted by bearing of the Outer Collar against the inclined surface of the shear lug. The angle of inclination, θ, is 5.68°.



Elevation of Inner Collar showing orientation of bearing force.

The dimensions of the bearing area are based on the Inner Collar,

where:

l_{brg} = length of bearing along the shear lug
= 9.00-*in.* (see figure above).
t_{brg} = thickness of the shear lug
= 3/4-*in.* from Figure 64 (see page 133).

For bearing at the shear lug:

$$A_{pb} = l_{brg} t_{brg} = (9.00\,in.)(0.750\,in.)$$
$$= 6.750\,in.^2$$

$$R_{nb} = 1.8 F_y A_{pb}$$
$$= 1.8(50\,ksi)(6.750\,in.^2)$$
$$= 607.5\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{nb} = 0.75(607.5\,kips)$ ENGLISH $\phi R_{nb} = 455.6\,kips$ METRIC $\phi R_{nb} = 2,027\,\text{kN}$	$\Omega = 2.00$ $\frac{R_{nb}}{\Omega} = \frac{607.5\,kips}{2.00}$ ENGLISH $\frac{R_{nb}}{\Omega} = 303.8\,kips$ METRIC $\frac{R_{nb}}{\Omega} = 1,351\,\text{kN}$

The Demand Capacity Ratio for the bearing strength of the shear lug is shown below:

LRFD	ASD
$DCR = \frac{V_{uy}\cos(5.68°)}{\phi R_{nb}}$	$DCR = \frac{V_{uy}\cos(5.68°)}{R_{nb}/\Omega}$

where

$V_{uy} =$ required weak-axis beam shear load from load combinations.

2.3.20 Friction between Outer Collar and Inner Collar (Y-Axis Shear)

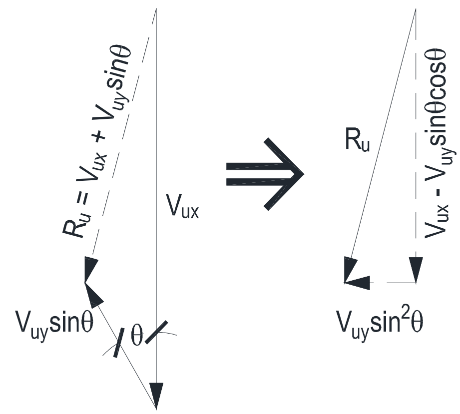
The uplift force induced by the weak-axis shear is resisted by friction at the interface between the Outer Collar and the Inner Collar. The slip resistance was previously determined in Section 2.3.17; note that the magnitude of slip resistance is the same regardless of the loading direction.

The Demand Capacity Ratio (DCR) for the shear strength of the friction force at the interface between the Outer Collar and the Inner Collar is as shown below:

LRFD	ASD
$DCR = \dfrac{V_{uy}\sin(5.68°)}{\phi R_{ncf}}$ ENGLISH $DCR = \dfrac{V_{uy}\sin(5.68°)}{173.6\ kips}$ METRIC $DCR = \dfrac{V_{uy}\sin(5.68°)}{772.2\ \mathrm{kN}}$	$DCR = \dfrac{V_{uy}\sin(5.68°)}{\left(R_{ncf}/\Omega\right)}$ ENGLISH $\mathrm{DCR} = \dfrac{V_{uy}\sin(5.68°)}{115.7\ kips}$ METRIC $\mathrm{DCR} = \dfrac{V_{uy}\sin(5.68°)}{514.6\ \mathrm{kN}}$

2.3.21 Friction: Combined Shear Loading

Where there is a combined shear load in both X and Y axes, the net shear load resisted by friction at the interface between the Outer Collar and the Inner Collar is the vector sum of V_{ux} and the uplift component of V_{uy}. V_{ux} is oriented downward vertically while the uplift component of V_{uy} is oriented at 5.68° from the vertical; as such, the uplift component of V_{uy} must be broken into horizontal and vertical components.



With the downward direction for vertical components taken as positive, the vector sum of the forces is determined as follows.

$$
\begin{aligned}
\langle R_u \rangle &= \langle V_{ux} \rangle + \langle V_{uy}\sin\theta \rangle \\
&= \langle 0,\ V_{ux} \rangle + \langle (V_{uy}\sin\theta)\sin\theta,\ (-V_{uy}\sin\theta)\cos\theta \rangle \\
&= \langle V_{uy}\sin^2\theta,\ V_{ux} - V_{uy}\sin\theta\cos\theta \rangle
\end{aligned}
$$

The magnitude of this force is determined as follows:

$$
\begin{aligned}
R_u^{\,2} &= (V_{ux} - V_{uy}\sin\theta\cos\theta)^2 + (V_{uy}\sin^2\theta)^2 \\
&= V_{ux}^{\,2} - 2V_{ux}V_{uy}\sin\theta\cos\theta + V_{uy}^{\,2}(\sin^2\theta)(\cos^2\theta) + V_{uy}^{\,2}\sin^4\theta \\
&= V_{ux}^{\,2} - V_{ux}V_{uy}\sin(2\theta) + V_{uy}^{\,2}\left((\sin^2\theta)(\cos^2\theta) + (\sin^2\theta)^2\right) \\
&= V_{ux}^{\,2} - V_{ux}V_{uy}\sin(2\theta) + V_{uy}^{\,2}\left(\left(\frac{1-\cos(2\theta)}{2}\right)\left(\frac{1+\cos(2\theta)}{2}\right) + \left(\frac{1-\cos(2\theta)}{2}\right)^2\right) \\
&= V_{ux}^{\,2} - V_{ux}V_{uy}\sin(2\theta) + V_{uy}^{\,2}\left(\frac{1-\cos^2(2\theta)}{4} + \frac{1-2\cos(2\theta)+\cos^2(2\theta)}{4}\right) \\
&= V_{ux}^{\,2} - V_{ux}V_{uy}\sin(2\theta) + V_{uy}^{\,2}\left(\frac{1-\cos(2\theta)}{2}\right) \\
&= V_{ux}^{\,2} - V_{ux}V_{uy}\sin(2\theta) + V_{uy}^{\,2}\sin^2\theta
\end{aligned}
$$

$$R_u = \sqrt{V_{ux}^{\,2} - V_{ux}V_{uy}\sin(11.36°) + (V_{uy}\sin(5.68°))^2}$$

The Demand Capacity Ratio for the shear strength of the combined friction force at the interface between the Outer Collar and the Inner Collar is determined as shown below:

LRFD
$DCR = \dfrac{\sqrt{V_{ux}{}^{2} - V_{ux}V_{uy}\sin\left(11.36°\right) + \left(V_{uy}\sin\left(5.68°\right)\right)^{2}}}{\phi R_{ncf}}$
ASD
$DCR = \dfrac{\sqrt{V_{ux}{}^{2} - V_{ux}V_{uy}\sin\left(11.36°\right) + \left(V_{uy}\sin\left(5.68°\right)\right)^{2}}}{R_{ncf}/\Omega}$

2.3.22 Bolted Connection: Combined Flexural, Shear (both Axes), and Axial Loading

Where there is a combined flexural (in both X and Y axes), shear (in both axes), and axial load on the bolted moment connection, the Demand Capacity Ratio is determined as shown below:

LRFD
$DCR = \dfrac{M_{ux}}{\phi_{bolts} M_{ntx}} + \dfrac{V_{ux}{}^2 - V_{ux} V_{uy} \sin(2\theta) + \left(V_{uy} \sin(\theta)\right)^2}{\left(\phi R_{ncf}\right)^2} + \dfrac{M_{uy}}{\phi_{bolts} M_{nty}} + \dfrac{P_{ub}}{n_b \phi_{bolts} R_n}$
ASD
$DCR = \dfrac{M_{ux}}{M_{ntx}/\Omega_{bolts}} + \dfrac{V_{ux}{}^2 - V_{ux} V_{uy} \sin(2\theta) + \left(V_{uy} \sin(\theta)\right)^2}{\left(R_{ncf}/\Omega\right)^2} + \dfrac{M_{uy}}{M_{nty}/\Omega_{bolts}} + \dfrac{P_{ub}}{n_b R_n/\Omega_{bolts}}$

where

V_{ux} = required strong-axis beam shear load from load combinations.

V_{uy} = required weak-axis beam shear from load combinations.

P_{ub} = required beam axial load from load combinations.

R_{ncf} = available slip resistance from Section 2.3.17.

θ = angle of inclination of the shear lug bearing surface from Section 2.3.19
= 5.68°.

2.3.23 Weld at Inner Collar and Column (Y-Axis Shear)

The ConXR-200 Inner Collars are attached to the moment column via a 1/4 *in*. (6.35 mm) fillet weld on two sides of the Inner Collar. The weak-axis shear is oriented 90° to the longitudinal axes of these welds.

The total length of each fillet weld is:

$$l_{we} = 12\,in.$$

The size of each fillet weld is:

$$t_{we} = \frac{\sqrt{2}}{2} \cdot \frac{1}{4}\,in.$$

The area of the fillet welds is:

$$A_{we} = t_{we} l_{we}$$

The strength of the fillet weld is:

$$\theta = 0^{\circ}$$

$$\begin{aligned} R_{nwy} &= 0.60 F_{EXX}\left(1.0 + 0.50 \sin^{1.5}(\theta)\right) A_{we} \\ &= 0.60(70\,ksi)\left(1.0 + 0.50 \sin^{1.5}(90^{\circ})\right) 2\left(\frac{\sqrt{2}}{2} \cdot \frac{1}{4}\,in.\right)(12\,in.) \\ &= 267.3\,kips \end{aligned}$$

$$R_{nwy} = 267.3\,kips$$

The design and allowable shear strength of the fillet welds that join the Inner Collar to the column is as shown below:

LRFD	ASD
$\phi = 0.75$ $\phi R_{nwy} = 0.75(267.3\ kips)$ $\phi R_{nwy} = 200.5\ kips\ \ (891.7\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_{nwy}}{\Omega} = \dfrac{267.3\ kips}{2.00}$ $\dfrac{R_{nwy}}{\Omega} = 133.6\ kips\ \ (594.5\ \text{kN})$

The Demand Capacity Ratio (DCR) for the design and allowable shear strength of the fillet welds that join the Inner Collar to the column is as shown below:

LRFD	ASD
ENGLISH $\text{DCR} = \dfrac{V_{uy}}{\phi R_{nwy}} = \dfrac{V_{uy}}{200.5\ kips}$ METRIC $\text{DCR} = \dfrac{V_{uy}}{\phi R_{nwy}} = \dfrac{V_{uy}}{891.7\ \text{kN}}$	ENGLISH $\text{DCR} = \dfrac{V_{uy}}{\left(R_{nwy}/\Omega\right)} = \dfrac{V_{uy}}{133.6\ kips}$ METRIC $\text{DCR} = \dfrac{V_{uy}}{\phi R_{nwy}} = \dfrac{V_{uy}}{594.5\ \text{kN}}$

2.3.24 Weld at Inner Collar and Column: Combined Shear Loading

Where there is a combined shear load in both X and Y axes, the Demand Capacity Ratio is determined as shown below:

LRFD	ASD
$DCR = \sqrt{\left(\frac{V_{ux}}{\phi R_{nwx}}\right)^2 + \left(\frac{V_{uy}}{\phi R_{nwy}}\right)^2}$	$DCR = \sqrt{\left(\frac{V_{ux}}{R_{nwx}/\Omega}\right)^2 + \left(\frac{V_{uy}}{R_{nwy}/\Omega}\right)^2}$

where

R_{nwx} = available strong-axis shear strength from Section 2.3.15.

R_{nwy} = available weak-axis shear strength from Section 2.3.23.

2.3.25 Panel Zone Shear Strength

The final step in the ConXR-200 moment connection design process is to determine the shear strength of the Panel Zone.

Panel Zone shear is resisted by the column walls and the Outer Collars. The Outer Collars act as reinforcing plates for the column webs.

In accordance with *Specification* Section J10.6, the available strength of the web panel zone for the limit state of shear yielding shall be determined as follows:

$$\phi = 0.90 \ \ (\text{LRFD}) \qquad \Omega = 1.67 \ \ (\text{ASD})$$

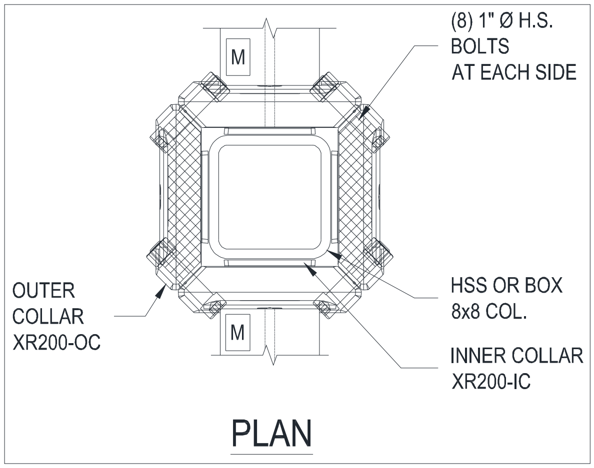


Figure 90: ConXR-200 Plan View at Panel Zone.

In the figure on the right, the cross-hatched regions represent the area effective in resisting panel zone shear (in the direction parallel to the connected beams).

The nominal strength, R_n, shall be determined as follows: when the effect of panel-zone deformation on frame stability is not considered in the analysis:

$$\text{For } P_r \le 0.4P_c \qquad R_n = 0.60F_y d_c t_w \qquad \text{(J10-9)}$$

$$\text{For } P_r > 0.4P_c \qquad R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \qquad \text{(J10-10)}$$

where

F_y = specified minimum yield stress of the column web, *ksi* (MPa)

d_c = depth of column, *in*. (mm)

t_w = thickness of column web, *in*. (mm)

P_r = required axial strength using LRFD or ASD load combinations, *kips* (kN)

P_c = P_y (LRFD) or 0.60P_y (ASD), *kips* (kN)

P_y = F_yA_g, axial yield strength of the column, *kips* (kN)

A_g = gross cross-sectional area of column, *in*.^{2} (mm^2)

The Outer Collars contribute to the steel area resisting the panel zone shear. The properties of the column and the Outer Collars pertaining to panel zone shear are shown below:

$A_{pz} =$ $2d_c t_c + 2d_{OC} t_{OC}$
$t_c =$ thickness of column wall, in. (mm)
$d_c =$ depth of column, 8 *in*. (203.2 mm)
$d_{OC} =$ effective depth of Outer Collar, 8 *in*. (203.2 mm)
$t_{OC} =$ effective thickness of Outer Collar, 2 *in*. (50.8 mm)

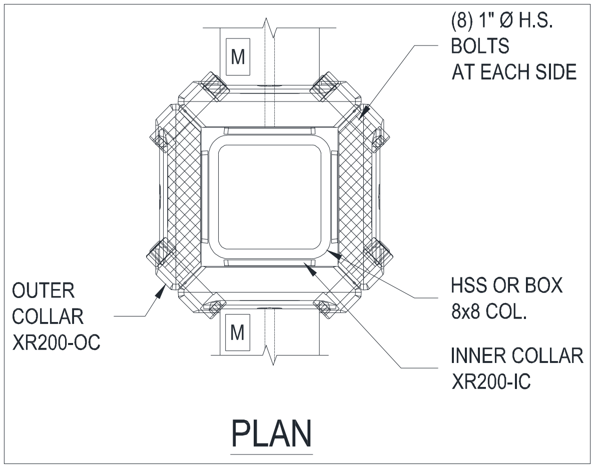


Figure 91: Effective Shear Areas of Column and Collar Corners.

The shear strength of the panel zone is as follows:

$$\text{For } P_r \le 0.4P_c \qquad R_{nPZ} = 0.60F_y\left(2d_c t_c + 2d_{OC} t_{OC}\right)$$

$$\text{For } P_r > 0.4P_c \qquad R_{nPZ} = 0.60F_y\left(2d_c t_c + 2d_{OC} t_{OC}\right)\left(1.4 - \frac{P_r}{P_c}\right)$$

The design panel zone shear strength, ϕR_n, and the allowable panel zone shear strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi R_{nPZ} = 0.90R_{nPZ}$	$\frac{R_{nPZ}}{\Omega} = \frac{R_{nPZ}}{1.67}$

The Demand Capacity Ratio for the panel zone shear strength is shown below:

LRFD	ASD
$DCR = \frac{V_{uPZ}}{\phi R_{nPZ}}$	$DCR = \frac{V_{uPZ}}{\left(R_{nPZ}/\Omega\right)}$

2.3.26 Local Yielding of HSS Sidewalls

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In accordance with Section J10.2 of the *Specification*, the limit state of Web Local Yielding (i.e., Local Yielding of HSS Sidewalls) is determined as follows:

$$R_n = F_{yw} t_w \left(5k + l_b\right) \quad \text{(J10-2)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

F_{yw} = specified minimum yield stress of HSS material, *ksi* (MPa)

l_b = bearing length perpendicular to the width of the Inner Collar

k = outside HSS corner radius (0 for Box)

$$F_y = 50\,ksi \quad (345\,MPa)$$
$$t_w = t_{des}$$
$$l_b = d_{CF} = 5.0\,in.$$
$$k = 1.5t_{des}$$

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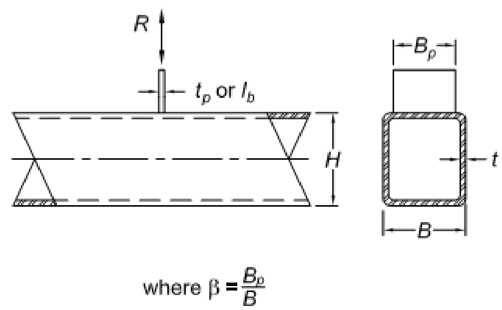
Connection Type

Transverse Plate T- and Cross-Connections, Under Plate Axial Load



Note: since HSS members have two webs, the available strength resulting from Eq. J10-2 will be multiplied by 2.

Note: Eq. J10-2 only applies when the concentrated load is applied at a distance from the member end that is greater than the nominal depth of the HSS (i.e., 8 *in.*). If the concentrated load is applied at a distance from the member end that is less than or equal to the nominal depth of the HSS (e.g., if the collar is near the top of the column), then Eq. J10-3 is used instead as demonstrated on the following pages.

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The total bearing area of four bolts at full strength was calculated on page 152; thus, the bearing length perpendicular to the width of the Inner Collar may be determined as follows:

$$A_{pb} = d_{IC} c b_{200} = (12\,in.)(0.370\,in.) = 4.44\,in^2$$

$$l_b = \frac{A_{pb}}{b_{IC}} = \frac{4.44\,in.^2}{6.0\,in.} = 0.74\,in.$$

The available strength is therefore calculated as follows:

$$R_n = F_{yw} t_w \left(5k + l_b\right) \times 2\, walls$$
$$= \left(50\, ksi\right)\left(t_{des}\right)\left(5\left(1.5 t_{des}\right) + 0.74\, in.\right) \times 2\, walls$$
$$= \left(100\, ksi\right) t_{des} \left(7.5 t_{des} + 0.74\, in.\right)$$

The design wall local yielding strength, ϕR_n, and the allowable wall local yielding strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00\left(100\ ksi\right) t_{des} \left(7.5 t_{des} + 0.74\ in.\right)$ ENGLISH $\phi R_n = \left(100\ ksi\right) t_{des} \left(7.5 t_{des} + 0.74\ in.\right)$ METRIC $\phi R_n = \left(689\ \text{MPa}\right) t_{des} \left(7.5 t_{des} + 18.8\ \text{mm}\right)$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{\left(100\ ksi\right) t_{des} \left(7.5 t_{des} + 0.74\ in.\right)}{1.50}$ ENGLISH $\dfrac{R_n}{\Omega} = \left(66.7\ ksi\right) t_{des} \left(7.5 t_{des} + 0.74\ in.\right)$ METRIC $\dfrac{R_n}{\Omega} = \left(460\ \text{MPa}\right) t_{des} \left(7.5 t_{des} + 18.8\ \text{mm}\right)$

When the concentrated force is applied at a distance from the member end that is less than or equal to the nominal depth of the HSS (i.e., 8 *in.*), the coefficient on *k* decreases to 2.5 (Eq. J10-3); for example, this condition should be checked when the collar is located near the top of the column. The design wall local yielding strength, ϕR_n, and the allowable wall local yielding strength, R_n/Ω, are then calculated as follows:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(100\ ksi)t_{des}\left(2.5(1.5t_{des}) + 0.74\ in.\right)$ ENGLISH $\phi R_n = (100\ ksi)t_{des}\left(3.75t_{des} + 0.74\ in.\right)$ METRIC $\phi R_n = (689\ \text{MPa})t_{des}\left(3.75t_{des} + 18.8\ \text{mm}\right)$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{(100\ ksi)t_{des}\left(2.5(1.5t_{des}) + 0.74\ in.\right)}{1.50}$ ENGLISH $\dfrac{R_n}{\Omega} = (66.7\ ksi)t_{des}\left(3.75t_{des} + 0.74\ in.\right)$ METRIC $\dfrac{R_n}{\Omega} = (460\ \text{MPa})t_{des}\left(3.75t_{des} + 18.8\ \text{mm}\right)$

The Demand Capacity Ratio for wall local yielding is shown below:

LRFD	ASD
$DCR = \dfrac{P_{rf}}{\phi R_n}$	$DCR = \dfrac{P_{rf}}{\left(R_n/\Omega\right)}$

where

$$P_{rf} = \left(\frac{M_{ux}}{d - t_f}\right) + \frac{P_{ub}}{2}$$

with M_{ux} and the P_{ub} are the required beam moment (X-axis) and the required beam axial demand, respectively. The term d denotes the depth of the beam and the term t_f is the thickness of the beam flange.

2.3.27 Local Crippling of HSS Sidewalls

In accordance with Section J10.3 of the *Specification*, the limit state of Web Local Yielding (i.e., Local Crippling of HSS Sidewalls) is determined as follows:

$$R_n = 0.80 t_w^2 \left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$$

(J10-4)

$\phi = 0.75$ (LRFD) $\qquad \Omega = 2.00$ (ASD)

where

E = modulus of elasticity of steel = 29,000 *ksi* (200,000 MPa)

F_{yw} = specified minimum yield stress of HSS main member material, *ksi* (MPa)

l_b = bearing length perpendicular to the width of the Inner Collar

t_w = design web thickness of HSS or Box column

= t_{des}

t_f = design flange thickness of HSS or Box column

= t_{des}

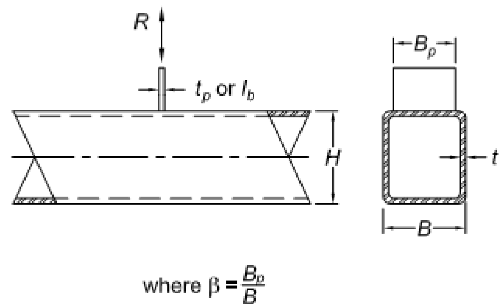
d = depth of web, *in*. (mm)

= $H - 3t_{des}$

Q_f = chord-stress interaction parameter

Connection Type

Transverse Plate T- and Cross-Connections, Under Plate Axial Load



Note: since HSS members have two webs, the available strength resulting from Eq. J10-4 will be multiplied by 2.

Note: Eq. J10-4 only applies when the concentrated compressive force is applied at a distance from the member end that is greater than or equal to $d/2$. If the concentrated compressive force is applied at a distance from the member end that is less than $d/2$ (e.g., if the collar is near the top of the column), then Eq. J10-5a or J10-5b should be used instead as demonstrated on the following pages.

From *Specification* Table K3.2, Q_f is determined as follows:

$Q_f = 1$ for connecting surface in tension,

$Q_f = 1.3 - 0.4\dfrac{U}{\beta}$ for connecting surface in compression.

Per *Specification* Section K3.1, the width ratio, β, is defined as the ratio of overall branch width (i.e., Inner Collar width) to chord width (i.e., column width) $= B_b / B$ for rectangular HSS. Since the Inner Collar width is less than the column width, the width ratio is calculated as follows:

$$\beta = \frac{B_b}{B}$$
$$= \frac{b_{IC}}{B} = \frac{6.0\,in.}{8.0\,in.}$$
$$= 0.75$$

The utilization ratio, *U*, is determined as follows using Eq. K2-4:

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right|$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS.

$P_{ro} = P_u$ for LRFD; P_a for ASD.
$M_{ro} = M_u$ for LRFD; M_a for ASD.

Using the bearing length calculated on page 177, the available strength based on the limit state of local crippling is calculated as follows:

$$F_y = 50\,ksi \quad (345\ \mathrm{MPa})$$
$$H = 8.0\,in. \quad (203.2\ \mathrm{mm})$$
$$l_b = 0.74\,in.$$

$$Q_f = 1.3 - 0.4\left(\frac{U}{0.75}\right) \le 1.0$$

$$\begin{aligned}
R_n &= 0.80 t_w^{\ 2}\left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f \times 2\,walls \\
&= 1.6 t_{des}^{\ 2}\left(1 + 3\left(\frac{l_b}{H - 3t_{des}}\right)\left(\frac{t_{des}}{t_{des}}\right)^{1.5}\right)\sqrt{\frac{EF_y t_{des}}{t_{des}}}Q_f \\
&= 1.6 t_{des}^{\ 2}\left(1 + \frac{3l_b}{H - 3t_{des}}\right)\sqrt{EF_y}Q_f \\
&= 1.6 t_{des}^{\ 2}\left(\frac{H - 3t_{des} + 3l_b}{H - 3t_{des}}\right)\sqrt{EF_y}Q_f \\
&= 1.6 t_{des}^{\ 2}\left(\frac{8.0\,in. - 3t_{des} + 3(0.74\,in.)}{8.0\,in. - 3t_{des}}\right)\sqrt{(29000\,ksi)(50\,ksi)}Q_f
\end{aligned}$$

$$R_n = (1{,}926\,ksi)\,t_{des}^{\ 2}\left(\frac{10.22\,in. - 3t_{des}}{8.0\,in. - 3t_{des}}\right)Q_f$$

The design local wall crippling strength, ϕR_n, and the allowable local wall crippling strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(1{,}926\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f$ ENGLISH $\phi R_n = (1{,}445\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f$ METRIC $\phi R_n = (9{,}962\ \text{MPa})t_{des}^{\ 2}\left(\frac{259.6\ \text{mm} - 3t_{des}}{203.2\ \text{mm} - 3t_{des}}\right)Q_f$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{(1{,}926\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f}{2.00}$ ENGLISH $\frac{R_n}{\Omega} = (963\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f$ METRIC $\frac{R_n}{\Omega} = (6{,}639\ \text{MPa})t_{des}^{\ 2}\left(\frac{259.6\ \text{mm} - 3t_{des}}{203.2\ \text{mm} - 3t_{des}}\right)Q_f$

If the concentrated compressive force is applied at a distance from the member end that is less than $d/2$, then either Eq. J10-5a or J10-5b shall be used instead depending on the quantity l_b/d.

$$\frac{l_b}{d} = \frac{l_b}{H - 3t_{des}} = \frac{0.74\,in.}{8.0\,in. - 3t_{des}}$$

The quantity l_b/d will always be less than 0.2 regardless of t_{des}; therefore, Eq. J10-5a applies and the local wall crippling strength shall be determined as follows.

$$R_n = 0.40t_w^{\ 2}\left(1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f \times 2\,walls$$
$$= \frac{1}{2}R_{n,EQ.J10\text{-}4}$$

$$R_n = 0.50 \times (1{,}926\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f$$
$$= (963\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f$$

The design local wall crippling strength, ϕR_n, and the allowable local wall crippling strength, R_n/Ω, are as follows:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(963\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f$ ENGLISH $\phi R_n = (722\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f$ METRIC $\phi R_n = (4{,}980\ \text{MPa})t_{des}^{\ 2}\left(\frac{259.6\ \text{mm} - 3t_{des}}{203.2\ \text{mm} - 3t_{des}}\right)Q_f$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{(963\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f}{2.00}$ ENGLISH $\frac{R_n}{\Omega} = (481\ ksi)t_{des}^{\ 2}\left(\frac{10.22\ in. - 3t_{des}}{8.0\ in. - 3t_{des}}\right)Q_f$ METRIC $\frac{R_n}{\Omega} = (3{,}320\ \text{MPa})t_{des}^{\ 2}\left(\frac{259.6\ \text{mm} - 3t_{des}}{203.2\ \text{mm} - 3t_{des}}\right)Q_f$

The Demand Capacity Ratio for wall local crippling is shown below:

LRFD	ASD
$DCR = \frac{P_{rf}}{\phi R_n}$	$DCR = \frac{P_{rf}}{\left(R_n/\Omega\right)}$

3 Gravity Connections

3.1 Installation Sequence

The ConXtech™ structural steel system uses “drop-in” style connections that reduce erection time while meeting safety regulations. The installation sequence for the beam-to-gravity-column and beam-to-beam connections is illustrated in the following figures.

Figure 92 shows the incoming beam that is lowered onto a shop-installed dowel (lowermost bolt). The incoming beam is fabricated with a saddle at the bottom cope to bear on the dowel. The beam-to-column connection is shown on the left.

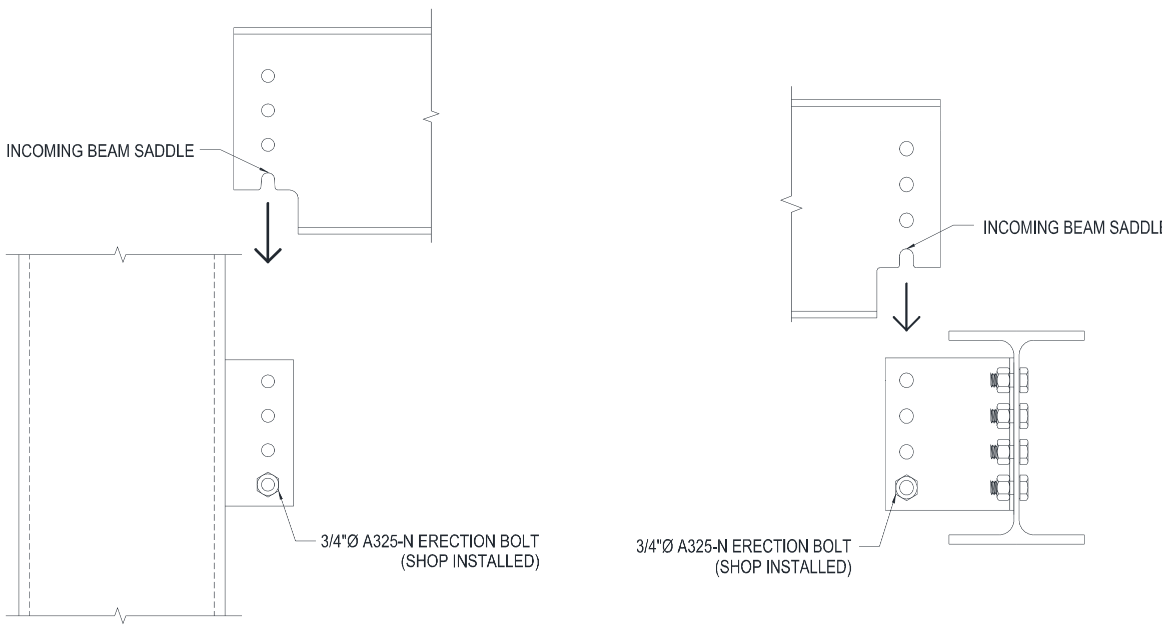


Figure 92: Gravity Beam Lowered onto Dowel.

Figure 93 shows the incoming beam web bearing on the dowel. The connection is stable for construction until the remaining bolts are installed.

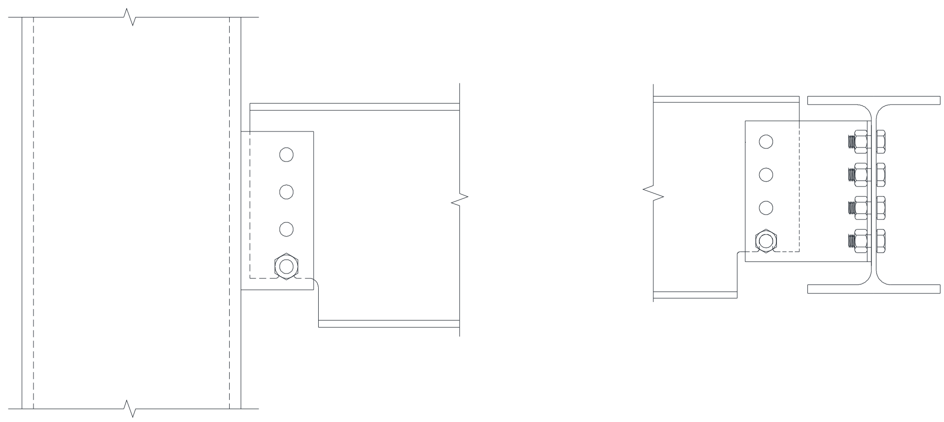


Figure 93: Gravity Beam Temporarily Bearing on Dowel.

Figure 94 shows the fully assembled double shear tab beam-to-column connection on the left.

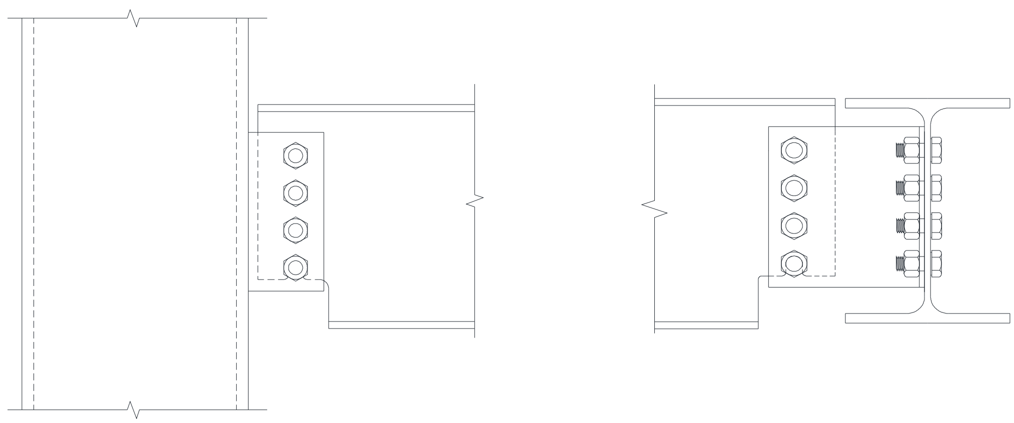


Figure 94: Gravity Connection with All Bolts Installed.

3.2 Design Overview

All the gravity connections are designed in accordance with the AISC *Steel Construction Manual* and the AISC 360-16 *Specification for Structural Steel Buildings*. The design strength, ϕR_n, and the allowable strength, R_n/Ω, of the connecting elements, connectors and the affected elements of connected members have been determined in accordance with the provisions of Chapter B and Chapter J of the *Specification*. For each type of connection, the design strength, ϕR_n, and the allowable strength, R_n/Ω, is taken as the lowest governing capacity from all the applicable limit states (weld strength, bolt shear strength, bearing at the bolt holes, block shear, shear yielding, shear rupture, tensile yielding, tensile rupture, HSS punching shear, and HSS plastification). Unless stated otherwise, all variables used in the equations herein are as defined in the *Specification*. In the welded/bolted double shear tab beam-to-column connections, the distance from the centerline of the bolts to the face of the column is less than 3 in. Therefore, the effects of eccentricity are ignored (except for limit states for the HSS walls), in conformance with the provisions of the *Manual*.

The calculations to determine the capacity of each connection treat the dowel in the beam cope saddle as a fastener in a long-slotted hole perpendicular to the direction of the load without vertical uplift capacity (see Figure 95). Therefore, the dowel behaves as a “roller” supported fastener for pure shear and pure drag. For combined shear and drag loading, however, the dowel is sometimes considered effective in resisting horizontal loads depending on the angle of the resultant (refer to Section 3.3.3, Combined Shear and Drag Capacity).

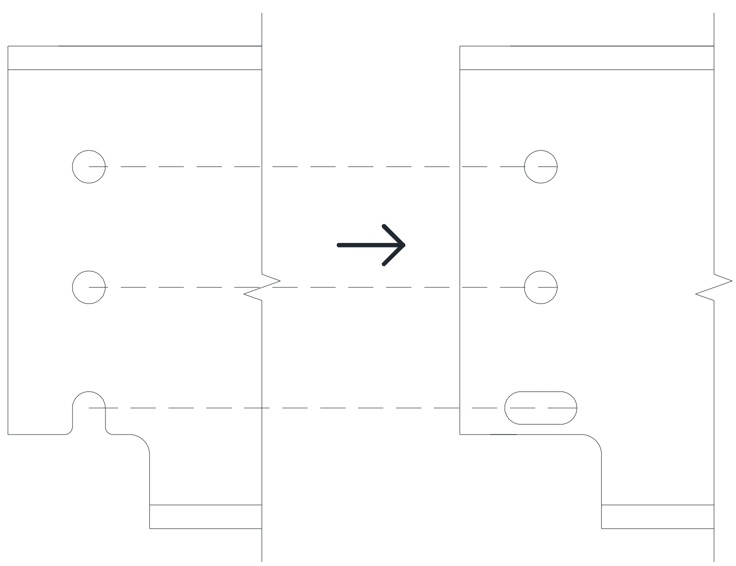


FIGURE 95: STRUCTURAL ANALOGY OF THE SADDLE.

3.3 Beam to Column Gravity Connection

The welded/bolted double shear tab connection to a column consists of two shear tabs that are welded onto the column and bolted to the incoming beam (not shown for clarity), as shown in Figure 96, Figure 97, and Figure 98.

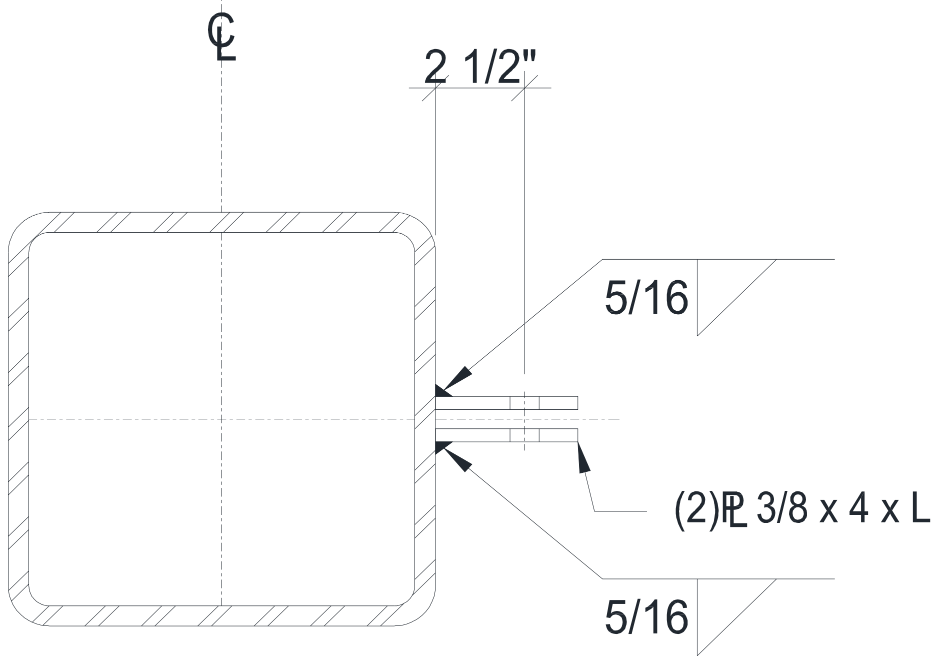


Figure 96: Plan View of a Welded/Bolted Double Shear Tab Connection.

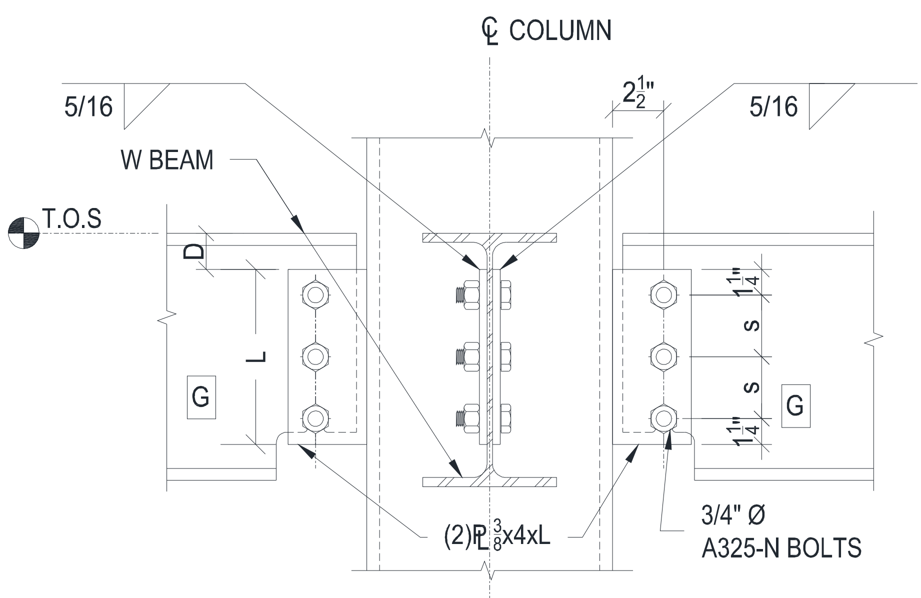


Figure 97: Elevation View of a Welded/Bolted Double Shear Tab Connection.

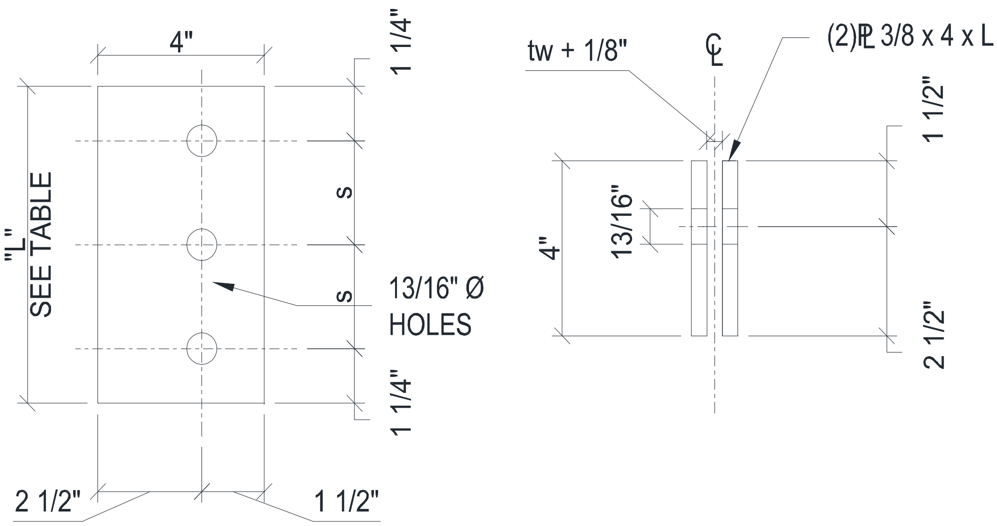


Figure 98: Elevation View and Plan View of Shear Tabs.

Throughout the document, all calculations are based on a 3-bolt configuration with two PL ⅜ in. x 4 in. x 7⅛ in. and a bolt spacing of 2$\frac{5}{16}$ in.; other bolt configurations follow similar formulation.

3.3.1 Shear Capacity

The following calculations determine the capacity of welded/bolted double shear tab connections in beam-to-column gravity connections. The available strength of welded/bolted double shear tab connections is determined from the limit states below:

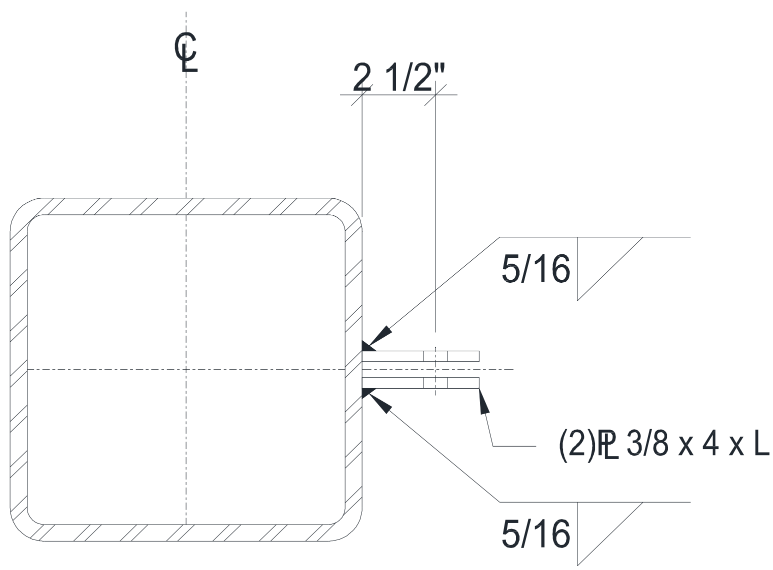
- Welds
- Bolt Shear
- Bearing at Bolt Holes
  - In the Shear tab
  - In the Beam Web
- Block Shear
  - In the Shear tab
- Shear Yielding
  - In the Shear tab
  - In the Beam Web
- Shear Rupture
  - In the Shear tab
  - In the Beam Web
- HSS Punching Shear

A table of the available shear strength of welded/bolted double shear tab connections is provided at the end of this section. The available strength of the beam-to-column gravity shear connection in the table is determined per the following:

1. Beam web available strength per thickness of web.
2. HSS wall available strength per thickness of wall.
3. Bolt, Weld and Plate available strength for ⅜ in. thick plates.

3.3.1.1 Minimum Base Metal Thickness for Weld

In accordance with *Manual* Part 9-5, the base metal should have adequate thickness to develop the full strength of the weld.



For fillet welds with F_{EXX} = 70 ksi on one side of the connection, the minimum base metal thickness required to match the shear rupture strength of the connecting element to the shear rupture strength of the base metal is as shown on the following page.

For a fillet weld on <u>one side</u> of the connection:

$$t_{\min} = \frac{0.60F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u}$$

$$= \frac{3.09\left(\frac{5/16\,in.}{1/16\,in.}\right)}{(62\,ksi)} = 0.25\,in.$$

Where

D = number of sixteenths of an inch of weld size
F_u = specified minimum tensile strength of the connecting element, ksi

Therefore, an HSS column with a design wall thickness, t_{des}, of ¼ in. or greater is adequate. The HSS column to be used in the example has a nominal thickness of ½ in. The design wall thickness of an HSS for A500 grade steel is $t_{des} = 0.93t_{nom}$.

3.3.1.2 Weld Strength

In accordance with *Specification* Section J2.4, the available strength of welded joints is determined as follows:

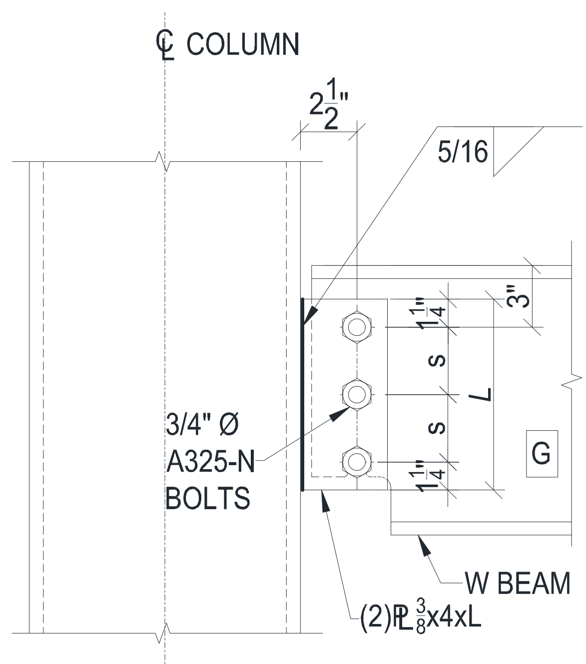
$$R_n = F_{nw}A_{we} \qquad \text{(J2-3)}$$

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$

For fillet welds,

$$F_{nw} = 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right) \qquad \text{(J2-5)}$$

The length of weld is equal to the length of the shear tab. Using n to denote the number of bolts, the available strength is calculated as shown on the following page.



The length of the weld is:

$$n = \text{Number of Bolts} = 3$$
$$L = 2\left(1\frac{1}{4}\,in.\right) + (n-1)2.3125\,in. = 2.50in. + (3-1)2.3125\,in. = 7.125\,in.$$

The size of the weld is:

$$t_{we} = \left(\frac{\sqrt{2}}{2}\cdot\frac{5}{16}\,in.\right) = 0.22\,in.$$

The area of the weld (per shear tab) is:

$$A_{we} = t_{we}L = (0.22\,in.)(7.125\,in.) = 1.56\,in.^2\,/\,Plate$$

The strength of the weld is:

$$\theta = 0^\circ$$
$$R_n = 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right)A_{we}\ x\ 2\ Plates$$
$$= 0.60(70\ ksi)\left(1.0 + 0.50\sin^{1.5}(0^\circ)\right)\left(1.56\,in.^2\right)\ x\ 2\ Plates$$
$$= 131.04\ kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(131.04\ kips)$ $= 98.28\ kips\ \ (437.15\ \text{kN})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{131.04\ kips}{2.00}$ $= 65.52\ kips\ \ (291.43\ \text{kN})$

3.3.1.3 Bolt Shear Strength

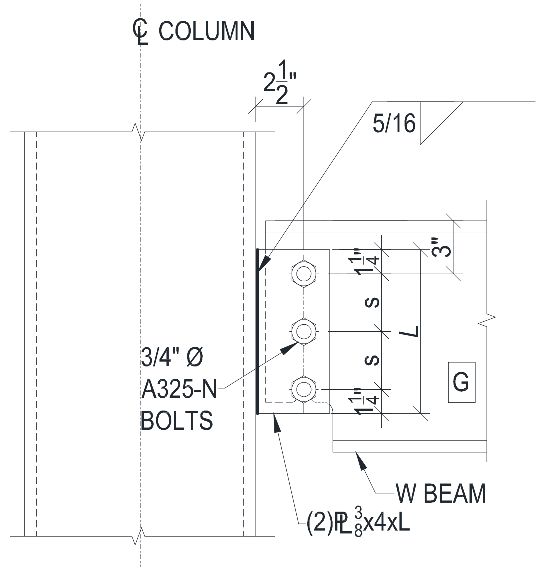
The design shear strength, ϕR_n, and the allowable shear strength, R_n/Ω, of snug-tightened or pretension high-strength bolt or threaded part shall be determined according to the limit states of shear rupture as follows:

$$R_n = F_{nv} A_b \quad \text{(J3-1)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

Per *Specification* Table J3.2, the nominal shear strength of A325 bolts when threads are not excluded from shear planes is

$$F_{nv} = 54 \ ksi$$



Using n to denote the number of bolts, the available shear strength of ¾ in. diameter ASTM A325-N bolts in double-shear is:

$$\text{Number of Fasteners, } n = 3$$

$$A_b = \frac{\pi d_b^{\,2}}{4} = \frac{\pi}{4}\left(\frac{3}{4}\ in.\right)^2 = 0.442\ in.^2$$

$$R_n = 3\left(54\,ksi\right)\left(0.442\ in.^2\right)x2 = 143.21\ kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(143.21\,kips\right)$ $= 107.41\,kips \ \left(477.76 \text{ kN}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{143.21\,kips}{2.00}$ $= 71.61\,kips \ \left(318.52 \text{ kN}\right)$

3.3.1.4 Bolt Bearing in the Plates

The available bearing strength at bolt holes shall be determined for the limit state of bearing as follows:

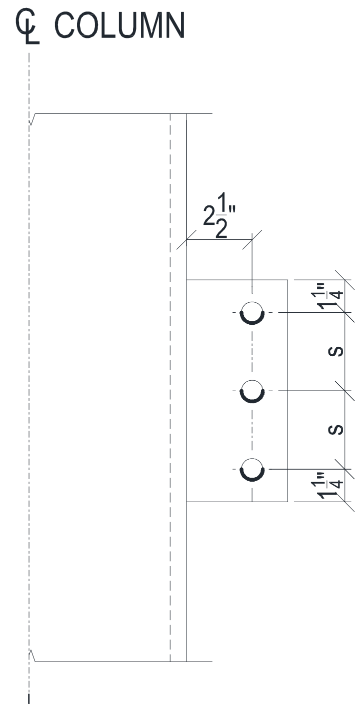
$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force, when deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5 l_c t F_u \leq 3.0 d t F_u \qquad \text{(J3-6b)}$$

The ASTM A36 plates have the following material specifications:

$$F_y = 36\ ksi$$
$$F_u = 58\ ksi$$



NOTE: The incoming beam delivers a downward force on the shear tabs.

Edge Bolt:

$$d_{hole} = d_b + \frac{1}{16}\,in. = \left(\frac{3}{4} + \frac{1}{16}\right) in. = \frac{13}{16}\,in.$$

$$l_c = 1\frac{1}{4}\,in. - \frac{1}{2}\left(\frac{13}{16}\,in.\right) = 0.84\,in.$$

$$R_n = (1)1.5(0.84\,in.)\left(\frac{3}{8}\,in.\right)58\,ksi = 27.41\,kips$$

$$\leq (1)3.0\left(\frac{3}{4}\,in.\right)\left(\frac{3}{8}\,in.\right)58\,ksi = 48.94\,kips \qquad \therefore R_n = 27.41\,kips\,/\,Plate$$

Interior Bolts:

$$d_{hole} = \frac{13}{16}\,in.$$

$$l_c = 2.3125\,in. - \left(\frac{13}{16}\,in.\right) = 1.50\,in.$$

$$R_n = (n-1)1.5(1.50\,in.)\left(\frac{3}{8}\,in.\right)58\,ksi$$

$$= (3-1)1.5(1.50\,in.)\left(\frac{3}{8}\,in.\right)58\,ksi = 97.88\,kips$$

$$\leq (3-1)3.0\left(\frac{3}{4}\,in.\right)\left(\frac{3}{8}\,in.\right)58\,ksi = 97.88\,kips \qquad R_n = 97.88\,kips/Plate$$

The available bearing strength (for both plates) at the bolt holes considering the edge and the interior bolts is as follows:

$$R_n = (27.41\,kips/Plate + 97.88\,kips/Plate)\,x2\,Plates = 250.58\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(250.58\,kips)$ $= 187.94\,kips \ (835.96\text{ kN})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{250.58\,kips}{2.00}$ $= 125.29\,kips \ (557.29\text{ kN})$

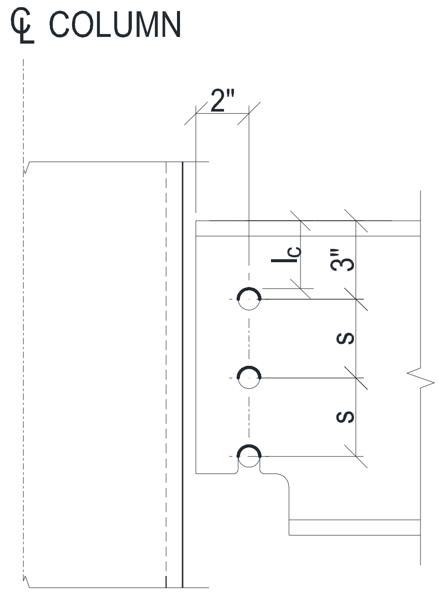
3.3.1.5 Bolt Bearing in the Beam

The available bearing strength at bolt holes shall be determined for the limit state of bearing as follows:

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force when deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5 l_c t F_u \le 3.0 d t F_u \qquad \text{(J3-6b)}$$



NOTE: The beam reaction is an upward force on the beam. The distance from the edge of the shear tab to the T.O.S, "D", is 1¾ in. as shown on the plans. Note that D is constant regardless of beam depth.

The ASTM A992 wide flange beam has the following material specifications:

$$F_y = 50\ ksi$$
$$F_u = 65\ ksi$$

The bearing strength on the bolt holes in the beam is calculated per inch of beam web thickness.

$$\therefore R_n = 1.5 l_c F_u \le 3.0 d F_u$$

Edge Bolt:

$$d_{hole} = d_b + \frac{1}{16}\,in. = \left(\frac{3}{4} + \frac{1}{16}\right) in. = \frac{13}{16}\,in.$$

$$l_c = 1.75\,in. + 1.25 in - \frac{1}{2}\left(\frac{13}{16}\,in.\right) = 2.59\,in.$$

$$R_n = (1)1.5(2.59\,in.)65\,ksi = 252.52\,kips/in.$$

$$\le (1)3.0\left(\frac{3}{4}\,in.\right)65\,ksi = 146.25\,kips/in. \qquad \therefore R_n = 146.25\,kips/in.$$

Interior Bolts:

$$d_{hole} = d_b + \frac{1}{16}\,in. = \left(\frac{3}{4} + \frac{1}{16}\right)in. = \frac{13}{16}\,in.$$

$$\begin{aligned}
l_c &= 2.3125\,in. - d_h \\
&= 2.3125\,in. - \frac{13}{16}\,in. = 1.50\,in. \\
R_n &= (n-1)1.5 l_c F_u \\
&= (3-1)1.5(1.50\,in.)65\,ksi = 292.50\,kips/in. \\
&\le (3-1)3.0\left(\frac{3}{4}\,in.\right)65\,ksi = 292.50\,kips/in. \qquad \therefore R_n = 292.50\,kips/in.
\end{aligned}$$

The available bearing strength at the bolt holes considering the edge bolt and the interior bolts is as follows:

$$R_n = 146.25\,kips/in. + 292.50\,kips/in. = 438.75\,kips/in.$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(438.75\,kips/in.)$ $= 329.06\,\frac{kips}{in.}\left(57.62\,\frac{\text{kN}}{\text{mm}}\right)$	$\frac{R_n}{\Omega} = \frac{438.75\,kips/in.}{2.00}$ $= 219.38\,\frac{kips}{in.}\left(38.42\,\frac{\text{kN}}{\text{mm}}\right)$

3.3.1.6 Block Shear

The available strength for the limit state of block shear rupture along a shear failure path or paths and a perpendicular tension failure path shall be taken as

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(J4-5)}$$

$$\phi = 0.75\ \text{(LRFD)} \qquad \Omega = 2.00\ \text{(ASD)}$$

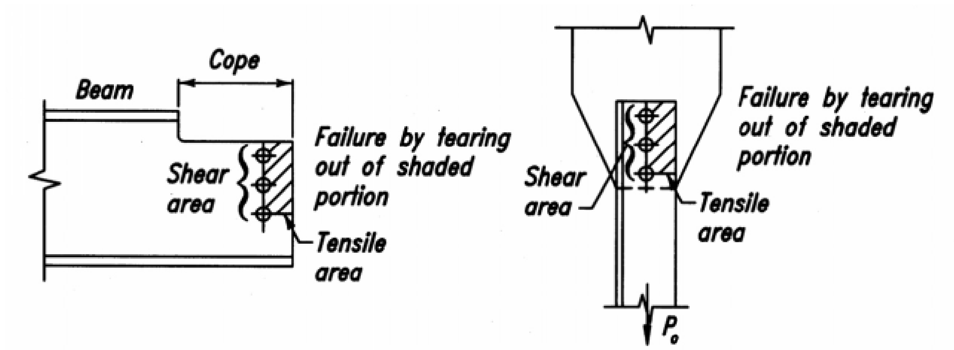


FIGURE 99: FAILURE SURFACES FOR BLOCK SHEAR RUPTURE LIMIT STATE.

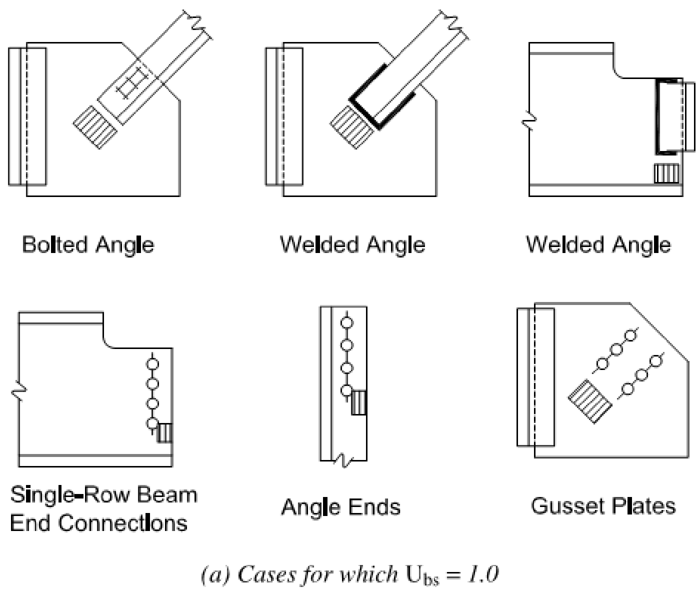


FIGURE 100: BLOCK SHEAR TENSILE STRESS DISTRIBUTIONS (AISC 360-16 COMMENTARY J4).

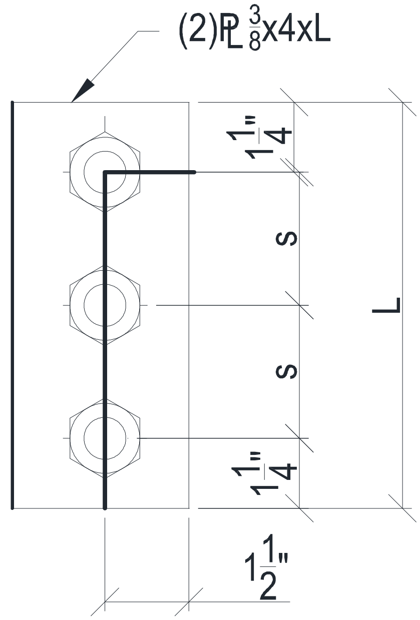
The double shear tab connection to a column can be categorized as a case where the tensile stress for block shear is uniform. Therefore, the reduction factor $U_{bs} = 1.0$.

The following calculations determine the block shear strength for the double shear tab and the beam.

Block Shear Strength in the Plates

The ASTM A36 plate material has the following material specifications:

$$F_y = 36\ ksi$$
$$F_u = 58\ ksi$$



The net shear, net tension and gross shear areas (per plate) are calculated as follows:

$$U_{bs} = 1.0$$

$$A_{nv} = \left(L - 1\frac{1}{4}\,in. - (n-0.5)\left(d_b + \frac{1}{16}\,in. + \frac{1}{16}\,in.\right)\right)t_p$$
$$= \left(7.125\,in. - 1\frac{1}{4}\,in. - (3-0.5)\left(\frac{3}{4}\,in. + \frac{1}{8}\,in.\right)\right)\left(\frac{3}{8}\,in.\right) = 1.38\,in.^2$$

$$A_{nt} = \left(1\frac{1}{2}\,in. - \frac{1}{2}\left(d_b + \frac{1}{8}\,in.\right)\right)t_p$$
$$= \left(1\frac{1}{2}\,in. - \frac{1}{2}\left(\frac{3}{4}\,in. + \frac{1}{8}\,in.\right)\right)\left(\frac{3}{8}\,in.\right) = 0.40\,in.^2$$

$$A_{gv} = \left(L - 1\frac{1}{4}\,in.\right)t_p = \left(7.125\,in. - 1\frac{1}{4}\,in.\right)\left(\frac{3}{8}\,in.\right) = 2.20\,in.^2$$

The block shear strength (per plate) is calculated as follows:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt}$$
$$= 0.60(58\ ksi)1.38\ in.^2 + (1.0)(58\ ksi)0.40\ in.^2 = 71.22\ kips/Plate$$

$$\leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$
$$= 0.60(36\ ksi)2.20\ in.^2 + (1.0)(58\ ksi)0.40\ in.^2 = 70.72\ kips/Plate$$

$$R_n = 70.72\ kips/Plate$$

The block shear strength (for both plates) is calculated as follows:

$$R_n = (70.72\ kips/Plate)x2\ Plates$$
$$= 141.44\ kips$$

The available design strength or allowable strength is therefore:

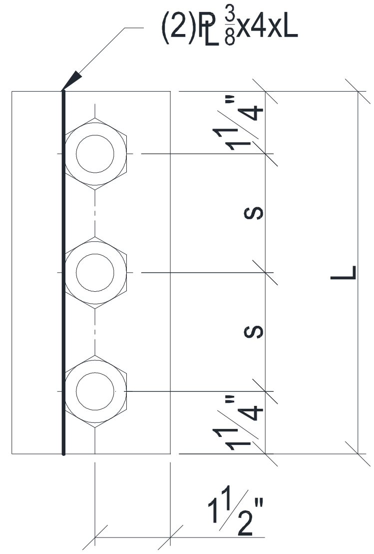
LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(141.44\ kips)$ $= 106.08\ kips\ \ (471.84\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{141.44\ kips}{2.00}$ $= 70.72\ kips\ \ (314.56\ \text{kN})$

Block Shear Strength in the Beam

The limit state of block shear is not applicable to the beam because of the presence of the saddle on the beam cope. The failure mechanisms in the beam are shear yielding in the gross section and shear rupture in the net section; refer to Section 3.3.1.8.

3.3.1.7 Shear Yielding and Shear Rupture in the Plates

In accordance with *Specification* Section J4.2, the available shear strength of the affected and connecting elements shall be the lower value obtained according to the limit states of shear yielding and shear rupture:



e. For shear yielding of the element:

$$R_n = 0.60 F_y A_{gv} \qquad \text{(J4-3)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

f. For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \qquad \text{(J4-4)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

The thickened vertical line in the figure on the right shows the shear plane of the gross section.

For shear yielding in the gross section:

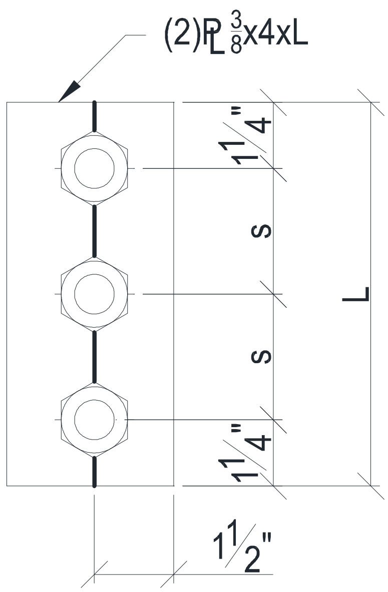
$$\begin{aligned} A_{gv} &= L \cdot t_p \\ &= 7.125\, in.\left(\frac{3}{8}\, in.\right) \\ &= 2.67\, in.^2 / Plate \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_y A_{gv} x2\ Plates \\ &= 0.60(36\, ksi)(2.67\, in.^2 / Plate) x2\ Plates \\ &= 115.34\, kips \end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(115.34\,kips)$ $= 115.34\,kips \;\; (513.03\,\text{kN})$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{115.34\,kips}{1.50}$ $= 76.89\,kips \;\; (342.00\,\text{kN})$

The thickened vertical lines in the figure on the right show the shear plane of the net section.



For shear rupture in the net section:

$$
\begin{aligned}
A_{nv} &= \left(L - n\left(d_b + \frac{1}{8}\,in. \right) \right) t_p \\
&= \left(7.125\,in. - 3\left(\frac{3}{4}\,in. + \frac{1}{8}\,in. \right) \right)\left(\frac{3}{8}\,in. \right) \\
&= 1.68\,in.^2 / Plate
\end{aligned}
$$

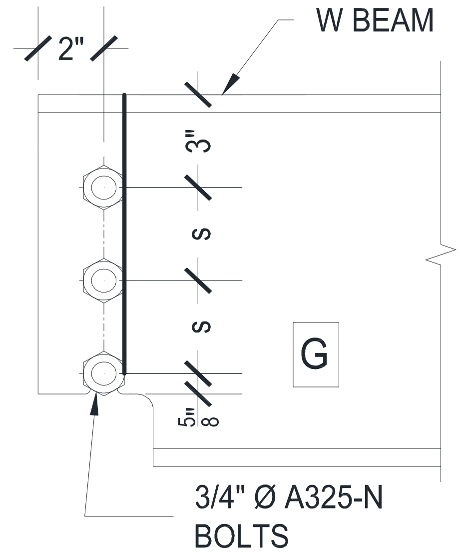
$$
\begin{aligned}
R_n &= 0.60 F_u A_{nv} \\
&= 0.60(58\,ksi)(1.68\,in.^2 / Plate)x2\ Plates \\
&= 116.92\,kips
\end{aligned}
$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(116.92\,kips)$ $= 87.69\,kips\ (390.04\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{116.92\,kips}{2.00}$ $= 58.46\,kips\ (260.03\ \text{kN})$

Note: Based on the preceding calculations, shear rupture is the governing limit state of the two.

3.3.1.8 Shear Yielding and Shear Rupture in the Beam



In accordance with *Specification* Section J4.2, the available shear strength of the affected and connecting elements shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

a. For shear yielding of the element:

$$R_n = 0.60 F_y A_{gv} \qquad \text{(J4-3)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

b. For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \qquad \text{(J4-4)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

In the figure above, the thickened vertical line represents the shear plane of the gross section.

NOTE: The shear yielding strength of the beam is calculated <u>per inch</u> of beam web thickness.

For shear yielding in the gross section:

$$A_{gv} = \left((n-1)2.3125 in. + 1.75 in. + 1.25 in.\right)\left(\frac{t_w}{t_w}\right) = \left((3-1)2.3125 in. + 3.00 in.\right)(1.0)$$

$$= 7.62 \frac{in.^2}{in.}$$

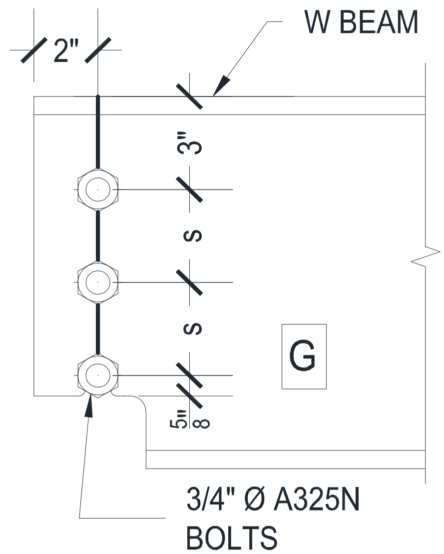
$$R_n = 0.60 F_y A_{gv} = 0.60(50\ ksi)\left(7.62 \frac{in.^2}{in.}\right)$$

$$= 228.60 \frac{kips}{in.}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00\left(228.60\,\frac{kips}{in.}\right)$ $= 228.60\,\frac{kips}{in.}\,\left(40.03\,\frac{\text{kN}}{\text{mm}}\right)$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{228.60\,kips/in.}{1.50}$ $= 152.40\,\frac{kips}{in.}\,\left(26.68\,\frac{\text{kN}}{\text{mm}}\right)$

In the figure on the right, the thickened vertical lines represent the shear plane of the net section.



For shear rupture in the net section:

NOTE: The shear rupture strength of the beam is calculated <u>per inch</u> of beam web thickness.

$$A_{nv} = \left(7.625in. - (n-0.5)\left(d_b + \frac{1}{8}in.\right)\right)\left(\frac{t_w}{t_w}\right)$$
$$= \left(7.625in. - (3-0.5)\left(\frac{3}{4}in. + \frac{1}{8}in.\right)\right)(1.0)$$
$$= 5.43\frac{in.^2}{in.}$$

$$R_n = 0.60 F_u A_{nv}$$
$$= 0.60(65\ ksi)\left(5.43\frac{in.^2}{in.}\right)$$
$$= 211.77\frac{kips}{in.}$$

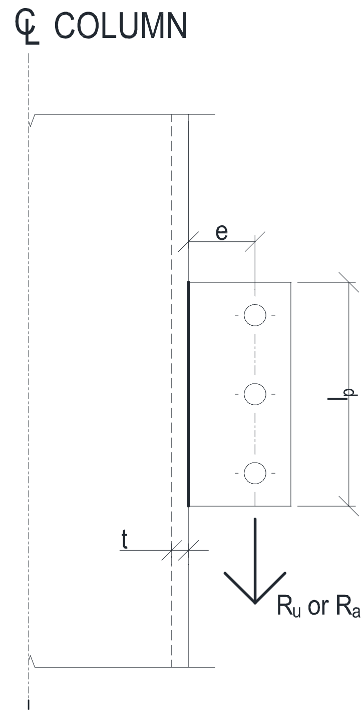
The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(211.77\frac{kips}{in.}\right)$ $= 158.82\frac{kips}{in.}\left(27.81\frac{\text{kN}}{\text{mm}}\right)$	$\frac{R_n}{\Omega} = \frac{211.77\frac{kips}{in.}}{2.00}$ $= 105.88\frac{kips}{in.}\left(18.54\frac{\text{kN}}{\text{mm}}\right)$

Note: Based on the preceding calculations, shear rupture is the governing limit state of the two.

3.3.1.9 HSS Punching Shear

In accordance with *Manual* Part 10, the limit state for punching shear in the HSS wall shall be determined as follows:



$$R_u e \leq \frac{\phi F_u t l_p^{\,2}}{5} \qquad (10\text{-}7a)$$

$$R_a e \leq \frac{F_u t l_p^{\,2}}{5\Omega} \qquad (10\text{-}7b)$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

Where

l_p = Length of plate.
F_u = Ultimate stress of HSS.
t = Design thickness of HSS wall.
e = Eccentricity of bolt group to HSS wall.
R_u = Required shear strength, LRFD.
R_a = Required shear strength, ASD.

Equation 10-7 is rearranged to treat the required shear strength as an available shear strength.

$$R_n = \frac{F_u t l_p^{\,2}}{5e}$$

The ASTM A500 Gr. C HSS material has the following material specifications:

$$F_y = 50\ ksi$$
$$F_u = 62\ ksi$$

The punching shear strength of the beam is calculated per inch of HSS wall thickness.

NOTE: to use the double shear tab connection, the HSS column wall must be nonslender for axial compression. Per *Specification* Table B4.1a, the limiting width-to-thickness ratio, λ_r, for nonslender HSS walls in compression is calculated as shown below (A500 Gr. C).

$$\lambda_r = 1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29000\,ksi}{50\,ksi}} = 33.7$$

The column to be used in the example is an HSS12x12x½, which has a width-to-thickness ratio, b/t, of 22.8; thus, the column wall is nonslender for axial compression and the double shear tab connection may be used.

The punching shear strength is calculated as follows:

$$e = 2.50 in.$$

$$R_n = \frac{F_u l_p^{\,2}}{5e}$$

$$= \frac{(62ksi)(7.125in.)^2}{5(2.50in.)} = 251.79\frac{kip}{in.}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(251.79\frac{kips}{in.}\right)$ $= 188.84\frac{kips}{in.}\left(33.07\frac{\text{kN}}{\text{mm}}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{251.79\frac{kips}{in.}}{2.00}$ $= 125.89\frac{kips}{in.}\left(22.04\frac{\text{kN}}{\text{mm}}\right)$

3.3.1.10 Design Aid

Table 5, below, shows the available shear strength of bolted/welded double shear tab connections. The table is a design aid for all the bolted/welded double shear tab connections that consist of seven (7) bolts or fewer in a single row.

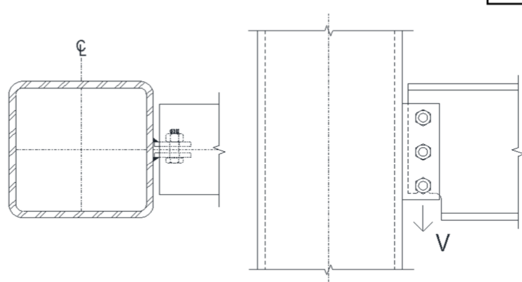
TABLE 5: AVAILABLE STRENGTH OF BOLTED/WELDED DOUBLE SHEAR TAB CONNECTIONS (ENGLISH).

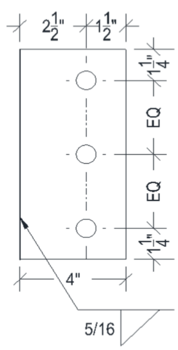
Available Shear Strength of Bolted/Welded Double Shear Tab Connections

$\frac{3}{4}$ – in. A325-N BOLTS

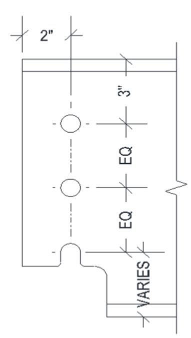
HSS	F_y =	50 ksi
	F_u =	62 ksi
Beam	F_y =	50 ksi
	F_u =	65 ksi
Plate	F_y =	36 ksi
	F_u =	58 ksi

W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





Number of Bolts (Beam Depth)	Length of Plate, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		HSS Wall Available Strength per Inch of Thickness, kips/in		Bolts, Welds, and Plate Available Strength, kips	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD
3 (W12)	7.1	106	159	125	188	58	88
4 (W14)	9.4	134	201	220	331	77	116
5 (W16)	11.8	162	243	342	513	96	144
6 (W18, 21)	14.1	190	285	490	735	115	172
7 (W24, 27, 30, 33, 36)	16.4	218	327	664	997	133	200



Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-16 Specification. All values tabulated above have been rounded down to the nearest whole number.
2. Values are based on one 5/16-in. fillet weld on each plate. The minimum HSS design wall thickness, t_{des}, to develop the full strength of the welds is 1/4-in.
3. The HSS wall must be nonslender for axial compression.
4. Limit states include weld strength, bolt shear, bearing, block shear, shear yielding, shear rupture, and HSS wall punching shear.
5. All values assume A325-N bearing type connections in STD holes. Bolt spacing is 2-5/16" for all beam sizes. Distance to top bolt from top of steel is assumed to be 3".
6. n = Number of bolts in connection.
7. L = Length of Plates, in. Values are based on a plate thickness of ⅜".

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3.3.2 Drag Capacity

When the double shear tab connection is subjected to an axial tensile (drag) load, the load travels from the beam, through the bolts, into the shear tabs and finally through the weld and into the HSS column.

The following calculations determine the capacity of double shear tab drag connections in beam-to-column connections. The available strength of double shear tab drag connections is determined from the limit states below:

- Weld Strength (Fillet Welds)
- Bolt Shear
- Bearing at Bolt Holes
    - In the Plate
    - In the Beam Web
- Block Shear
    - In the Plate
    - In the Beam Web
- Tensile Yielding
    - In the Plate
    - In the Beam Web
- Tensile Rupture
    - In the Plate
    - In the Beam Web
- HSS Plastification

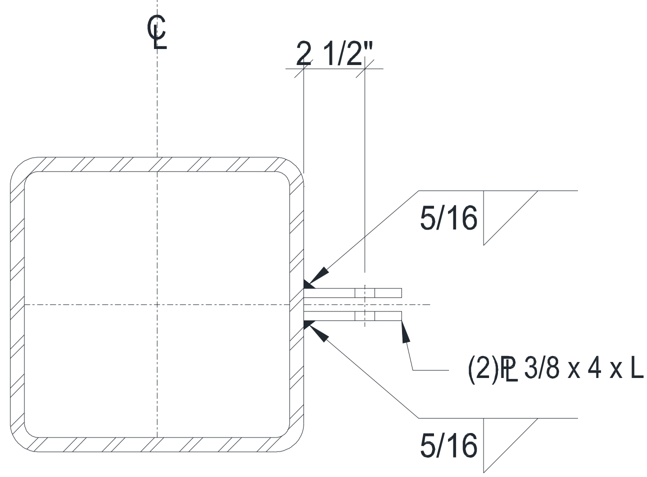
NOTE: For all the applicable limit states above, the contribution of the dowel is ignored in the determination of drag capacity (conservative). Bolt eccentricity to the centerline of the beam is minimal and thus is ignored.

A table of the available axial tensile strength of bolted/welded double-plate connections is provided at the end of this section. The available strength of bolted/welded double-plate connections in the table is provided for the following:

1. Beam web available strength per thickness of web.
2. HSS wall available strength per square of the wall thickness.
3. Bolt, Plate, and Weld available strength for ⅜ in. thick plates.

3.3.2.1 Minimum Base Metal Thickness for Weld

In accordance with *Manual* Part 9-5, the base metal should have adequate thickness to develop the full strength of a weld.



For fillet welds with F_{EXX} = 70 ksi on one side of the connection, the minimum base metal thickness required to match the shear rupture strength of the connecting element to the shear rupture strength of the base metal is as shown below. Though the *Manual* and the *Specification* permit an increase in weld strength due to the loading angle, the increased strength corresponds with a decrease in weld ductility. For the sake of conservatism, the increased weld strength will be ignored and the load angle will be set to zero.

For a fillet weld on <u>one side</u> of the connection:

$$\theta = 0°$$

$$t_{\min} = \frac{0.60F_{EXX}\left(1.0+0.50\sin^{1.5}\left(0°\right)\right)\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u}$$

$$= \frac{3.09\left(1.0\right)D}{F_u} = \frac{3.09\left(1.0\right)\left(\frac{5/16\,in.}{1/16\,in.}\right)}{\left(62\ ksi\right)} = 0.249\ in.$$

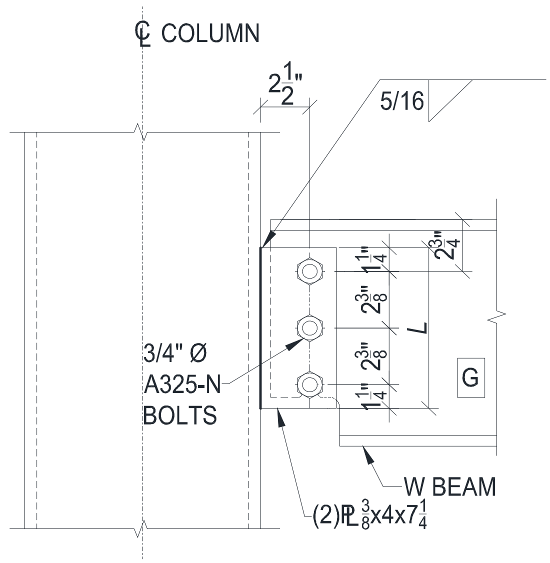
Where

D = number of sixteenths of an inch of weld size
F_u = specified minimum tensile strength of the connecting element, ksi.

Therefore, an HSS column with a design wall thickness, t_{des}, of ¼ in. or greater is adequate. The HSS column to be used in the example has a nominal thickness of ½ in. The design wall thickness of an HSS is $t_{des} = 0.93t_{nom}$.

3.3.2.2 Weld Strength

In accordance with *Specification* Section J2.4, the available strength of welded joints is determined as follows:



$$R_n = F_{nw} A_{we} \qquad \text{(J2-3)}$$

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$

For fillet welds,

$$F_{nw} = 0.60 F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right) \qquad \text{(J2-5)}$$

The length of the weld is equal to the length of the shear tab. Using n to denote the number of bolts, the available strength is calculated as shown below.

The length of the weld is:

$$n = \text{Number of Bolts} = 3$$

$$L = 2\left(1\frac{1}{4}\ in.\right) + (n-1)2.375\ in. = 2.50 in. + (3-1)2.3125\ in. = 7.125\ in.$$

The size of the weld is:

$$t_{we} = \left(\frac{\sqrt{2}}{2} \cdot \frac{5}{16}\ in.\right) = 0.22\ in.$$

The area of the weld (per shear tab) is:

$$A_{we} = t_{we} L = (0.22\ in.)(7.125\ in.) = 1.56\ in.^2 / Plate$$

The strength of the weld (for both shear tabs) is:

$$\theta = 0°$$

$$\begin{aligned} R_n &= 0.60F_{EXX}\left(1.0+0.50\sin^{1.5}(\theta)\right)A_{we}\ x\ 2\ Plates \\ &= 0.60(70\ ksi)\left(1.0+0.50\sin^{1.5}(0°)\right)\left(1.56\ in.^2\right)x\ 2\ Plates \\ &= 131.04\ kips \end{aligned}$$

The available design strength or allowable strength is:

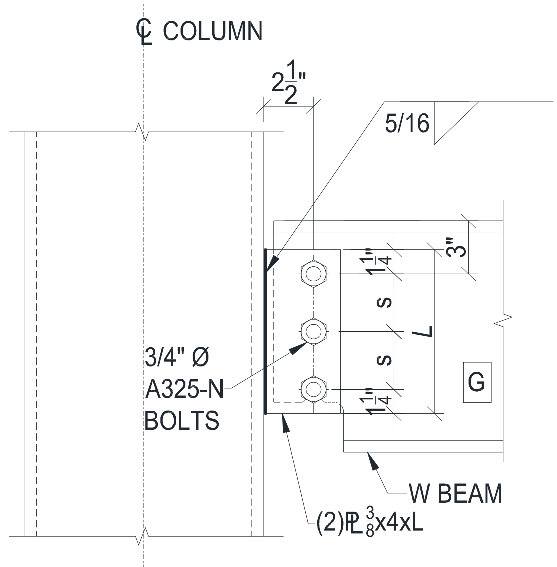
LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(131.04\ kips)$ $= 98.28\ kips\ \ (437.15\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{131.04\ kips}{2.00}$ $= 65.52\ kips\ \ (291.43\ \text{kN})$

3.3.2.3 Bolt Shear Strength

The design shear strength, ϕR_n, and the allowable shear strength, R_n/Ω, of snug-tightened or pretension high-strength bolt or threaded part shall be determined according to the limit states of shear rupture as follows:

$$R_n = F_{nv} A_b \quad \text{(J3-1)}$$

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$



Per *Specification* Table J3.2, the nominal shear strength of A325 bolts when threads are not excluded from shear planes is

$$F_{nv} = 54\,ksi$$

Using n to denote the number of bolts and neglecting the contribution of the dowel, the available shear strength of ¾ in. diameter ASTM A325-N bolts in double-shear is:

$$\text{Number of Fasteners,} = (n-1) = (3-1) = 2$$

$$A_b = \frac{\pi d_b^{\,2}}{4} = \frac{\pi}{4}\left(\frac{3}{4}\,in.\right)^2 = 0.442\ in.^2$$

$$R_n = 2(54\,ksi)(0.442\ in.^2)x2 = 95.47\ kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(95.47\ kips)$ $= 71.60\ kips\ \ (318.48\ \text{kN})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{95.47\ kips}{2.00}$ $= 47.74\ kips\ \ (212.35\ \text{kN})$

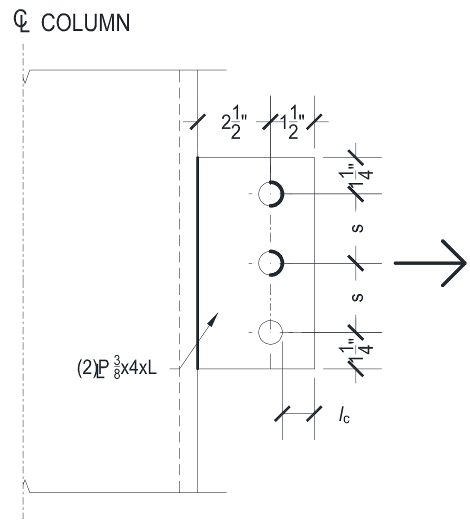
3.3.2.4 Bolt Bearing in the Plates

The available bearing strength at bolt holes shall be determined for the limit state of bearing as follows:

$$\phi = 0.75\ (\text{LRFD}) \qquad \Omega = 2.00\ (\text{ASD})$$

For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force when deformation at the bolt hole at service load is <u>not</u> a design consideration

$$R_n = 1.5 l_c t F_u \leq 3.0 d t F_u \qquad \text{(J3-6b)}$$



NOTE: The incoming beam delivers an axial tensile force pulling the shear tabs away from the column.

The ASTM A36 plates have the following material specifications:

$$F_y = 36\ ksi$$
$$F_u = 58\ ksi$$

The contribution of the dowel (lowermost bolt) to bearing is ignored due to the presence of the saddle on the beam cope.

Using n to denote the number of bolts, the available bearing strength is as calculated as follows:

$$d_{hole} = d_b + \frac{1}{16}\,in. = \left(\frac{3}{4} + \frac{1}{16}\right) in. = \frac{13}{16}\,in.$$

$$l_c = 1\frac{1}{2}\,in. - \frac{1}{2} d_{hole} = 1\frac{1}{2}\,in. - \frac{1}{2}\left(\frac{13}{16}\,in.\right) = 1.09\,in.$$

$$R_n = (n-1)\left(1.5 l_c t F_u \leq 3.0 d t F_u\right)$$

$$R_n = (3-1)1.5(1.09\,in.)\left(\frac{3}{8}\,in.\right)58\,ksi\ x\ 2\ Plates = 142.25\,kips$$

$$\leq (3-1)3.0\left(\frac{3}{4}\,in.\right)\left(\frac{3}{8}\,in.\right)58\,ksi\ x\ 2\ Plates = 195.75\,kips \qquad \therefore R_n = 142.25\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(142.25\ kips)$ $= 106.69\ kips\ \ (474.56\ \mathrm{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{142.25\ kips}{2.00}$ $= 71.13\ kips\ \ (316.39\ \mathrm{kN})$

3.3.2.5 Bolt Bearing in the Beam

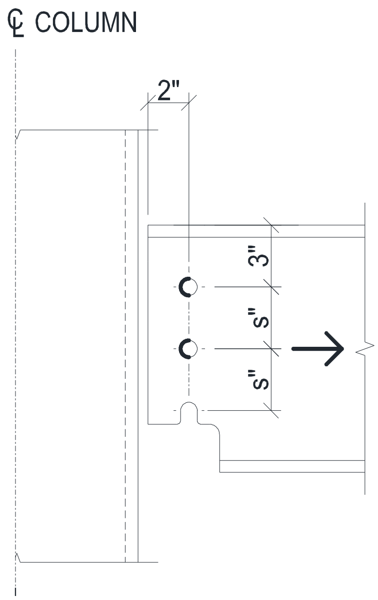
The available bearing strength at bolt holes shall be determined for the limit state of bearing as follows:

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

The nominal bearing strength of the connected material is determined as follows:

For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force when deformation at the bolt hole at service load is <u>not</u> a design consideration

$$R_n = 1.5 l_c t F_u \leq 3.0 d t F_u \qquad \text{(J3-6b)}$$



NOTE: The incoming beam delivers an axial tensile force pulling the beam away from the column.

The contribution of the dowel (lowermost bolt) to bearing is ignored due to the presence of the saddle on the beam cope.

The bearing strength on the bolt holes in the beam is calculated <u>per inch</u> of beam web thickness.

$$\therefore R_n = (n-1)\cdot(1.5 l_c F_u \leq 3.0 d F_u)$$

The ASTM A992 wide flange beam has the following material specifications:

$$F_y = 50\ ksi$$

$$F_u = 65\ ksi$$

The available bearing strength is as calculated as follows:

$$d_{hole} = d_b + \frac{1}{16}\,in. = \left(\frac{3}{4} + \frac{1}{16}\right) in. = \frac{13}{16}\,in.$$

$$l_c = 2.00\,in. - \frac{1}{2}\left(\frac{13}{16}\,in.\right) = 1.59\,in.$$

$$R_n = (3-1)1.5(1.59\,in.)65\,ksi = 310.05\,kips/in.$$

$$\leq (3-1)3.0\left(\frac{3}{4}\,in.\right)65\,ksi = 292.50\,kips/in. \qquad \therefore R_n = 292.50\,kips/in.$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(292.50\,\frac{kips}{in.}\right)$ $= 219.38\,\frac{kips}{in.}\ \left(38.42\,\frac{\mathrm{kN}}{\mathrm{mm}}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{292.50\,\frac{kips}{in.}}{2.00}$ $= 146.25\,\frac{kips}{in.}\ \left(650.52\,\frac{\mathrm{kN}}{\mathrm{mm}}\right)$

3.3.2.6 Block Shear

The available strength for the limit state of block shear rupture along a shear failure path or paths and a perpendicular tension failure path shall be taken as

$$R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(J4-5)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

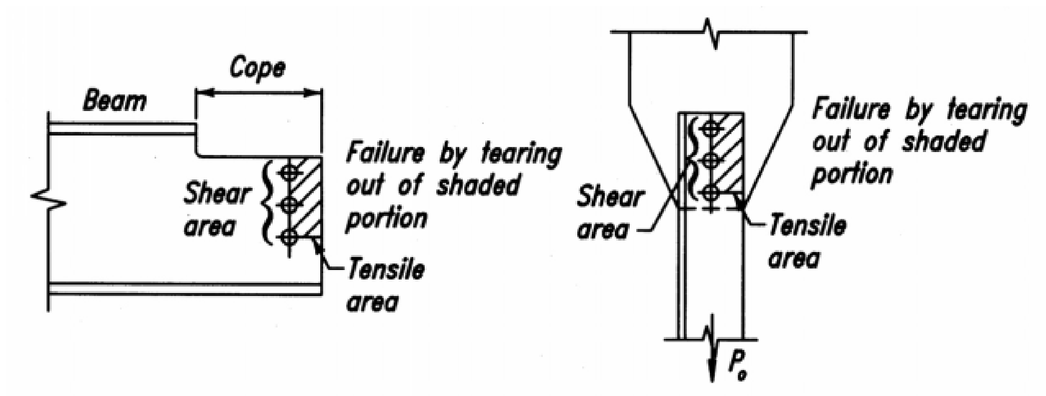
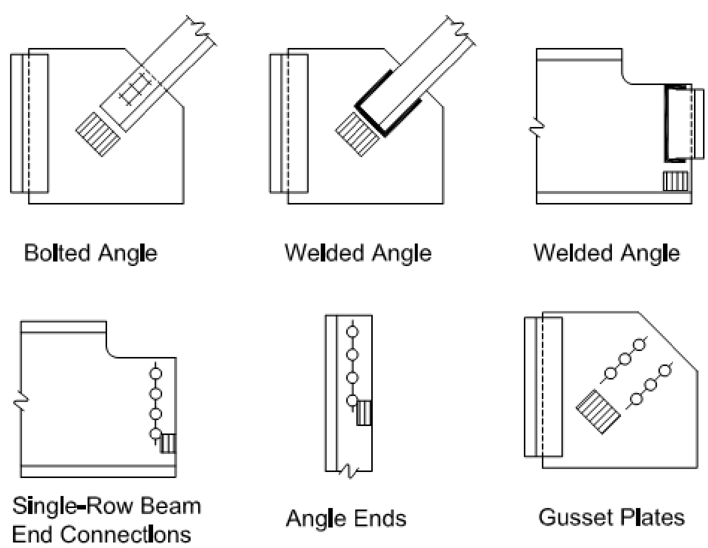


Figure 101: Failure surfaces for block shear rupture limit state.



(a) Cases for which $U_{bs} = 1.0$

Figure 102: Block shear tensile stress distributions (AISC 360-16 Commentary J4).

The double shear tab connection to a column can be categorized as a case where the tensile stress for block shear is uniform. Therefore, the reduction factor $U_{bs} = 1.0$.

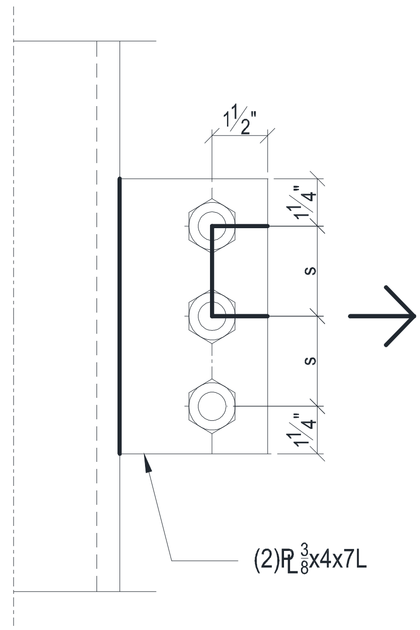
The following calculations determine the block shear strength for the double shear tab and the beam.

Block Shear Strength in the Plates

The ASTM A36 plates have the following material specifications:

$$F_y = 36\ ksi$$
$$F_u = 58\ ksi$$

NOTE: The thickened line on this figure represents the block-shear path in the shear tab. The contribution of the dowel (lowermost bolt) to block shear is ignored due to the presence of the saddle on the beam cope.



Using n to denote the number of bolts, the net shear, net tension and gross shear areas are calculated as shown below:

$$A_{nv} = 2\left(1\frac{1}{2}\ in. - \frac{1}{2}\left(\frac{13}{16} + \frac{1}{16}\right) in.\right) t_p\ x\ 2$$
$$= 2\left(1\frac{1}{2}\ in. - \frac{1}{2}\left(\frac{7}{8}\ in.\right)\right)\left(\frac{3}{8}\ in.\right) x\ 2 = 1.59\ in.^2$$

$$A_{nt} = (n-2)\left(2.3125\ in. - \frac{7}{8}\ in.\right) t_p\ x\ 2$$
$$= (3-2)\left(2.3125\ in. - \frac{7}{8}\ in.\right)\left(\frac{3}{8}\ in.\right) x\ 2 = 1.07\ in.^2$$

$$A_{gv} = 2\left(1\frac{1}{2}\ in.\right)\left(\frac{3}{8}\ in.\right) x\ 2 = 2.25\ in.^2$$

The block shear strength is calculated as follows:

$$R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt}$$
$$= 0.60(58\ ksi)1.59\ in.^2 + (1.0)(58\ ksi)1.07\ in.^2 = 117.39\ kips$$

$$\leq 0.60F_y A_{gv} + U_{bs} F_u A_{nt}$$
$$= 0.60(36\ ksi)2.25\ in.^2 + (1.0)(58\ ksi)1.07\ in.^2 = 110.66\ kips$$

$$R_n = 110.66\ kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(110.66\ kips)$ $= 82.99\ kips\ \ (369.14\,\mathrm{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{110.66\ kips}{2.00}$ $= 55.33\ kips\ \ (246.10\,\mathrm{kN})$

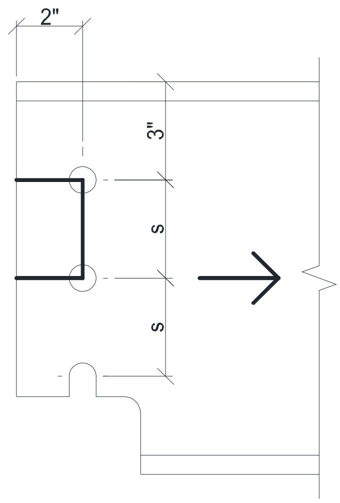
Block Shear Strength in the Beam

The ASTM A992 beam has the following material specifications:

$$F_y = 50\ ksi$$
$$F_u = 65\ ksi$$

The block shear strength of the beam will be determined per inch of beam web thickness.

The thickened lines in the figure on the right represent the block-shear path.



Using n to denote the number of bolts, the net shear, net tension and gross shear areas are calculated as shown below:

NOTE: The contribution of the dowel (lowermost bolt) to block shear is ignored due to the presence of the saddle on the beam cope.

$$A_{nv} = 2\left(2.00\ in. - \frac{1}{2}\left(\frac{13}{16} + \frac{1}{16}\right) in.\right)\frac{t_w}{t_w}$$
$$= 2\left(2.00\ in. - \frac{1}{2}\left(\frac{13}{16} + \frac{1}{16}\right) in.\right)(1.0)$$
$$= 3.13\ \frac{in.^2}{in.}$$

$$A_{nt} = (n-2)\left(2.3125\ in. - \frac{7}{8}\ in.\right)\frac{t_w}{t_w}$$
$$= (3-2)\left(2.3125\ in. - \frac{7}{8}\ in.\right)(1.0)$$
$$= 1.43\ \frac{in.^2}{in.}$$

$$A_{gv} = 2(2.00\ in.)\frac{t_w}{t_w}$$
$$= 2(2.00\ in.)(1.0) = 4.00\ \frac{in.^2}{in.}$$

The block shear strength is calculated as follows:

$$U_{bs} = 1.0$$

$$\begin{aligned} R_n &= 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \\ &= 0.60(65\ ksi)3.13\frac{in.^2}{in.} + (1.0)(65\ ksi)1.43\frac{in.^2}{in.} = 215.02\frac{kips}{in.} \end{aligned}$$

$$\begin{aligned} &\leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \\ &= 0.60(50\ ksi)4.00\frac{in.^2}{in.} + (1.0)(65\ ksi)1.43\frac{in.^2}{in.} = 212.95\frac{kips}{in.} \end{aligned}$$

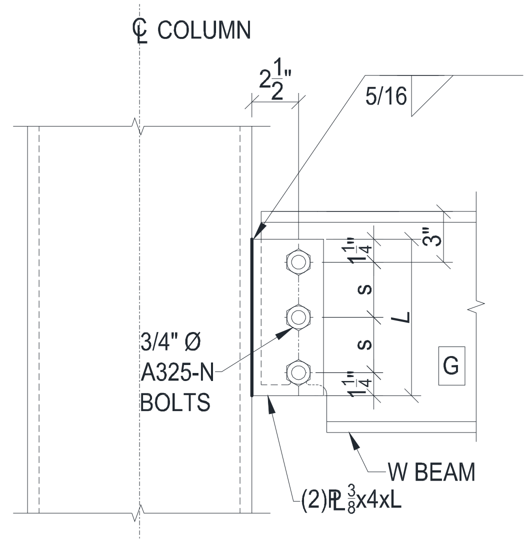
$$R_n = 212.95\frac{kips}{in.}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(212.95\frac{kips}{in.}\right)$ $= 159.71\frac{kips}{in.}\ \left(27.96\frac{\text{kN}}{\text{mm}}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{212.95\frac{kips}{in.}}{2.00}$ $= 106.47\frac{kips}{in.}\ \left(18.64\frac{\text{kN}}{\text{mm}}\right)$

3.3.2.7 Tensile Yielding and Tensile Rupture in the Plates

The design strength, ϕR_n, and the allowable strength, R_n/Ω, of affected and connecting elements loaded in tension shall be the lower value obtained according to the limit states of tensile yielding and tensile rupture.



(a) For tensile yielding of the element:

$$R_n = F_y A_g \qquad \text{(J4-1)}$$

$$\phi = 0.90 \ \text{(LRFD)} \qquad \Omega = 1.67 \ \text{(ASD)}$$

(b) For tensile rupture of the element:

$$R_n = F_u A_e \qquad \text{(J4-2)}$$

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$

where A_e, A_g, F_y and F_u are as defined in Sections D2, D3 and J4.1 of the *Specification*.

The effective net area of tension members shall be determined as follows:

$$A_e = A_n U$$

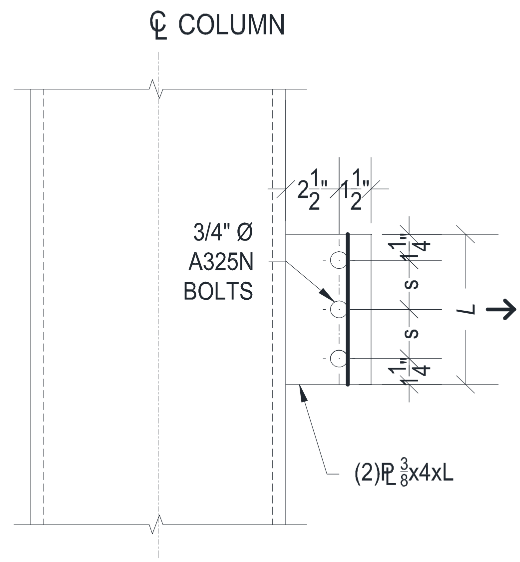
where U, the shear lag factor, is determined using Table D3.1 of the *Specification*.

NOTE: *Specification* Commentary D3 states the following: "There is insufficient data for establishing a value of U if all lines have only one bolt, but it is probably conservative to use A_e equal to the net area of the connected element. The limit states of block shear (Section J4.3) and bearing (Section J3.10), which must be checked, will probably control the design."

For tensile yielding in the gross section:

$$A_g = L \cdot t_p \ x \ 2 \ Plates$$
$$= (7.125 in.)(t_p) \ x \ 2 \ Plates$$
$$= (7.125 \ in.)\left(\frac{3}{8} \ in.\right) x \ 2 \ Plates$$
$$= 5.34 \ in.^2$$

$$R_n = (36 \ ksi)(5.34 \ in.^2) = 192.24 \ kips$$



Note: The thickened line represents the gross section in the shear tabs. The weld is not shown for clarity.

The available design strength or allowable strength is:

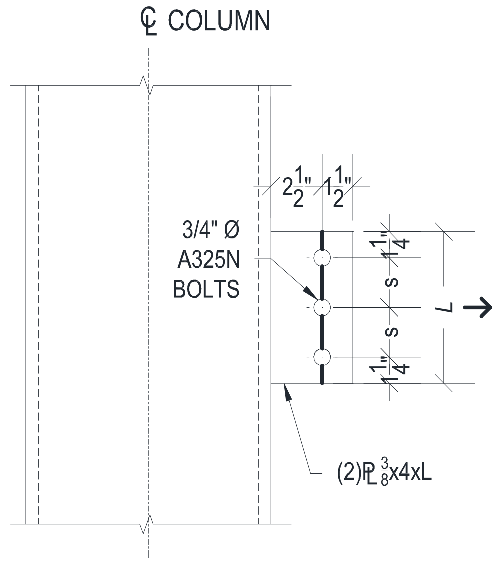
LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(192.24 \ kips)$ $= 173.01 \ kips \ \ (769.54 \text{ kN})$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{192.24 \ kips}{1.67}$ $= 115.11 \ kips \ \ (512.01 \text{ kN})$

For tensile rupture in the net section:

$$A_n = \left(L - n\left(\frac{13}{16}\,in. + \frac{1}{16}\,in.\right)\right)(t_p)\ x\ 2\ Plates$$
$$= \left(7.125\,in. - (3)\left(\frac{7}{8}\,in.\right)\right)\left(\frac{3}{8}\,in.\right)\ x\ 2\ Plate$$
$$= 3.37\,in.^2$$

$$A_e \approx A_n = 3.37\,in.^2$$

$$R_n = (58\,ksi)(3.37\,in.^2) = 195.46\,kips$$



Note: The thickened lines represent the net section in the shear tabs.

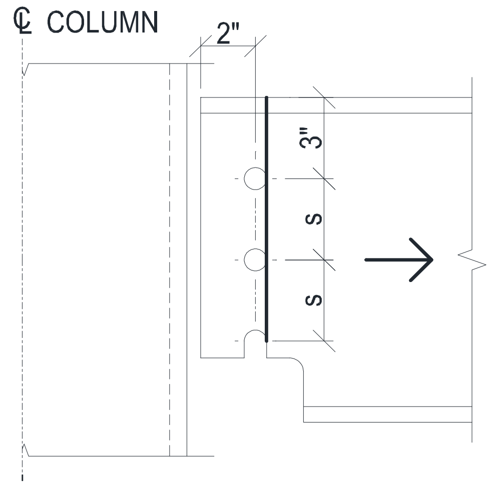
The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(195.46\,kips)$ $= 146.59\,kips\ \ (652.03\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{195.46\,kips}{2.00}$ $= 97.73\,kips\ \ (434.70\ \text{kN})$

Note: tensile rupture is the governing limit state of the two.

3.3.2.8 Tensile Yielding and Tensile Rupture in the Beam

The tensile yielding strength of the beam will be determined per inch of beam web thickness.



Note: The thickened line represents the gross section in the beam.

For tensile yielding in the gross section:

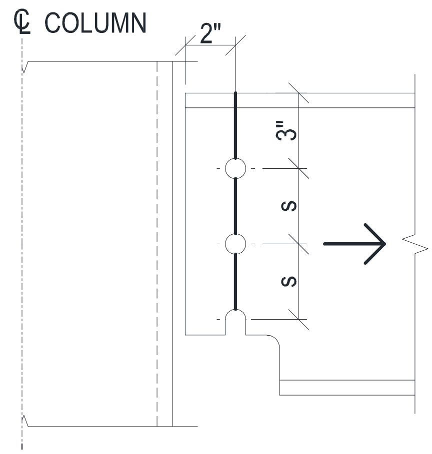
$$A_g = \left((n-1)2.3125in. + 1.75in. + 1.25in.\right)\left(\frac{t_w}{t_w}\right)$$
$$= \left((3-1)2.3125in. + 3.00in.\right)(1.0)$$
$$= 7.62\ in.^2 / in.$$

$$R_n = F_y A_g$$
$$= (50\ ksi)\left(7.62\frac{in.^2}{in.}\right) = 381.00\frac{kips}{in.}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90\left(381.00\frac{kips}{in.}\right)$ $= 342.90\frac{kips}{in.}\ \left(60.04\frac{\text{kN}}{\text{mm}}\right)$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{381.00\frac{kips}{in.}}{1.67}$ $= 228.14\frac{kips}{in.}\ \left(39.95\frac{\text{kN}}{\text{mm}}\right)$

The tensile rupture strength of the beam will be determined per inch of beam web thickness.



Note: The thickened lines represent the net section in the beam.

For tensile rupture in the net section:

$$A_n = \left(7.625\,in. - (n-0.5)\left(\frac{13}{16}\,in. + \frac{1}{16}\,in.\right)\right)\left(\frac{t_w}{t_w}\right)$$

$$= \left(7.625\,in. - (3-0.5)\left(\frac{13}{16}\,in. + \frac{1}{16}\,in.\right)\right)(1.0)$$

$$= 5.43\,\frac{in.^2}{in.}$$

$$A_e \approx A_n = 5.43\,\frac{in.^2}{in.}$$

$$R_n = (65\,ksi)\left(5.43\,\frac{in.^2}{in.}\right) = 352.95\,\frac{kips}{in.}$$

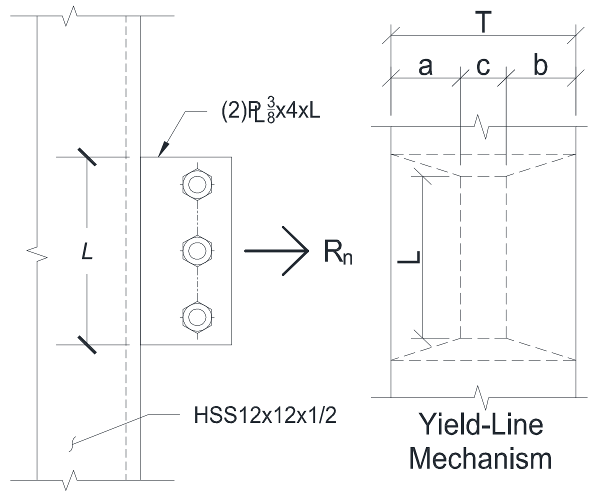
The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(352.95\,\frac{kips}{in.}\right)$ $= 264.71\,\frac{kips}{in.}\ \left(46.35\,\frac{\text{kN}}{\text{mm}}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{352.95\,\frac{kips}{in.}}{2.00}$ $= 176.47\,\frac{kips}{in.}\ \left(30.90\,\frac{\text{kN}}{\text{mm}}\right)$

Note: tensile rupture is the governing limit state of the two.

3.3.2.9 HSS Plastification

The design strength, ϕR_n, and the allowable strength, R_n/Ω, of connections shall be determined in accordance with the provisions of *Manual* Part 9 and *Specification* Chapter K.



$$R_n = \frac{t^2 F_y}{2}\left(\frac{(a+b)\left(4\sqrt{\frac{Tab}{a+b}}+L\right)}{ab}\right)Q_f$$

(9-30)

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

T = the width of the HSS wall
L = the length over which the load is delivered (i.e., the length of the shear tabs)
a = the distance from the edge of the shear tab to one edge of the HSS wall
b = the distance from the edge of the shear tab to the other edge of the HSS wall
c = the width over which the load is delivered
= $t_w + 1/8 \text{ in.} + 2t_p$
Q_f = the chord-stress interaction parameter

The available strength, R_n, is reduced by 50% when the concentrated force is applied at a distance from the member end that is less than $2\sqrt{Tab/(a+b)}$; this condition should be checked when the connection is located near the top of the column (e.g., at the roof) unless a cap plate is welded to the column to restrain deformations of the column walls.

From *Specification* Eq. K2-3, Q_f is determined as follows:

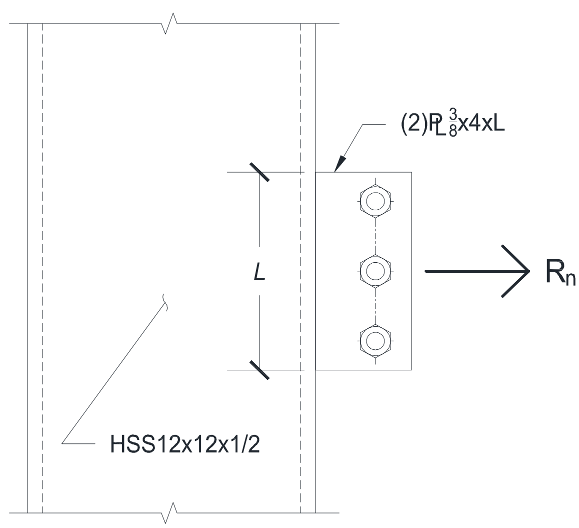
$Q_f = 1$ for connecting surface in tension,
$Q_f = 1.0 - 0.3U(1+U)$ for connecting surface in compression.

Since the column in question only supports gravity loads, it is assumed that the column is always in compression; thus, the second formula for Q_f applies. U is defined as the utilization ratio of the column wall; the calculations herein assume that the column wall has a utilization ratio of 0.90 (conservative). Consequently, the chord-stress interaction parameter, Q_f, is determined as follows:

$$\begin{aligned} Q_f &= 1.0 - 0.3U(1+U) \\ &= 1.0 - 0.3(0.9)(1+0.9) \\ &= 0.487 \end{aligned}$$

The HSS plastification strength will be determined per square inch of design wall thickness. The beam with the thinnest web that can be used with this connection is a W12x14 ($t_w = 0.200$ in.); this web thickness will be assumed for the following calculations for conservatism. (Note that changes in the beam web thickness contribute very little to the available strength; even if the beam web thickness were thrice the assumed value, the wall strength would only increase by ~2% for this HSS12x12 column.)

For the HSS12x12x½ with two ⅜ in. shear tabs:



$$F_y = 50\ ksi$$

$$t = t_{des} = 0.93\left(\frac{1}{2}in.\right) = 0.465\,in.$$

$$T = B = 12\,in.$$

$$L = 7.125\ in.$$

$$\begin{aligned} c &= t_w + \frac{1}{8}in. + 2t_p \\ &= 0.200\,in. + \frac{1}{8}in. + 2\left(\frac{3}{8}in.\right) \\ &= 1.075\,in. \end{aligned}$$

$$\begin{aligned} a = b &= \frac{T-c}{2} = \frac{12\,in. - 1.075\,in.}{2} \\ &= 5.4625\,in. \end{aligned}$$

The available strength for the limit state of HSS plastification is computed below:

$$R_n = \frac{t^2 F_y}{2}\left(\frac{(a+b)\left(4\sqrt{\frac{Tab}{a+b}}+L\right)}{ab}\right)Q_f\left(\frac{1}{t^2}\right)$$

$$R_n = \frac{50\,ksi}{2}\left(\frac{(2\times 5.4625\,in.)\left(4\sqrt{\frac{(12\,in.)(5.4625\,in.)^2}{2\times 5.4625\,in.}}+7.125\,in.\right)}{(5.4625\,in.)^2}\right)(0.487)\left(\frac{t_{des}{}^2}{t_{des}{}^2}\right)$$

$$R_n = \frac{50\,ksi}{2}\left(\frac{328.02\,in.^2}{29.84\,in^2}\right)(0.487)(1.0)$$

$$R_n = 133.83\,\frac{kips}{in.^2}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 1.00\left(133.83\,\frac{kips}{in.^2}\right)$ $= 133.83\,\frac{kips}{in.^2}\ \left(0.923\,\frac{\text{kN}}{\text{mm}^2}\right)$	$\frac{R_n}{\Omega} = \frac{1}{1.50}\left(133.83\,\frac{kips}{in.^2}\right)$ $= 89.22\,\frac{kips}{in.^2}\ \left(0.615\,\frac{\text{kN}}{\text{mm}^2}\right)$

If the connection is located near the top of the column, the available strength, R_n, may need to be reduced by 50%. The critical distance for this case is computed as follows.

$$L_{crit} = 2\sqrt{\frac{Tab}{a+b}} = 2\sqrt{\frac{(12\,in.)(5.4625\,in.)^2}{2\times 5.4625\,in.}} = 11.45\,in.$$

If the distance from the centerline of the shear plates to the top of the column is less than L_{crit}, then the available design strength or allowable strength is reduced by 50%:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00\left(133.83\,\frac{kips}{in.^2}\right)\times 0.50$ $= 66.92\,\frac{kips}{in.^2}\ \left(0.461\,\frac{\text{kN}}{\text{mm}^2}\right)$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{1}{1.50}\left(133.83\,\frac{kips}{in.^2}\right)\times 0.50$ $= 44.61\,\frac{kips}{in.^2}\ \left(0.308\,\frac{\text{kN}}{\text{mm}^2}\right)$

3.3.2.10 Design Aid

Table 6 shows the available axial tensile strength of bolted/welded double shear tab connections. The table is a design aid for all bolted/welded double shear tab connections that consist of seven (7) bolts or fewer in a single row.

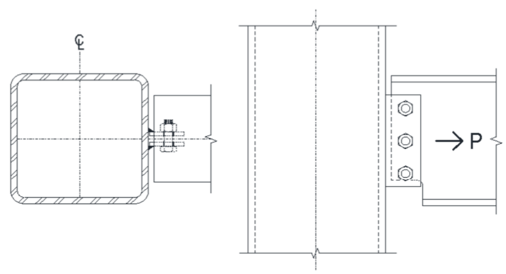
TABLE 6: AVAILABLE AXIAL TENSILE STRENGTH OF BOLTED/WELDED DOUBLE SHEAR TAB CONNECTIONS (ENGLISH).

Available Axial Tensile Strength of Bolted/Welded Double Shear Tab Connections to 12-in. wide HSS Columns

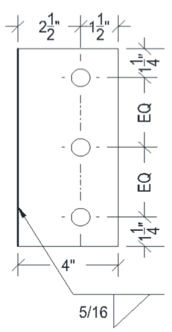
$\frac{3}{4}$ – in.
A325-N
BOLTS

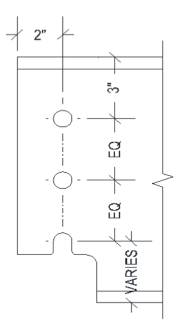
HSS	F_y =	50 ksi
	F_u =	62 ksi
Beam	F_y =	50 ksi
	F_u =	65 ksi
Plate	F_y =	36 ksi
	F_u =	58 ksi

W36, 33, 30, 27, 24, 21, 18, 16, 14, 12



L_{crit} = 11.45 in.





Number of Bolts (Beam Depth)	Length of Plate, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		*HSS Wall Available Strength per Square of Thickness, kips/sq.in.		Bolts, Welds, and Plates Available Strength, kips	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD
3 (W12)	7.1	106	160	89	133	47	71
4 (W14)	9.4	153	230	96	144	71	107
5 (W16)	11.8	200	300	102	154	95	143
6 (W18, 21)	14.1	246	370	109	164	119	178
7 (W24, 27, 30, 33, 36)	16.4	293	440	116	175	143	214

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-16 Specification. All values tabulated above have been rounded down to the nearest whole number.
2. Values are based on one 5/16-in fillet weld on each plate. The minimum HSS design wall thickness, t_{des}, to develop the full strength of the welds is 1/4-in.
3. *To obtain the HSS wall capacity for a given design wall thickness, t_{des}, multiply the tabulated value by the *square* of t_{des}. If the distance from the centerline of the connection to the top of the column is less than L_{crit} (above), then reduce the HSS wall capacity by 50%.
4. Limit states include weld strength, bolt shear, bearing at the bolt holes, block shear, tensile yielding, tensile rupture, and HSS wall plastification (assuming t_w = 0.200-in.).
5. All values assume A325-N bearing type connections in STD holes. Bolt spacing is 2-5/16" for all beam sizes. The saddle on the beam cope is treated as a long-slotted hole parallel to the load. Distance to top bolt from top of steel is assumed to be 3".
6. n = Number of bolts in connection.
7. L = Length of plates, in. Values are based on a plate thickness of ⅜".

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Checked by: I. You

3.3.3 Combined Shear and Drag Capacity

When the connection is subjected to both shear and drag loads, the available capacity of the connection depends on the angle of the resultant of the applied loads. The available capacity of the connection is determined from the limit states in three independent directions: the resultant direction, the pure shear direction, and the pure drag direction.

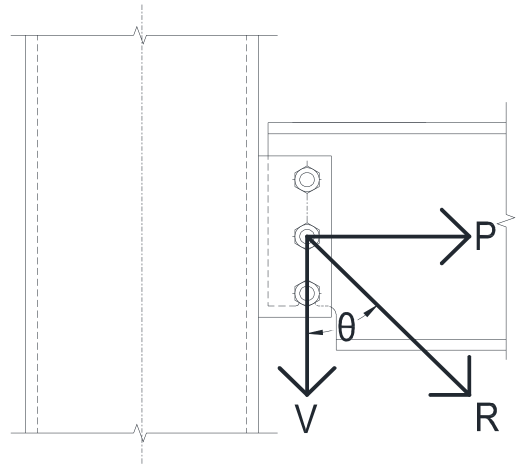


Figure 103: Combined Shear and Drag Loads.

For the formulations that follow, the following definitions apply:

P = Drag Load, kips or kN
V = Shear Load, kips or kN
R = Resultant of Shear and Drag Loads, kips or kN
θ = Angle that the Resultant Load makes with respect to the Shear Load.

The angle that the Resultant Load makes with respect to the Shear Load is determined as follows:

$$\theta = \arctan\left(\frac{P}{V}\right)$$

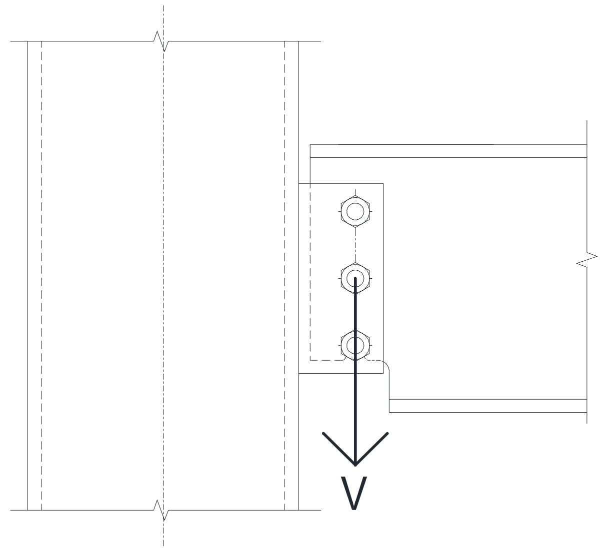
The Resultant Load is determined as follows:

$$R = \sqrt{V^2 + P^2}$$

3.3.3.1 Shear Component

When there is a combined shear and drag load on the connection, the capacity of the connection in the pure shear direction is the governing limit state from the following applicable limit states for shear:

1. Block Shear
2. Shear Yielding (Shear Tabs and Beam)
3. Shear Rupture (Shear Tabs and Beam)
4. HSS Wall Punching Shear

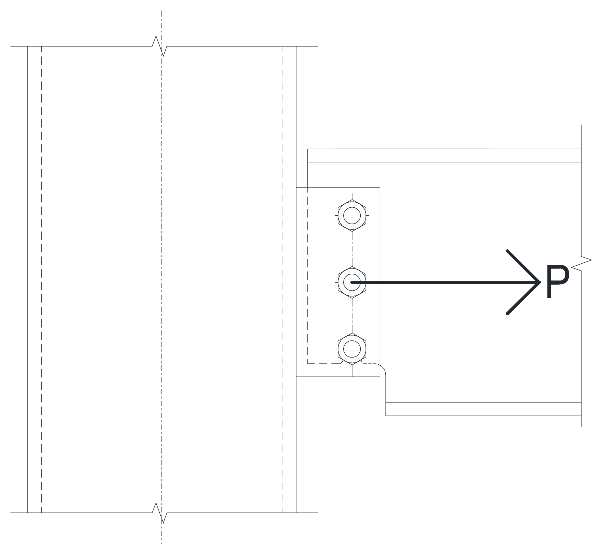


The demand-to-capacity ratio in the shear direction is V divided by the governing limit state for the shear component. Note that the slenderness of the HSS wall was checked in Section 3.3.1.9

3.3.3.2 Drag Component

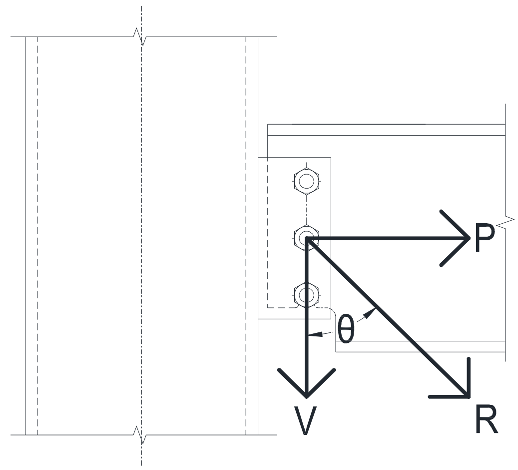
When there is a combined shear and drag load on the connection, the capacity of the connection in the pure drag direction is the governing limit state from the following applicable limit states for drag:

1. Block Shear
2. Tensile Yielding (Shear Tabs and Beam)
3. Tensile Rupture (Shear Tabs and Beam)
4. HSS Plastification



The demand-to-capacity ratio in the drag direction is P divided by the governing limit state for the drag component. Note that HSS Plastification does not apply to the resultant load since the drag load is the component of the resultant load that induces this failure mechanism.

3.3.3.3 Resultant Component



When there is a combined shear and drag load on the connection, the capacity of the connection in the resultant direction is the governing limit state from the following applicable limit states for the resultant:

1. Weld Strength
2. Bolt Group Shear
3. Bearing at the Bolt Holes (Shear Tabs and Beam)

The demand-to-capacity ratio in the shear direction is ***R*** divided by the governing limit state for the resultant.

Weld Strength

The strength of the weld, in the direction of the resultant load, is determined in accordance with *Specification* Chapter J, as shown below:

$$R_n = 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right)A_{we} \qquad \text{(J2-5)}$$

The double shear tab connection consists of a weld on each tab. Therefore, the weld strength is the strength of both welds. For the sake of conservatism, the increase in weld strength due to the load angle will be ignored.

Bolt Group Shear Strength

The bolt group shear strength is calculated as follows depending on the drag load:

1. If the drag load is zero, then the bolt group shear strength is calculated based on all n bolts.

2. If the drag load is smaller than the maximum strength of the saddle (shear rupture), the shear strength in the direction of the resultant load is the sum of the shear strengths of all the bolts (including the dowel). Thus, $R_n = nF_{nv}A_b$.

3. If the drag load is greater than the maximum strength of the saddle (shear rupture), the shear strength in the direction of the resultant load is the strength of $(n - 1)$ bolts: the contribution of the dowel to the bolt group shear strength is ignored. Thus, $R_n = (n - 1)F_{nv}A_b$.

The contribution of the dowel to the bolt group shear strength depends on the resultant angle. To determine the range of θ where the dowel is effective in contributing to the bolt group, the shear rupture strength of the beam web is equated to the shear strength of the bolt in double shear. The length of the shear plane is equal to the clear distance from the edge of the bolt hole to the edge of the beam web along the direction of the load angle.

$$l_c = \frac{L_{eh}}{\sin(\theta)} - \frac{d_h}{2}$$
$$\phi R_{n,web} = \phi 0.6 F_u t_w l_c$$
$$\phi R_{n,bolt} = 2\phi F_{nv} A_b$$

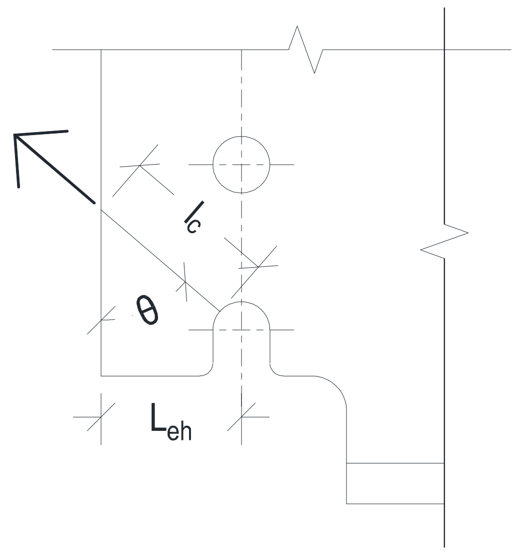


Figure 104: Shear Rupture in Beam Web.

The capacities of the beam web and bolt are equated and rearranged as follows to determine the critical angle at which the beam web strength exceeds the bolt strength (note that $\phi = 0.75$ for both limit states):

$$\phi 0.6 F_u t_w l_c = 2\phi F_{nv} A_b$$
$$0.6 F_u t_w \left(\frac{L_{eh}}{\sin(\theta)} - \frac{d_h}{2} \right) = 2 F_{nv} A_b$$
$$\frac{L_{eh}}{\sin(\theta)} = \frac{2 F_{nv} A_b}{0.6 F_u t_w} + \frac{d_h}{2}$$
$$\sin(\theta) = \frac{L_{eh}}{\frac{F_{nv} A_b}{0.3 F_u t_w} + \frac{d_h}{2}}$$
$$\theta_{crit} = \arcsin\left(\frac{L_{eh}}{\frac{F_{nv} A_b}{0.3 F_u t_w} + \frac{d_h}{2}} \right)$$

For the double shear tab connection, only the beam web thickness will change the value of θ_{crit}: all other values in the equation are constant. When the angle of the resultant is less than the critical angle, the dowel will be fully effective; otherwise, the contribution of the dowel is ignored.

Bearing at the Bolt Holes

The available bearing strength at the bolt holes, in the direction of the resultant load, is determined in accordance with *Specification* Chapter J, as shown below.

$$R_n = 1.5 l_c t F_u \le 3.0 d t F_u \qquad \text{(J3-6b)}$$

Where

l_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in.

For both the plates and the beam web, when <u>$\theta \le 21°$</u> (see Figure 105) the clear distance of the upper bolt encroaches on the bolt below it. For such cases, the clear distance, l_c, is limited to the bolt spacing less the diameter of one bolt hole, d_h.

$$l_c = s - d_h$$
$$= s - \frac{13}{16} \, in.$$

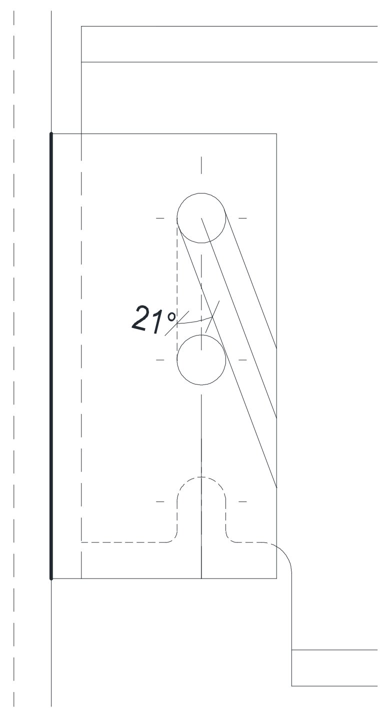


Figure 105: Clear distance for $\theta \le 21°$.

NOTE: the angle of encroachment will change based on the bolt spacing.

Using n to denote the number of bolts in the connection, for a given angle, θ, the value of the clear distance, l_c, is determined as follows (see Figure 106).

For the upper $(n - 1)$ bolts in the plates, or all bolts in the beam web (including the dowel), the clear distance can be expressed as follows:

$$\left(r + l_c\right)\sin\left(\theta\right) = L_{eh}$$

And therefore,

$$l_c = \frac{L_{eh}}{\sin\left(\theta\right)} - r$$

Where

L_{eh} = Edge distance in the horizontal direction.

r = Radius of bolt hole.

For ¾ in. diameter ASTM A325-N high strength bolts,

$$r = \frac{d_h}{2} = \frac{13/16\,in.}{2} = \frac{13}{32}\,in.$$

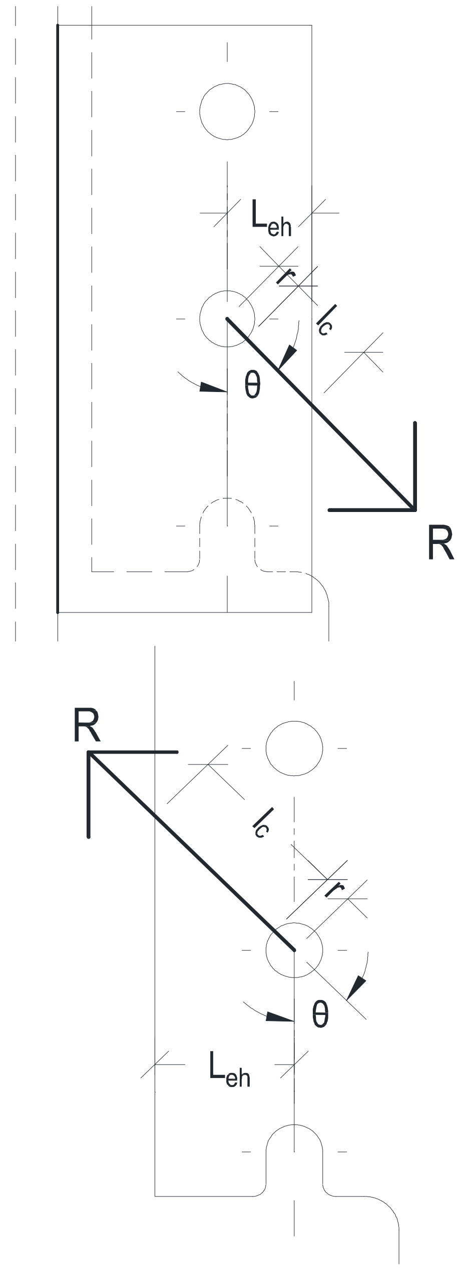


Figure 106: Clear Distance for Bolts in the Plates and the Beam Web.

For the dowel in the plates (presuming the dowel contributes to the bolt group shear strength), the value of the clear distance is dependent on the angle of the resultant load and corresponding nearest edge. The transition angle, θ_{trn}, is the angle at which the resultant load coincides with the corner of the plate and is determined as follows:

$$\begin{aligned}\theta_{trn} &= \arctan\left(\frac{L_{eh}}{L_{ev}}\right)\\ &= \arctan\left(\frac{1.50\,in.}{1.25\,in.}\right)\\ &= 50.2^\circ\end{aligned}$$

Where

L_{ev} = vertical edge distance.
L_{eh} = horizontal edge distance.

As can be seen in Figure 107, if $\theta < \theta_{trn}$, then

$$\begin{aligned}&(r + l_c)\cos(\theta) = L_{ev}\\ &\therefore\ l_c = \frac{L_{ev}}{\cos(\theta)} - r\end{aligned}$$

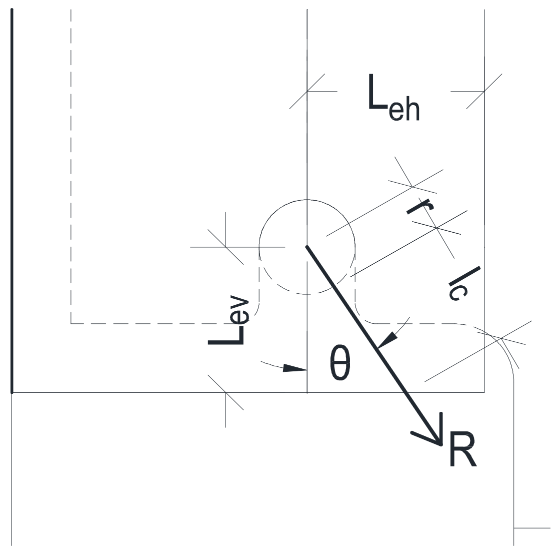


Figure 107: Clear Distance at the Dowel in the Plates when $\theta < \theta_{trn}$.

As can be seen in Figure 108, if $\theta \geq \theta_{trn}$, then

$$(r + l_c)\sin(\theta) = L_{eh}$$

$$\therefore\ l_c = \frac{L_{eh}}{\sin(\theta)} - r$$

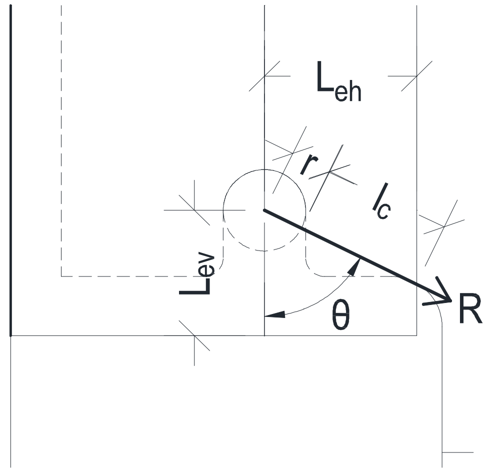


Figure 108: Clear Distance at the Dowel in the Plates when $\theta \geq \theta_{trn}$.

3.3.4 Example

The following is an example that illustrates how to determine the demand-to-capacity ratio (DCR) of a welded/bolted double shear tab connection. The beam is a W12x35 (t_w = 0.300 in.) in a 3-bolt configuration. Given a Shear Load, ***V***, of 22 kips (LRFD) and a Drag Load, ***P***, of 27.5 kips (LRFD), the DCR for the connection is determined as shown below.

3.3.4.1 Limit States for the Resultant Load

For the resultant load, the load ***R*** is compared to the Weld Strength, Bolt Group Shear, and Bearing at the Bolt Holes.

The resultant load is determined as follows:

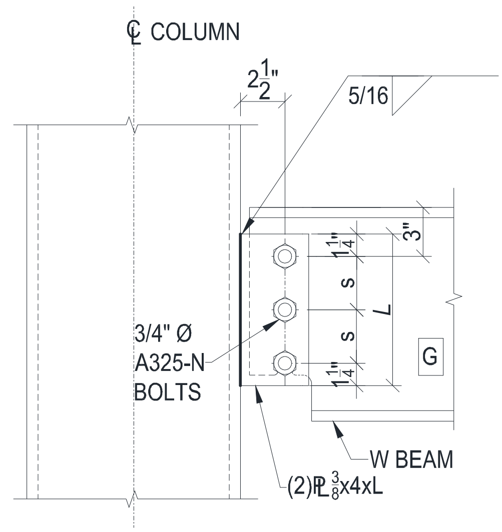
$$\begin{aligned} R &= \sqrt{V^2 + P^2} \\ &= \sqrt{(22\ kips)^2 + (27.5\ kips)^2} \\ &= 35.22\ kips \end{aligned}$$

The angle that the resultant load makes with respect to the shear load component is determined as follows:

$$\begin{aligned} \theta &= \arctan\left(\frac{P}{V}\right) \\ &= \arctan\left(\frac{27.5\ kips}{22\ kips}\right) \\ &= 51.34^\circ \end{aligned}$$

3.3.4.1.1 Weld Strength

In accordance with *Specification* Section J2.4, the available strength of welded joints is determined as follows:



$$R_n = F_{nw} A_{we} \qquad \text{(J2-3)}$$

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$

For fillet welds,

$$F_{nw} = 0.60 F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right) \qquad \text{(J2-5)}$$

The length of weld is equal to the length of the shear tab. The available strength is calculated as shown below.

The length of the weld is:

$$n = \text{Number of Bolts} = 3$$

$$L = 2\left(1\frac{1}{4}\,in.\right) + (n-1)\,2.375\,in. = 2.50in. + (3-1)\,2.3125\,in. = 7.125\,in.$$

The size of the weld is:

$$t_{we} = \left(\frac{\sqrt{2}}{2}\cdot\frac{5}{16}\,in.\right) = 0.22\,in.$$

The area of the weld (for both shear tabs) is:

$$A_{we} = t_{we}L = (0.22\,in.)(7.125\,in.) = 1.56\,in.^2\,/\,Plate$$

Ignoring the effect of the load angle, the total strength of the weld is:

$$\begin{aligned}
&\theta = 0^\circ \\
&R_n = 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}(\theta)\right)A_{we} \\
&\quad = 0.60(70\,ksi)\left(1.0 + 0.50\sin^{1.5}(0^\circ)\right)\left(1.56\,in.^2\,/\,Plate\right)\times 2\,Plates \\
&\quad = 131.04\,kips
\end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(131.04\ kips)$ $= 98.28\ kips\ (437.15\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{131.04\ kips}{2.00}$ $= 65.52\ kips\ (291.43\ \text{kN})$

3.3.4.1.2 Bolt Group Shear Strength

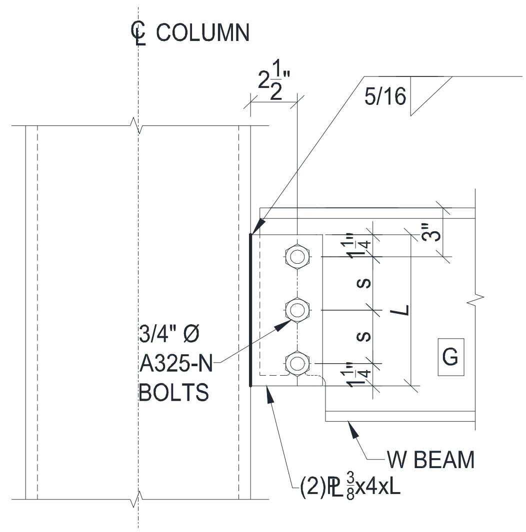
The design shear strength, ϕR_n, and the allowable shear strength, R_n/Ω, of snug-tightened or pretension high-strength bolt or threaded part shall be determined according to the limit states of shear rupture as follows:

$$R_n = F_{nv} A_b \qquad \text{(J3-1)}$$

$$\phi = 0.75 \;\; \text{(LRFD)} \qquad \Omega = 2.00 \;\; \text{(ASD)}$$

Per *Specification* Table J3.2, the nominal shear strength of A325 bolts when threads are not excluded from shear planes is

$$F_{nv} = 54\ ksi$$



The bolts are in standard holes with 2 5⁄16 in. spacing and 2 in. horizontal edge distance on the beam web. The critical angle, θ_{crit}, for determining the contribution of the dowel is determined below.

$$\begin{aligned}
\theta_{crit} &= \arcsin\left(\frac{L_{eh}}{\dfrac{F_{nv} A_b}{0.3 F_u t_w} + \dfrac{d_h}{2}}\right) \\
&= \arcsin\left(\frac{2.00\,in.}{\dfrac{(54\,ksi)(0.442\,in.^2)}{0.3(65\,ksi)(0.300\,in.)} + \dfrac{13/16\,in.}{2}}\right) \\
&= \arcsin\left(\frac{2.00\,in.}{4.486\,in.}\right) \\
&= 26.47^\circ
\end{aligned}$$

Since the resultant force angle, $\theta = 51.34°$, is greater than θ_{crit}, the contribution of the dowel is ignored.

Using n to denote the number of bolts and neglecting the contribution of the dowel, the available shear strength of ¾ in. diameter ASTM A325-N bolts in double-shear is:

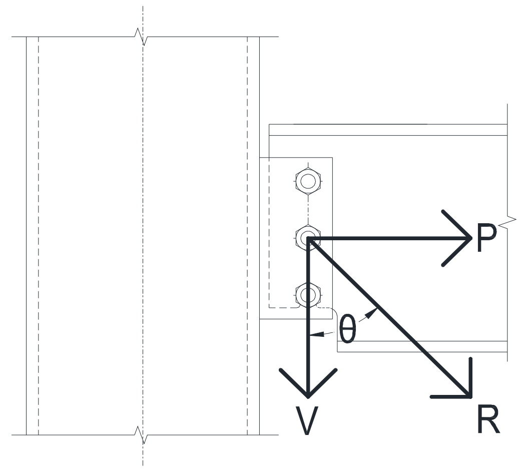
$$
\begin{aligned}
R_n &= (n-1)F_{nv}A_b \ x\ 2\ \textit{Shear Planes} \\
&= (3-1)\left((54\,ksi)\left(0.442\,in.^2\right)\right) x\ 2\ \textit{Shear Planes} \\
&= 95.47\ kips
\end{aligned}
$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(95.47\ kips)$ $= 71.60\ kips\ \ (318.48\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{95.47\ kips}{2.00}$ $= 47.74\ kips\ \ (212.35\ \text{kN})$

3.3.4.1.3 Bearing at the Bolt Holes

As explained in Section 3.3.3.3 regarding the resultant component, the contribution of the dowel to the bearing capacity of the beam is ignored if the angle of the resultant is greater than θ_{crit}. The resultant force angle, θ = 51.34° is greater than θ_{crit} and thus the dowel is ignored in the calculation of the bearing capacity.



For the Plates

Using n to denote the number of bolts, the available bearing strength on the plates is calculated as follows:

$$d_{hole} = \frac{13}{16}\,in.$$

$$l_c = \frac{L_{eh}}{\sin(\theta)} - \frac{d_h}{2} = \frac{1.50\,in.}{\sin(51.34°)} - \frac{13/16\,in.}{2} = 1.51\,in.$$

$$R_n = (n-1)(1.5 l_c t F_u \leq 3.0 d t F_u)$$

$$= (3-1)1.5(1.51\,in.)\left(2\times\frac{3}{8}\,in.\right)58\,ksi = 197.05\,kips$$

$$\leq (3-1)3.0\left(\frac{3}{4}\,in.\right)\left(2\times\frac{3}{8}\,in.\right)58\,ksi = 195.75\,kips \qquad \therefore R_n = 195.75\,kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(195.75\,kips)$ $= 146.81\,kips\ (653.01\,\text{kN})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{195.75\,kips}{2.00}$ $= 97.88\,kips\ (435.35\,\text{kN})$

For the Beam

Using n to denote the number of bolts, the available bearing strength on the beam web is calculated as follows:

$$d_h = \frac{13}{16}\,in.$$

$$l_c = \frac{L_{eh}}{\sin(\theta)} - \frac{d_h}{2} = \frac{2.00\,in.}{\sin(51.34°)} - \frac{13/16\,in.}{2} = 2.15\,in.$$

$$\begin{aligned} R_n &= (n-1)\left(1.5 l_c t F_u \le 3.0 d t F_u\right) \\ &= (3-1)1.5(2.15\,in.)(0.30\,in.)65\,ksi = 125.78\,kips \\ &\le (3-1)3.0(0.75\,in.)(0.30\,in.)65\,ksi = 87.75\,kips \qquad \therefore R_n = 87.75\,kips \end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(87.75\,kips)$ $= 65.81\,kips\ (292.72\text{ kN})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{87.75\,kips}{2.00}$ $= 43.88\,kips\ (195.18\text{ kN})$

3.3.4.2 DCR for the Resultant Component

The demand-to-capacity ratio (DCR) for the resultant component, $\boldsymbol{R}$, is determined as follows. For the resultant component, the limit states are Weld Strength, Bolt Group Shear Strength, and Bearing at the Bolt Holes.

NOTE: The capacities (LRFD) are typically rounded down to the nearest whole number (conservative).

Limit State	DCR (English Units)	
Weld Strength	$\text{DCR} = \dfrac{R}{\phi R_n} = \dfrac{35.2\ kips}{98\ kips} = 0.36$	$< 1.0 \quad \therefore OK$
Bolt Group Shear Strength	$\text{DCR} = \dfrac{R}{\phi R_n} = \dfrac{35.2\ kips}{71\ kips} = 0.50$	$< 1.0 \quad \therefore OK$
Bearing at the Bolt Holes: Plates	$\text{DCR} = \dfrac{R}{\phi R_n} = \dfrac{35.2\ kips}{146\ kips} = 0.24$	$< 1.0 \quad \therefore OK$
Bearing at the Bolt Holes: Beam	$\text{DCR} = \dfrac{R}{\phi R_n} = \dfrac{35.2\ kips}{65\ kips} = 0.54$	$< 1.0 \quad \therefore OK$
The maximum DCR for the Resultant Load, $\boldsymbol{R}$, is 0.54.		$< 1.0 \quad \therefore OK$

3.3.4.3 DCR for the Shear Component

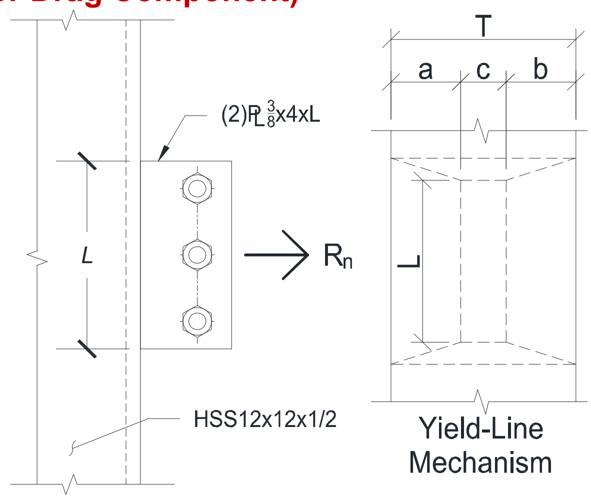
The DCR for the shear component, V, is determined as follows. For the shear component, the limit states are Block Shear, Shear Yielding, Shear Rupture, and HSS Punching Shear.

NOTE: The capacities (LRFD) are those determined in the "Shear Capacity" section of this document and are typically rounded down to the nearest whole number (conservative). Also note that the capacities for the beam are multiplied by the beam web thickness (W12x35 has t_w = 0.300 in.).

Limit State	DCR (English Units)	
Block Shear in the Plates (in the Shear Direction)	$\text{DCR} = \frac{V}{\phi R_n} = \frac{22\ kips}{106\ kips} = 0.21$	$< 1.0 \quad \therefore OK$
Shear Yielding: Plates	$\text{DCR} = \frac{V}{\phi R_n} = \frac{22\ kips}{115\ kips} = 0.19$	$< 1.0 \quad \therefore OK$
Shear Rupture: Plates	$\text{DCR} = \frac{V}{\phi R_n} = \frac{22\ kips}{87\ kips} = 0.25$	$< 1.0 \quad \therefore OK$
Shear Yielding: Beam	$\text{DCR} = \frac{V}{\phi R_n} = \frac{22\ kips}{\left(228.60 \frac{kips}{in.}\right)(0.300\ in.)} = \frac{22\ kips}{68\ kips} = 0.32$	$< 1.0 \quad \therefore OK$
Shear Rupture: Beam	$\text{DCR} = \frac{V}{\phi R_n} = \frac{22\ kips}{\left(158.82 \frac{kips}{in.}\right)(0.300\ in.)} = \frac{22\ kips}{47\ kips} = 0.47$	$< 1.0 \quad \therefore OK$
HSS Punching Shear	$\text{DCR} = \frac{V}{\phi R_n} = \frac{22\ kips}{\left(188.84 \frac{kips}{in.}\right)(0.300\ in.)} = \frac{22\ kips}{56\ kips} = 0.39$	$< 1.0 \quad \therefore OK$
The maximum DCR for the Shear Load, V, is 0.47.		$< 1.0 \quad \therefore OK$

3.3.4.4 Limit State of HSS Plastification (for Drag Component)

The design strength, ϕR_n, and the allowable strength, R_n/Ω, of connections shall be determined in accordance with the provisions of *Manual* Part 9 and *Specification* Chapter K.



$$R_n = \frac{t^2 F_y}{2}\left(\frac{(a+b)\left(4\sqrt{\frac{Tab}{a+b}}+L\right)}{ab}\right)Q_f$$

(9-30)

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

The limit state of HSS Plastification for the HSS12x12x1/2 column is computed below (refer to Section 3.3.2.9 for derivation of the available strength). The connection is not located near the top of the column and thus the 50% reduction in available strength does not apply.

$$t_{des} = 0.465\, in.$$

$$\begin{aligned} R_n &= \left(133.83\,\frac{kips}{in.^2}\right)t_{des}{}^2 \\ &= \left(133.83\,\frac{kips}{in.^2}\right)(0.465\, in.)^2 \\ &= 28.94\, kips \end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(28.94\, kips)$ $= 28.94\, kips \ (128.7\,\text{kN})$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{1}{1.50}(28.94\, kips)$ $= 19.29\, kips \ (85.82\,\text{kN})$

3.3.4.5 DCR for the Drag Component

The DCR for the drag component, $\boldsymbol{P}$, is determined as follows. For the drag component, the limit states are Block Shear, Tensile Yielding, Tensile Rupture, and HSS Plastification.

NOTE: The capacities (LRFD) are those determined in the "Drag Capacity" section of this document and are typically rounded down to the nearest whole number (conservative). Also note that the capacities for the beam are multiplied by the beam web thickness (W12x35 has t_w = 0.300 in.) and the capacity for the HSS Plastification limit state was computed on page 251.

Limit State	DCR (English Units)	
Block Shear in the Plates (in the Drag Direction)	$\text{DCR} = \dfrac{P}{\phi R_n} = \dfrac{27.5\ kips}{82\ kips} = 0.34$	$<1.0\ \therefore OK$
Block Shear in the Beam (in the Drag Direction)	$\text{DCR} = \dfrac{P}{\phi R_n} = \dfrac{27.5\ kips}{\left(159.71\dfrac{kips}{in.}\right)(0.300\ in.)} = \dfrac{27.5\ kips}{48\ kips} = 0.57$	$<1.0\ \therefore OK$
Tensile Yielding: Plates	$\text{DCR} = \dfrac{P}{\phi R_n} = \dfrac{27.5\ kips}{173\ kips} = 0.16$	$<1.0\ \therefore OK$
Tensile Rupture: Plates	$\text{DCR} = \dfrac{P}{\phi R_n} = \dfrac{27.5\ kips}{146\ kips} = 0.19$	$<1.0\ \therefore OK$
Tensile Yielding: Beam	$\text{DCR} = \dfrac{P}{\phi R_n} = \dfrac{27.5\ kips}{\left(342.90\dfrac{kips}{in.}\right)(0.300\ in.)} = \dfrac{27.5\ kips}{102\ kips} = 0.27$	$<1.0\ \therefore OK$
Tensile Rupture: Beam	$\text{DCR} = \dfrac{P}{\phi R_n} = \dfrac{27.5\ kips}{\left(264.71\dfrac{kips}{in.}\right)(0.300\ in.)} = \dfrac{27.5\ kips}{79\ kips} = 0.35$	$<1.0\ \therefore OK$
HSS Plastification:	$\text{DCR} = \dfrac{P}{\phi R_n} = \dfrac{27.5\ kips}{28\ kips} = 0.98$	$<1.0\ \therefore OK$
The maximum DCR for the Drag Load, $\boldsymbol{P}$, is 0.98.		$<1.0\ \therefore OK$

Based on the preceding DCR computations, the overall DCR for the connection is the maximum of all the DCRs (DCR = 0.98). This ratio is less than unity and therefore the connection is adequate to resist the concurrent shear and drag loads.

Table 7, below, shows the available strength and DCR for bolted/welded double shear tab connections for the given loads (V = 22 kips, P = 27.5 kips). Note that the DCR values that are reported on the table match those previously computed.

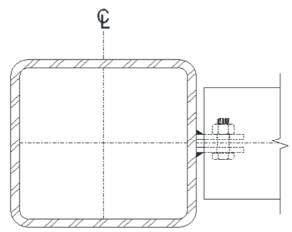
Table 7: Available Strength and DCR for Bolted/Welded Double Shear Tab Connections (V=22 kips, P=27.5 kips).

Available Strength and Demand-To-Capacity Ratio (DCR) of W12X35 Beam to HSS12X12X1/2 Column Bolted/Welded Double Shear Tab Connection

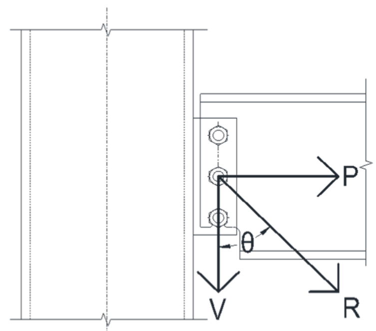
3/4-in. A325-STD BOLTS

HSS	F_y = 50 ksi
	F_u = 62 ksi
Beam	F_y = 50 ksi
	F_u = 65 ksi
Plate	F_y = 36 ksi
	F_u = 58 ksi

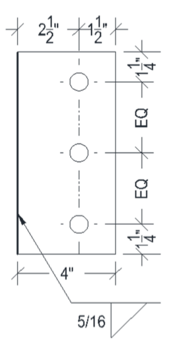
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12

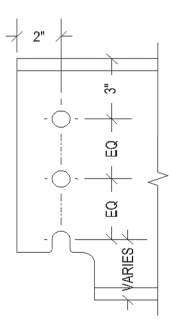


(Shear Tabs on Short Side of HSS.)



P = 27.5 kips
V = 22 kips
R = 35.2 kips
θ = 51.3 deg





Number of Bolts (Beam Depth)	Length of Plate, in.	Available Strength, kips						DCR, LRFD			
		RESULTANT		PURE SHEAR		PURE DRAG					
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	R	V	P	Max.
3 (W12)	***7.1***	***43***	***65***	***31***	***47***	***19***	***28***	***0.54***	***0.47***	***0.98***	***0.98***
4 (W14)	9.4	65	98	40	60	20	31	0.36	0.37	0.89	0.89
5 (W16)	11.8	87	131	48	72	22	33	0.27	0.31	0.83	0.83
6 (W18, 21)	14.1	109	164	57	85	23	35	0.21	0.26	0.79	0.79
7 (W24, 27, 30, 33, 36)	16.4	131	197	65	98	25	37	0.18	0.22	0.74	0.74

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-16 Specification. All values tabulated above have been rounded down to the nearest whole number.
2. Values are based on one 5/16-in fillet weld on each plate. The minimum HSS design wall thickness, t_{des}, to develop the full strength of the welds is 1/4-in.
3. The HSS wall must be nonslender for axial compression.
4. Limit states include weld strength, bolt shear, bearing at the bolt holes, block shear, tensile yielding, tensile rupture, shear yielding, shear rupture, HSS wall punching shear, and HSS wall plastification (evaluated conservatively with U=0.90 and assuming t_w = 0.200-in.).
5. All values assume A325-N bearing type connections in STD holes. Bolt spacing is 2-5/16" for all beam sizes. Distance to top bolt from top of steel is assumed to be 3".
6. n = Number of bolts in connection.
7. L = Length of plates, in. Values are based on a plate thickness of ⅜".
8. P = Drag Load, V = Shear Load, R = Resultant of Drag and Shear.

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3.3.5 Weak-Axis Shear Capacity

The following calculations determine the capacity of welded/bolted double shear tab connections in beam-to-column gravity connections when there is an applied shear load in the weak-axis of the beam. The available strength of welded/bolted double shear tab connections is determined from the limit states below:

- Welds
- Bolt Tension
- Shear Yielding in the Shear Tab
- Shear Rupture in the Shear Tab
- Tensile Yielding in the Shear Tab
- Tensile Rupture in the Shear Tab
- HSS Wall Flexural Yielding

The following calculations are based on the same connection geometry as Section 3.3.1. The incoming beam is assumed to have sufficient strength to transfer the weak-axis shear through the beam web.

3.3.5.1 Weld Strength

In accordance with *Specification* Section J2.4, the available strength of welded joints is determined as follows:

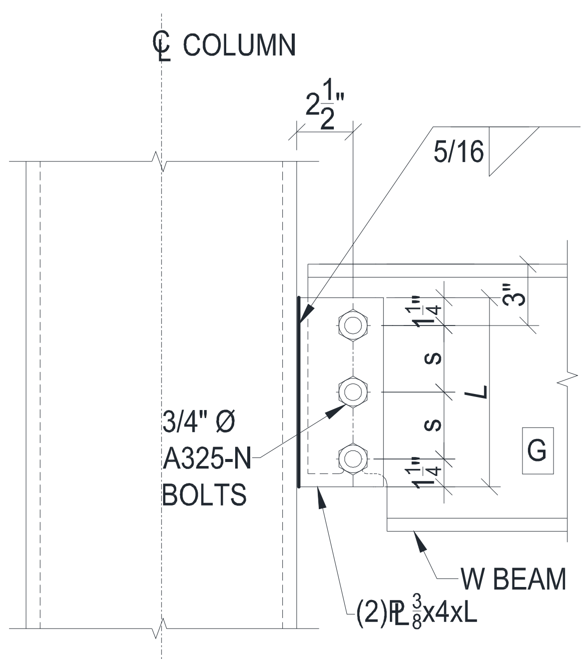
$$R_n = F_{nw} A_{we} \qquad \text{(J2-3)}$$

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$

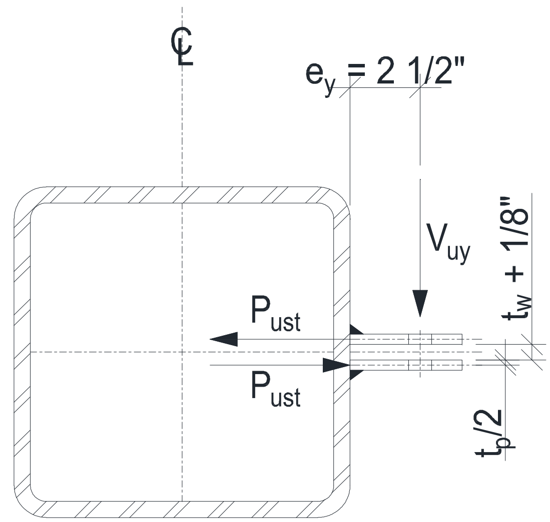
For fillet welds,

$$F_{nw} = 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5}(\theta)\right) \qquad \text{(J2-5)}$$

The length of weld is equal to the length of the shear tab. Using n to denote the number of bolts, the available strength is calculated as shown on the following page.



The weak-axis shear load is eccentric to the column face and thus produces a moment on the welds through axial forces in the shear tabs. For conservatism and ease of calculation, the axial forces in the shear tabs will be assumed to act at the centerline of each shear tab.



Based on the figure on the right, the axial force on one shear tab (and consequently on one weld) is calculated as follows:

$$\Sigma M = 0$$

$$0 = V_{uy}e_y - P_{ust}\left(\left(t_w + \frac{1}{8}in.\right) + 2\times\frac{t_p}{2}\right)$$

$$0 = V_{uy}e_y - P_{ust}\left(t_w + \frac{1}{8}in. + \frac{3}{8}in.\right)$$

$$P_{ust} = \frac{V_{uy}e_y}{t_w + 1/2\,in.}$$

where

e_y = the eccentricity of the weak-axis shear to the column wall face
= 2.50 *in.*

The resultant load on one weld is:

$$R_u = \sqrt{\left(\frac{V_{uy}}{2}\right)^2 + P_{ust}^{\ 2}} = \sqrt{\left(\frac{V_{uy}}{2}\right)^2 + \left(\frac{V_{uy}e_y}{t_w + 1/2\,in.}\right)^2}$$

Replace R_u with ϕR_n, V_{uy} with ϕV_n, and solve for ϕV_n.

$$\phi R_n = R_u$$

$$(\phi R_n)^2 = \frac{V_{uy}^{\ 2}}{4} + \frac{V_{uy}^{\ 2}e_y^{\ 2}}{(t_w + 1/2\,in.)^2} = V_{uy}^{\ 2}\left(\frac{1}{4} + \frac{e_y^{\ 2}}{(t_w + 1/2\,in.)^2}\right)$$

$$V_{uy}^{\ 2} = \frac{(\phi R_n)^2}{\frac{1}{4} + \frac{e_y^{\ 2}}{(t_w + 1/2\,in.)^2}}$$

$$\phi V_n = \frac{\phi R_n}{\sqrt{\frac{1}{4} + \frac{e_y^{\ 2}}{(t_w + 1/2\,in.)^2}}}$$

The formula for ϕV_n is dependent on the beam web thickness, t_w, and the eccentricity. Since the eccentricity remains constant, the relationship between ϕV_n and ϕR_n depends only on the beam web thickness.

$$e_y = 2.50\,in.$$

$$\phi V_n = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{{e_y}^2}{\left(t_w+\frac{1}{2}\,in.\right)^2}}} = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{(2.50\,in.)^2}{(t_w+0.500\,in.)^2}}} = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{6.25\,in.^2}{(t_w+0.500\,in.)^2}}}$$

While not immediately apparent, the resulting coefficient for ϕR_n varies nearly linearly with t_w. Therefore, to simplify the formulation, a linear approximation is developed for a range of web thicknesses that may be expected for use with this connection.

$$if\ t_w = 0.200\,in.$$

$$\phi V_n = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{6.25\,in.^2}{(0.200\,in.+0.500\,in.)^2}}} = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{6.25}{0.490}}} = 0.277\phi R_n$$

$$if\ t_w = 0.900\,in.$$

$$\phi V_n = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{6.25\,in.^2}{(0.900\,in.+0.500\,in.)^2}}} = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{6.25}{1.96}}} = 0.539\phi R_n$$

$$y = mx + b$$

$$m = \frac{\Delta y}{\Delta x} = \frac{0.539-0.277}{0.900-0.200} = 0.374$$

$$b = y - mx = 0.539 - 0.375\times 0.900 = 0.201$$

$$\therefore coeff = 0.374t_w + 0.201$$

Verify the accuracy of the simplified formula for an intermediate value of t_w. As shown below, the simplified formula is conservative within 1% accuracy.

$$if\ t_w = 0.550\,in.$$

$$\phi V_n = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{6.25\,in.^2}{\left(0.550\,in.+0.500\,in.\right)^2}}} = \frac{\phi R_n}{\sqrt{\frac{1}{4}+\frac{6.25}{1.1025}}} = 0.411\phi R_n$$

$$coeff = 0.374\times 0.550+0.201 = 0.407$$

$$error = \frac{0.407-0.411}{0.411}\times 100\% = -0.97\%$$

Therefore, the relationship between ϕV_n and ϕR_n simplifies to the following formula.

$$\phi V_n = \left(\frac{0.374}{in.}t_w + 0.201\right)\phi R_n$$

The length of the weld is:

$$n = \text{Number of Bolts} = 3$$

$$L = 2L_{ev} + \left(n-1\right)s = 2\left(1.25\,in.\right)+\left(3-1\right)\left(2.3125\,in.\right) = 7.125\,in.$$

The size of the weld is:

$$t_{we} = \frac{\sqrt{2}}{2}\cdot\frac{5}{16}in. = 0.22\,in.$$

The area of the weld is:

$$A_{we} = t_{we}L = \left(0.22\,in.\right)\left(7.125\,in.\right) = 1.56\,in.^2$$

The strength of the weld is:

$$\theta = 90^\circ$$

$$\begin{aligned} R_n &= 0.60F_{EXX}\left(1.0+0.5\sin^{1.5}\theta\right)A_{we} \\ &= 0.60\left(70\,ksi\right)\left(1.0+0.5\sin^{1.5}\left(90^\circ\right)\right)\left(1.56\,in.^2\right) \\ &= 98.28\,kips \end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ ENGLISH $\phi V_n = 0.75\left(\left(\frac{0.374}{in.}t_w + 0.201\right)98.28\,kips\right)$ $= \left(\frac{0.374}{in.}t_w + 0.201\right)73.71\,kips$ METRIC $\phi V_n = \left(\frac{0.0147}{\text{mm}}t_w + 0.201\right)327.8\,\text{kN}$	$\Omega = 2.00$ ENGLISH $\frac{V_n}{\Omega} = \frac{\left(\frac{0.374}{in.}t_w + 0.201\right)98.28\,kips}{2.00}$ $= \left(\frac{0.374}{in.}t_w + 0.201\right)49.14\,kips$ METRIC $\frac{V_n}{\Omega} = \left(\frac{0.0174}{\text{mm}}t_w + 0.201\right)218.6\,\text{kN}$

The example beam is a W12x35 with a beam web thickness, t_w, of 0.300 *in.* The available design strength or allowable strength is therefore:

LRFD	ASD
$\phi = 0.75$ $coeff = \frac{0.374}{in.}t_w + 0.201$ $= \frac{0.374}{in.} \times 0.300\,in. + 0.201$ $= 0.313$ $\phi V_n = (0.313)73.71\,kips$ $= 23.07\,kips \quad (102.6\,\text{kN})$	$\Omega = 2.00$ $coeff = \frac{0.374}{in.}t_w + 0.201$ $= \frac{0.374}{in.} \times 0.300\,in. + 0.201$ $= 0.313$ $\frac{V_n}{\Omega} = (0.313)49.14\,kips$ $= 15.38\,kips \quad (68.41\,\text{kN})$

3.3.5.2 Bolt Tensile Strength

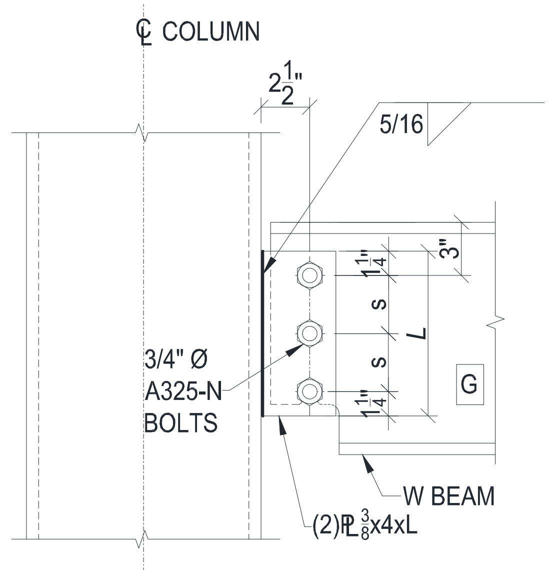
The design tensile strength, ϕR_n, and the allowable tensile strength, R_n/Ω, of snug-tightened or pretension high-strength bolt or threaded part shall be determined according to the limit state of tensile rupture as follows:

$$R_n = F_{nt} A_b \qquad \text{(J3-1)}$$

$$\phi = 0.75 \ \text{(LRFD)} \qquad \Omega = 2.00 \ \text{(ASD)}$$

Per *Specification* Table J3.2, the nominal tensile strength of A325 bolts is

$$F_{nv} = 90\ ksi$$



Using n to denote the number of bolts, the available tensile strength of ¾ in. diameter ASTM A325-N bolts is:

$$\text{Number of Fasteners, } n = 3$$

$$A_b = \frac{\pi d_b^{\,2}}{4} = \frac{\pi}{4}\left(\frac{3}{4}\,in.\right)^2 = 0.442\ in.^2$$

$$R_n = 3\left(90\,ksi\right)\left(0.442\ in.^2\right) = 119.34\ kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(119.34\ kips\right)$ $= 89.51\ kips\ \ (398.12\ \text{kN})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{119.34\ kips}{2.00}$ $= 59.67\ kips\ \ (265.41\ \text{kN})$

Note: the limit state of prying action is not considered for this connection since the shear tabs restrain deformations of the beam web in the vicinity of the bolts.

3.3.5.3 Shear Yielding and Shear Rupture in the Plates

The shear planes for shear yielding and shear rupture in the shear tabs are identical to those described in Section 3.3.1.7. Therefore, although the loading direction is different, the shear strengths are the same.

The available design strength or allowable strength for shear yielding is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(115.34\ kips)$ $= 115.34\ kips\ \ (513.03\ \text{kN})$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{115.34\ kips}{1.50}$ $= 76.89\ kips\ \ (342.00\ \text{kN})$

The available design strength or allowable strength for shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(116.92\ kips)$ $= 87.69\ kips\ \ (390.04\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{116.92\ kips}{2.00}$ $= 58.46\ kips\ \ (260.03\ \text{kN})$

Note: shear rupture is the governing limit state of the two.

3.3.5.4 Tensile Yielding and Tensile Rupture in the Plates

The tension planes for tensile yielding and tensile rupture in the shear tabs are identical to those described in Section 3.3.2.7. For the case of weak-axis shear on the connection, only one shear tab is in tension; therefore, the available design strengths and allowable design strengths will be one-half of those computed in Section 3.3.2.7.

The available design strength or allowable strength for tensile yielding is:

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = 0.90\left(192.24\ kips\right) \times \frac{1}{2}$ $= 86.51\ kips\ \left(384.79\ \text{kN}\right)$	$\frac{R_n}{\Omega} = \frac{192.24\ kips}{1.67} \times \frac{1}{2}$ $= 57.56\ kips\ \left(256.01\ \text{kN}\right)$

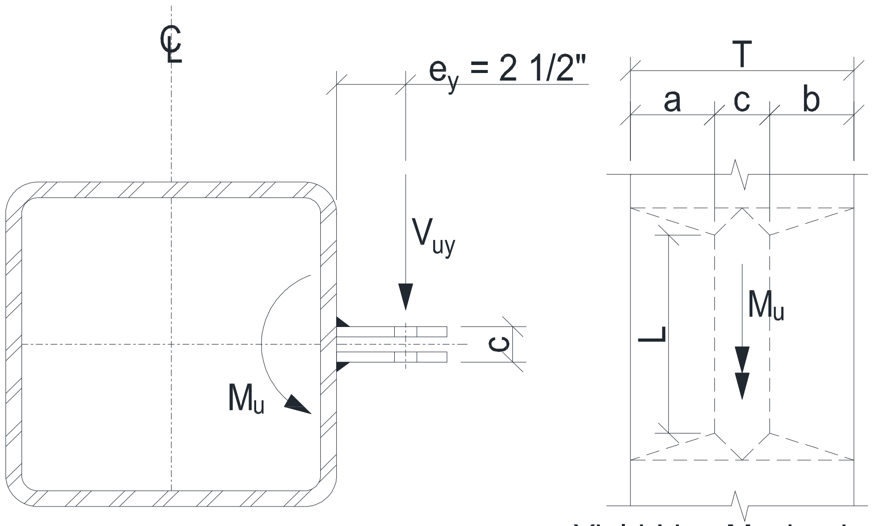
The available design strength or allowable strength for tensile rupture is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(195.46\ kips\right) \times \frac{1}{2}$ $= 73.30\ kips\ \left(326.03\ \text{kN}\right)$	$\frac{R_n}{\Omega} = \frac{195.46\ kips}{2.00} \times \frac{1}{2}$ $= 48.87\ kips\ \left(217.35\ \text{kN}\right)$

Note: tensile rupture is the governing limit state of the two.

3.3.5.5 HSS Wall Flexural Yielding

As shown in the figure below, the weak-axis shear force is applied eccentrically to the face of the HSS wall, thereby inducing localized moments in the HSS wall. The HSS wall strength will be determined in accordance with the yield-line analysis presented in *Manual* Part 9. The HSS12x12x1/2 column referenced throughout this document will be used as an example.



The flexural strength of the HSS wall for out-of-plane moments is determined using *Manual* Eq. 9-34.

$$M_n = \frac{t^2 F_y}{4}\left(\frac{4\sqrt{abcT\rho} + L\rho}{ab}\right)Q_f \qquad (9-34)$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

T = the width of the HSS wall
L = the length over which the load is delivered (i.e., the length of the shear tabs)
a = the distance from the edge of the shear tab to one edge of the HSS wall
b = the distance from the edge of the shear tab to the other edge of the HSS wall
c = the width over which the load is delivered
  $= t_w + 1/8 \textit{ in.} + 2t_p$
$\rho = 2ab + ac + bc$ (*Manual* Eq. 9-36)
Q_f = the chord-stress interaction parameter
  $= 0.487$ (see Section 3.3.2.9)

The available strength, R_n, is reduced by 50% when the concentrated force is applied at a distance from the member end that is less than $2\sqrt{Tab/(a+b)}$; this condition should be checked when the connection is located near the top of the column (e.g., at the roof) unless a cap plate is welded to the column to restrain deformations of the column walls.

The maximum moment in the HSS wall is:

$$M_u = V_{uy} e_y$$

where

e_y = the eccentricity of the weak-axis shear to the column wall face
$= 2.50\ in.$

Rearrange the formula for M_u to treat the required shear strength as an available shear strength.

$$M_n = V_{ny} e_y \quad \Rightarrow \quad R_n = \frac{M_n}{e_y}$$

Using the same web thickness, t_w, as in Section 3.3.2.9 (HSS Plastification), the limit state of flexural yielding (per square inch of wall thickness) is determined as follows:

$$T = B = 12\,in.$$
$$L = l_p = 7.125\,in.$$
$$c = \left(t_w + \frac{1}{8}in.\right) + 2t_p = 0.200\,in. + 0.125\,in. + 2(0.375\,in.)$$
$$= 1.075\,in.$$
$$a = b = \frac{T-c}{2} = \frac{12\,in. - 1.075\,in.}{2}$$
$$= 5.4625\,in.$$
$$\rho = 2ab + ac + bc = 2(5.4625\,in.)^2 + (5.4625\,in.)(1.075\,in.) \times 2$$
$$= 71.42\,in.^2$$

$$M_n = \frac{t_{des}{}^2 F_y}{4}\left(\frac{4\sqrt{abcT\rho}+L\rho}{ab}\right)Q_f \times \frac{1}{t_{des}{}^2}$$

$$= \frac{50\,ksi}{4}\left(\frac{4\sqrt{(5.4625)^2(1.075)(12)(71.42)in.^6}+(7.125)(71.42)in.^3}{(5.4625\,in.)^2}\right)(0.487)$$

$$= 239.1\frac{kip-in.}{in.^2}$$

The available shear strength is therefore:

$$R_n = \frac{M_n}{e_y}$$

$$= \frac{239.1\frac{kip-in.}{in.^2}}{2.50\,in.}$$

$$= 95.64\frac{kips}{in.^2}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = (1.00)\left(95.64\frac{kips}{in.^2}\right)$ $= 95.64\frac{kips}{in.^2} \quad \left(0.659\frac{\text{kN}}{\text{mm}^2}\right)$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{95.64\frac{kips}{in.^2}}{1.50}$ $= 63.76\frac{kips}{in.^2} \quad \left(0.440\frac{\text{kN}}{\text{mm}^2}\right)$

If the connection is located near the top of the column, the available strength, R_n, may need to be reduced by 50%. The critical distance for this case is computed as follows.

$$L_{crit} = 2\sqrt{\frac{Tab}{a+b}} = 2\sqrt{\frac{(12\,in.)(5.4625\,in.)^2}{2\times 5.4625\,in.}} = 11.45\,in.$$

If the distance from the centerline of the shear plates to the top of the column is less than L_{crit}, then the available design strength or allowable strength is reduced by 50%:

LRFD	ASD
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = (1.00)\left(95.64\dfrac{kips}{in.^2}\right)\times 0.50$ $= 47.82\dfrac{kips}{in.^2} \quad \left(0.330\dfrac{\text{kN}}{\text{mm}^2}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{95.64\dfrac{kips}{in.^2}}{1.50}\times 0.50$ $= 31.88\dfrac{kips}{in.^2} \quad \left(0.220\dfrac{\text{kN}}{\text{mm}^2}\right)$

The example column used throughout this document has a design wall thickness, t_{des}, of 0.465 *in.*; considering the connection to be located far from the top of the column (i.e., the 50% reduction in available strength does not apply), the available design strength or allowable strength is therefore:

LRFD	ASD
$t_{des} = 0.465\,in.$ $\phi R_n = \left(95.64\dfrac{kips}{in.^2}\right)\times t_{des}{}^2$ $= \left(95.64\dfrac{kips}{in.^2}\right)(0.465\,in.)^2$ $= 20.68\,kips \quad (91.98\,\text{kN})$	$t_{des} = 0.465\,in.$ $\dfrac{R_n}{\Omega} = \left(63.76\dfrac{kips}{in.^2}\right)\times t_{des}{}^2$ $= \left(63.76\dfrac{kips}{in.^2}\right)(0.465\,in.)^2$ $= 13.79\,kips \quad (61.32\,\text{kN})$

Note: as was the case for HSS plastification (Section 3.3.2.9), changes in the beam web thickness contribute very little to the available strength and the above results for flexural yielding of the HSS wall are therefore conservative for all beams accommodated by the connection.

3.4 Beam to Beam Connection

The beam to beam connection consists of two angles that are shop-bolted to the supporting beam and field-bolted to the supported beam as shown in Figure 109 and Figure 110.

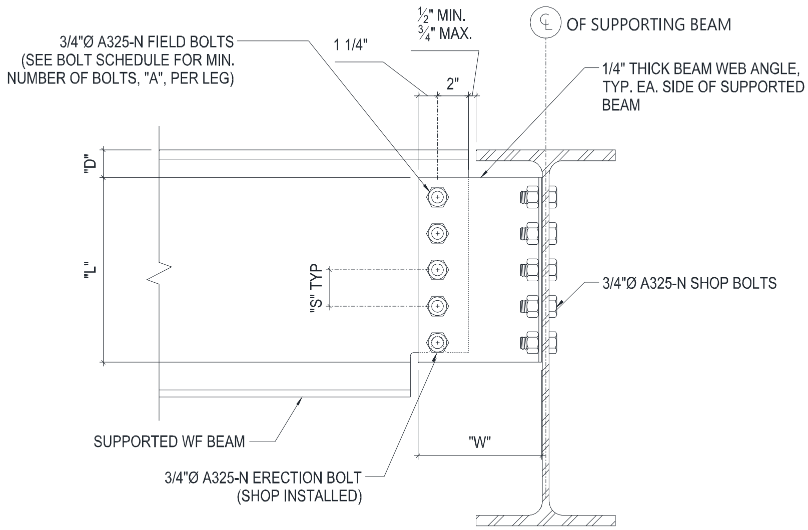


Figure 109: Elevation View of a Beam to Beam Connection.

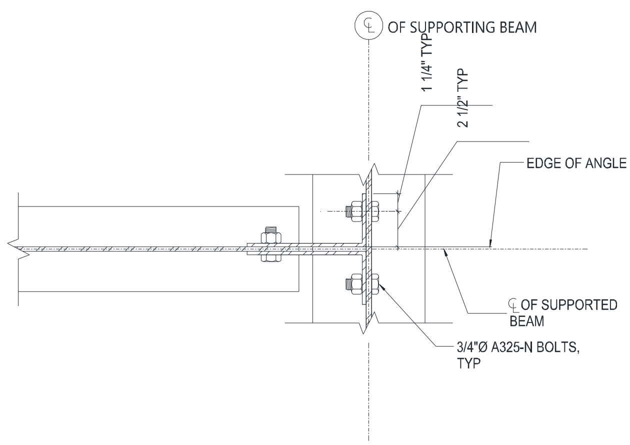


Figure 110: Plan View of a Beam to Beam Connection.

3.4.1 Connection Properties and Dimensions

The ConXtech beam-to-beam connection has a predefined set of standardized beam web angle sizes (see Figure 111).

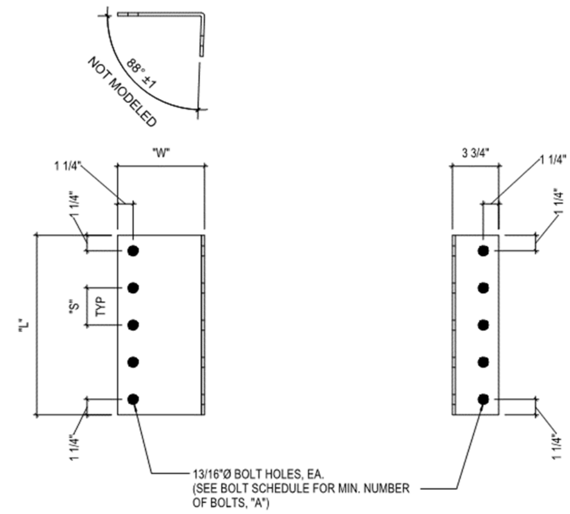


Figure 111: Elevations of Beam Web Angle

The long leg width, W, of the beam web angle is determined by locating the supporting beam flange width dimension in the left column of Table 8 and retrieving the width in the right column of the chosen row.

Table 8: Long Leg Width of Beam Web Angle (BWA)

bf of supporting beam (in)	W (in)
4	5 5/8
5 - 5 1/4	6 1/4
5 1/2 - 6	6 9/16
6 1/2	6 7/8
6 3/4 - 7 1/8	7 1/8
7 1/2 - 7 5/8	7 5/16
8 - 8 3/8	7 11/16
9 - 9 1/8	8 1/16
10 - 10 1/8	8 9/16
10 1/4 - 10 1/2	8 3/4
11 - 11 3/8	9 1/16
11 1/2 - 11 7/8	9 5/16
12 - 12 3/8	9 5/8
12 1/2 - 12 5/8	9 3/4
12 3/4 - 13	9 7/8
14 - 14 3/8	10 9/16
14 1/2 - 15 1/8	10 15/16
15 1/2 - 15 3/4	11 3/16

Table 9 contains the beam web angle length and bolt schedule for the beam to beam connections.

Table 9: Bolt Schedule

INCOMING	"L" CLIP	"A"	"S" BOLT	"D" T.O.S.
W12	7 1/8	3	2 5/16	1 3/4
W14	9 7/16	4	2 5/16	1 3/4
W16	11 3/4	5	2 5/16	1 3/4
W18	14 1/16	6	2 5/16	1 3/4
W21	14 1/16	6	2 5/16	1 3/4
W24	16 3/8	7	2 5/16	1 3/4
W27	16 3/8	7	2 5/16	1 3/4
W30	16 3/8	7	2 5/16	1 3/4
W33	16 3/8	7	2 5/16	1 3/4
W36	16 3/8	7	2 5/16	1 3/4

3.4.2 Connection Eccentricity

Typically, Table 10-1 of the *Manual* is used as a design aid for all-bolted double-angle connections. Due to the "drop-in" style connection, the bolt group in the ConXtech beam-to-beam connection is outside of the supporting beam flange and thus the eccentricity is larger than those considered in the aforementioned table. As a result, the effect of the connection eccentricity must be considered using the plastic stress distribution method in the capacity calculations as described below.

Free body diagrams similar to Figure 112 are used to determine the vertical, horizontal, and resultant forces on each bolt; the instantaneous center of rotation is assumed to occur at the center of the bolts effective at resisting horizontal loads. The eccentrically loaded bolt group generates force couples; since the dowel is considered to have no horizontal contribution, however, only the bolts above the dowel are considered in resisting the horizontal effects of connection eccentricity. Section 3.4.2.1 determines the equations for the vertical, horizontal, and resultant shear forces using the static equilibrium of the forces for a range of bolt quantities. The equations use the following variables

n = number of bolts (including the dowel) in the connection.
s = vertical spacing of bolts, in. (mm)
e = connection eccentricity, in. (mm)

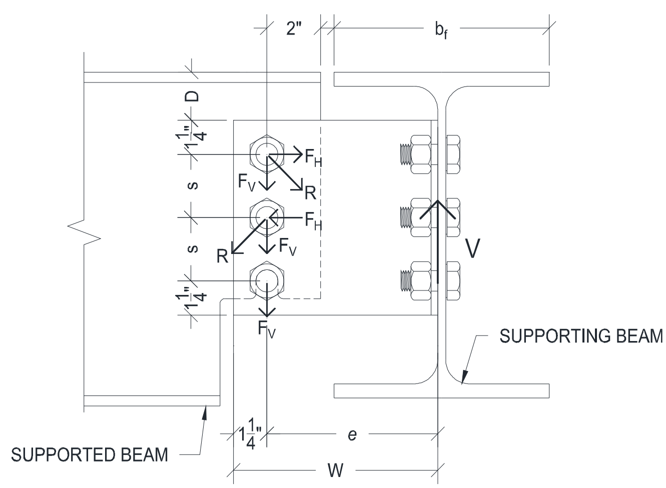
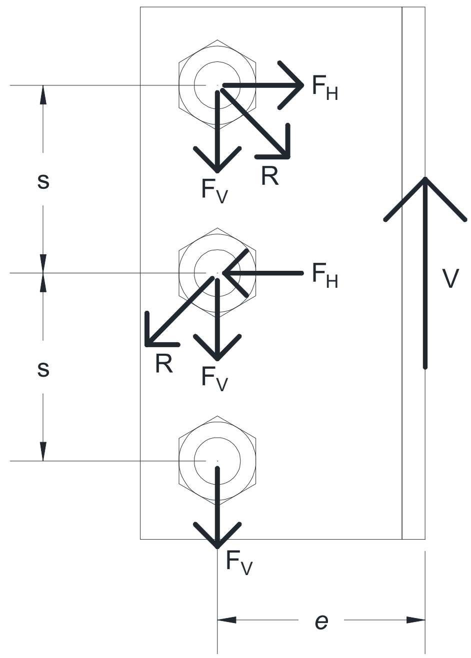


Figure 112: Diagram of the Beam Reaction V, and the Bolt Forces

3.4.2.1 Eccentricity Equations

The figure on the right shows a free body diagram of a 3-bolted double-angle connection. Static equilibrium of the double-angle is used to determine the expressions for the forces on the bolts.



Using n to denote the number of fasteners in the connection, the vertical component of the force on each bolt is determined as shown below.

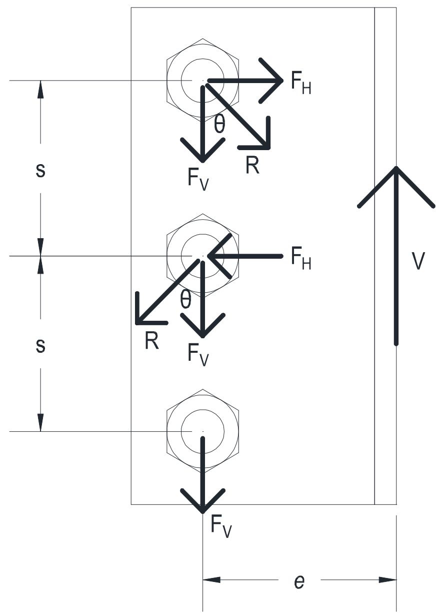
$$\sum F_y = 0 = -nF_V + V$$

$$F_V = \frac{V}{n}$$

The magnitude of the horizontal component F_H varies depending on the number of bolts as well as the eccentricity. Therefore, F_H must be determined separately for each value of n. The expressions for F_H for n = 3 to n = 7 are derived in Sections 3.4.2.1.1 through 3.4.2.1.5. In the figures that accompany the derivations, the girder and the bolts that connect to the double-angle are not shown for clarity. Furthermore, the counter-clockwise direction is taken as the positive direction for moments.

3.4.2.1.1 For n=3

The figure on the right shows a free body diagram of a **3-bolted** double-angle connection. Static equilibrium of the double-angle is used to determine the expressions for the forces on the bolts.



Using n to denote the number of fasteners in the connection, the horizontal and resultant forces on the bolts are determined as shown below.

The sum of moments about the middle fastener yields the following:

$$\sum M = 0$$

$$Ve - F_H s = 0$$

$$F_H = \frac{Ve}{s} \quad or \quad V = F_H \frac{s}{e}$$

$$(F_V)^2 + (F_H)^2 = R^2$$

$$\left(\frac{V}{n}\right)^2 + \left(\frac{Ve}{s}\right)^2 = R^2$$

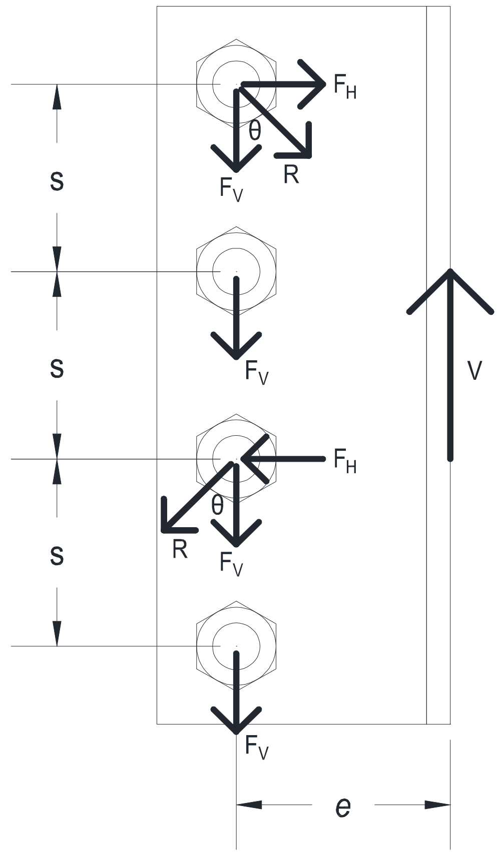
$$V = \frac{R}{\sqrt{\left(\frac{1}{n}\right)^2 + \left(\frac{e}{s}\right)^2}}$$

$$\theta = \arctan\left(\frac{F_H}{F_V}\right) = \arctan\left(\frac{\frac{Ve}{s}}{\frac{V}{n}}\right)$$

$$= \arctan\left(n\frac{e}{s}\right)$$

3.4.2.1.2 For n=4

The figure on the right shows a free body diagram of a **4-bolted** double-angle connection. Static equilibrium of the double-angle is used to determine the expressions for the forces on the bolts.



Using n to denote the number of fasteners in the connection, the horizontal and resultant forces on the bolts are determined as shown below.

The sum of moments about the second fastener from the top yields the following:

$$\sum M = 0$$

$$Ve - 2F_H s = 0$$

$$F_H = \frac{Ve}{2s} \quad or \quad V = F_H \frac{2s}{e}$$

$$(F_V)^2 + (F_H)^2 = R^2$$

$$\left(\frac{V}{n}\right)^2 + \left(\frac{Ve}{2s}\right)^2 = R^2$$

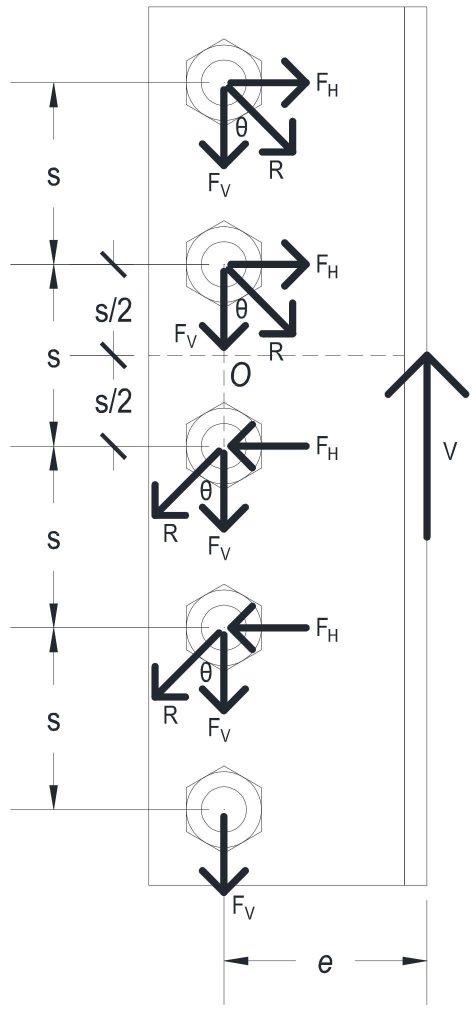
$$V = \frac{R}{\sqrt{\left(\frac{1}{n}\right)^2 + \left(\frac{e}{2s}\right)^2}}$$

$$\theta = \arctan\left(\frac{F_H}{F_V}\right) = \arctan\left(\frac{\frac{V \cdot e}{2 \cdot s}}{\frac{V}{n}}\right)$$

$$= \arctan\left(n \cdot \frac{e}{2 \cdot s}\right)$$

3.4.2.1.3 For n=5

The figure on the right shows a free body diagram of a **5-bolted** double-angle connection. Static equilibrium of the double-angle is used to determine the expressions for the forces on the bolts.



Using n to denote the number of fasteners in the connection, the horizontal and resultant forces on the bolts are determined as shown below.

The sum of moments about the point O yields the following:

$$\sum M_O = 0$$

$$Ve - 2F_H\left(\frac{s}{2}\right) - 2F_H\left(s + \frac{s}{2}\right) = 0$$

$$Ve = 2\cdot F_H\left(\frac{s}{2}\right) + 2F_H\left(s + \frac{s}{2}\right) = 4F_H s$$

$$F_H = \frac{Ve}{4s} \quad or \quad V = F_H\frac{4s}{e}$$

$$(F_V)^2 + (F_H)^2 = R^2$$

$$\left(\frac{V}{n}\right)^2 + \left(\frac{Ve}{4s}\right)^2 = R^2$$

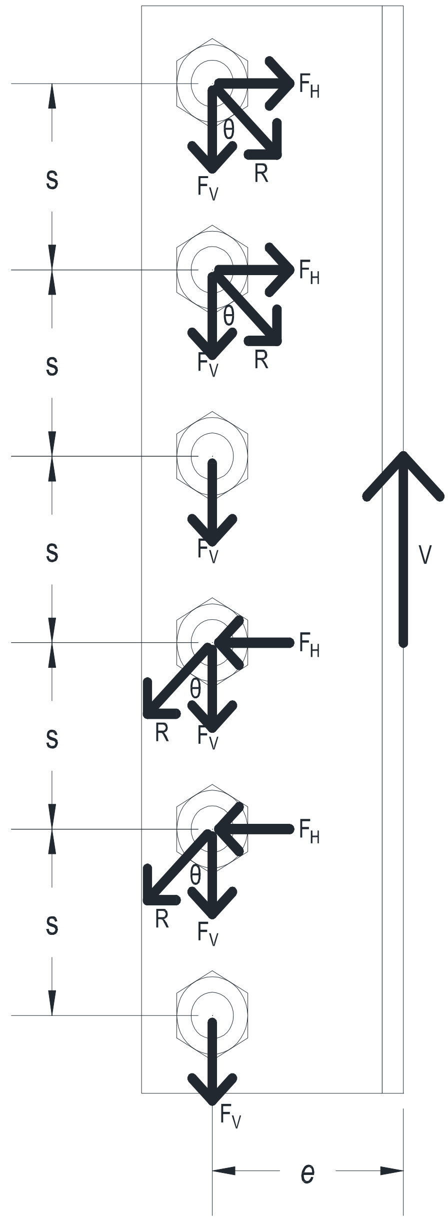
$$V = \frac{R}{\sqrt{\left(\frac{1}{n}\right)^2 + \left(\frac{e}{4s}\right)^2}}$$

$$\theta = \arctan\left(\frac{F_H}{F_V}\right) = \arctan\left(\frac{\frac{Ve}{4s}}{\frac{V}{n}}\right)$$

$$= \arctan\left(n\frac{e}{4s}\right)$$

3.4.2.1.4 For n=6

The figure on the right shows a free body diagram of a **6-bolted** double-angle connection. Static equilibrium of the double-angle is used to determine the expressions for the forces on the bolts.



Using n to denote the number of fasteners in the connection, the horizontal and resultant forces on the bolts are determined as shown below.

The sum of moments about the third fastener from the top yields the following:

$$\sum M = 0$$

$$Ve - 2F_H s - 2F_H (2s) = 0$$

$$Ve = 6F_H s$$

$$F_H = \frac{Ve}{6s} \quad or \quad V = F_H \frac{6s}{e}$$

$$(F_V)^2 + (F_H)^2 = R^2$$

$$\left(\frac{V}{n}\right)^2 + \left(\frac{Ve}{6s}\right)^2 = R^2$$

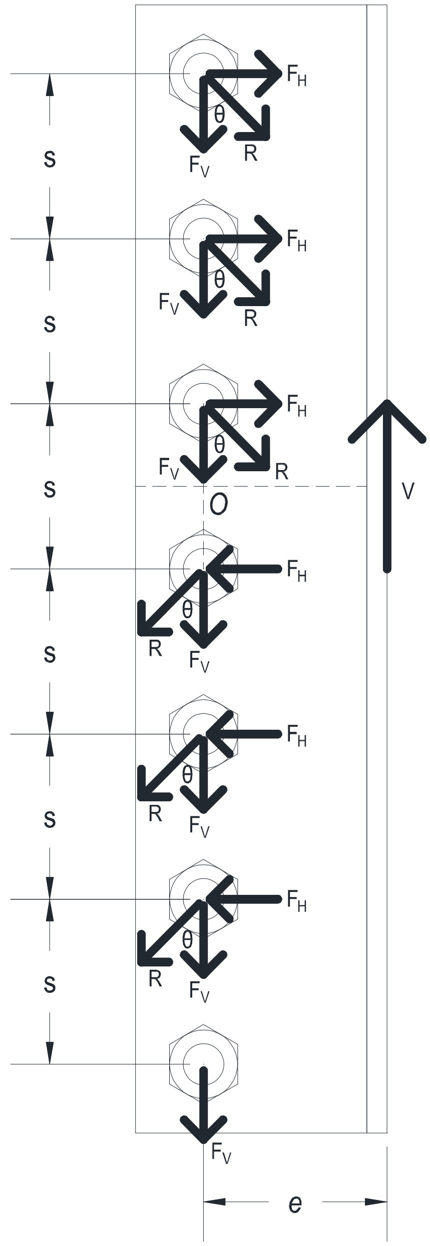
$$V = \frac{R}{\sqrt{\left(\frac{1}{n}\right)^2 + \left(\frac{e}{6s}\right)^2}}$$

$$\theta = \arctan\left(\frac{F_H}{F_V}\right) = \arctan\left(\frac{\frac{Ve}{6s}}{\frac{V}{n}}\right)$$

$$= \arctan\left(n\frac{e}{6s}\right)$$

3.4.2.1.5 For n=7

The figure on the right shows a free body diagram of a **7-bolted** double-angle connection. Static equilibrium of the double-angle is used to determine the expressions for the forces on the bolts.



Using n to denote the number of fasteners in the connection, the horizontal and resultant forces on the bolts are determined as shown below.

The sum of moments about the point O yields the following:

$$\sum M_O = 0$$

$$Ve - F_H\left(\frac{s}{2}\right)2 - F_H(1.5s)2 - F_H(2.5s)2 = 0$$

$$Ve = 9F_H s$$

$$F_H = \frac{Ve}{9s} \quad or \quad V = F_H\frac{9s}{e}$$

$$(F_V)^2 + (F_H)^2 = R^2$$

$$\left(\frac{V}{n}\right)^2 + \left(\frac{Ve}{9s}\right)^2 = R^2$$

$$V = \frac{R}{\sqrt{\left(\frac{1}{n}\right)^2 + \left(\frac{e}{9s}\right)^2}}$$

$$\theta = \arctan\left(\frac{F_H}{F_V}\right) = \arctan\left(\frac{\frac{Ve}{9s}}{\frac{V}{n}}\right)$$

$$= \arctan\left(n\frac{e}{9s}\right)$$

3.4.3 Connection Limit States

Section 3.4.4 contains example calculations to determine the available strength of the bolted/bolted double-angle connection. The available strength of the connection is determined from the limit states below:

- Bolt Shear
- Bearing at the Bolt Holes
    - In the Beam Web
    - In the Angles
- Shear Rupture on Effective Area
    - In the Beam Web
    - In the Angles
- Block Shear
    - In the Beam Web
    - In the Angles
- Shear Yielding and Shear Rupture
    - In the Beam Web
    - In the Angles
- Flexural Bending in the Beam (Reduced Tee Section)
- Flexural Bending in the Beam (Net Section)
- Flexural bending in the Double-Angle

Section 3.4.5 contains design tables of the available shear strength of the discrete number of bolted/bolted double-angle connections. The tables provide two values for the available strength based on the following limit states:

- Beam web available strength per thickness of web.
- Bolt and Double-Angle available strength for various angle thicknesses.

3.4.4 Example Calculation

The figure on the right shows a W12x26 to W12x40 beam-to-beam shear connection.

The vector arrows on the figure represent the forces on the bolts. F_H and F_V are the horizontal and vertical forces on the bolts, respectively, and R is the resultant of F_H and F_V.

The vertical force V represents the reaction on the double-angle.

Note that the formulations that follow (and all those in this example) are only valid for a 3-bolt beam-to-beam shear connection with 2 5/16in. bolt spacing.

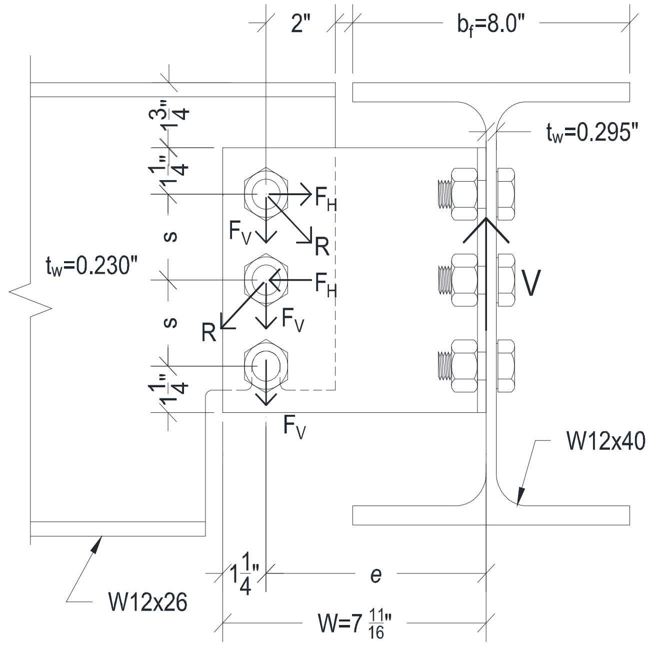


Figure 113: Beam-To-Beam Eccentric Shear Connection.

HORIZONTAL DIRECTION:

$$V \times e = F_H \times s$$

NOTE: The contribution of the dowel (lower-most bolt) to resisting horizontal forces is ignored. The dowel is only used to resist vertical forces.

VERTICAL DIRECTION:
Using n to denote the number of bolts,

$$F_V = \frac{V}{n}$$

RESULTANT FORCE:

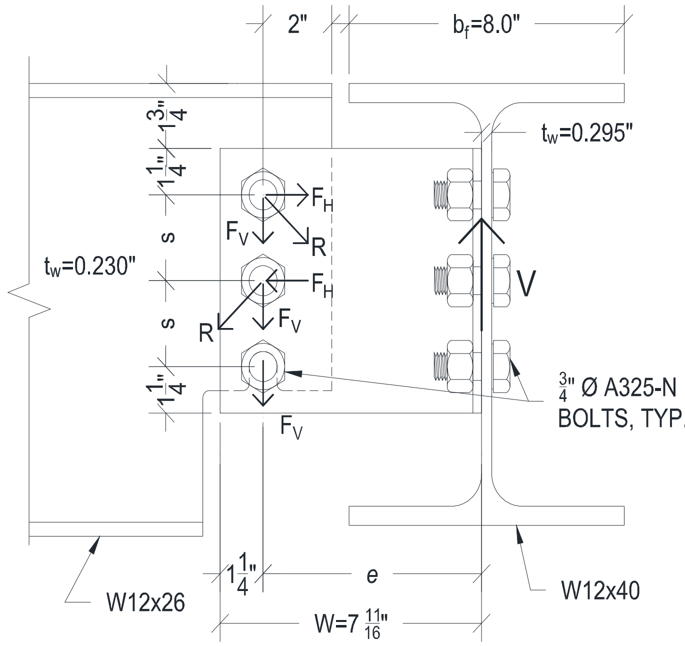
$$(F_H)^2 + (F_V)^2 = R^2$$

$$\left(V \cdot \frac{e}{s}\right)^2 + \left(\frac{V}{n}\right)^2 = R^2$$

$$V = \frac{R}{\sqrt{\left(\frac{e}{s}\right)^2 + \left(\frac{1}{n}\right)^2}} \quad \rightarrow \quad V = \frac{(\phi R_n) \text{ or } (R_n / \Omega)}{\sqrt{\left(\frac{e}{s}\right)^2 + \left(\frac{1}{n}\right)^2}}$$

3.4.4.1 Bolt Shear

The design shear strength, ϕR_n, and the allowable shear strength, R_n/Ω, of snug-tightened or pretension high-strength bolt or threaded part shall be determined according to the limit states of shear rupture as follows:



$$R_n = F_{nv} A_b \qquad (J3-1)$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

Per Table J3.2 of the *Specification*, the nominal shear strength of A325 bolts when threads are not excluded from shear planes is

$$F_{nv} = 54 \ ksi$$

The available shear strength of a single ¾" diameter ASTM A325-N bolt in double-shear is:

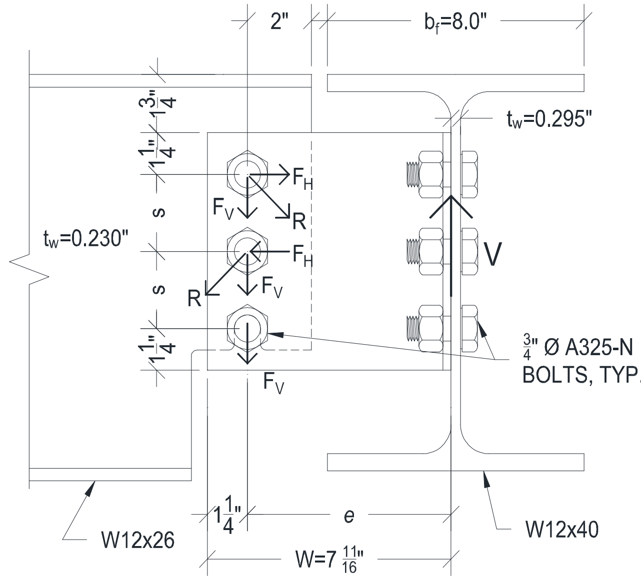
$$A_b = \frac{\pi d_b^{\,2}}{4} = \frac{\pi\left(\frac{3}{4}\,in.\right)^2}{4} = 0.442\,in.^2$$

$$R_n = (54\,ksi)\left(0.442\,in.^2\right)x2 = 47.74\,kips$$

The available design strength or allowable strength of a single bolt in double shear is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(47.74\,kips)$ $= 35.81\,kips \ (159.28\text{ kN})$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{47.74\,kips}{2.00}$ $= 23.87\,kips \ (106.17\text{ kN})$

Calculate the eccentricity e:



The detailing dimension for the beam width is 8in. Using the ConXtech beam web angle dimension schedule (Table 8 on page 267), the long leg length dimensions is 7 11/16 in.

$$
\begin{aligned}
e &= W - 1\frac{1}{4}\,in. \\
&= 7\frac{11}{16}\ in. - 1\frac{1}{4}\,in. \\
&= 6\frac{7}{16}\,in.
\end{aligned}
$$

Recall that

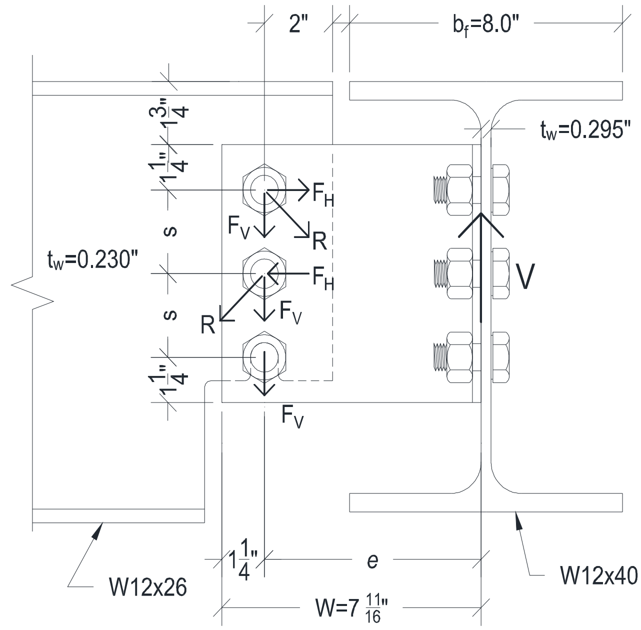
$$V = \frac{(\phi R_n)\ or\ (R_n/\Omega)}{\sqrt{\left(\frac{e}{s}\right)^2 + \left(\frac{1}{n}\right)^2}}$$

The available design strength or allowable strength is:

LRFD	ASD
$V = \frac{\phi R_n}{\sqrt{\left(\frac{e}{s}\right)^2 + \left(\frac{1}{n}\right)^2}}$ $= \frac{0.75(47.74\,kips)}{\sqrt{\left(\frac{6.4375\,in.}{2.3125\,in.}\right)^2 + \left(\frac{1}{3}\right)^2}}$ $= 12.77\,kips \quad (57.78\,\text{kN})$	$V = \frac{R_n/\Omega}{\sqrt{\left(\frac{e}{s}\right)^2 + \left(\frac{1}{n}\right)^2}}$ $= \frac{47.74\,kips/2.0}{\sqrt{\left(\frac{6.4375\,in.}{2.3125\,in.}\right)^2 + \left(\frac{1}{3}\right)^2}}$ $= 8.51\,kips \quad (37.85\,\text{kN})$

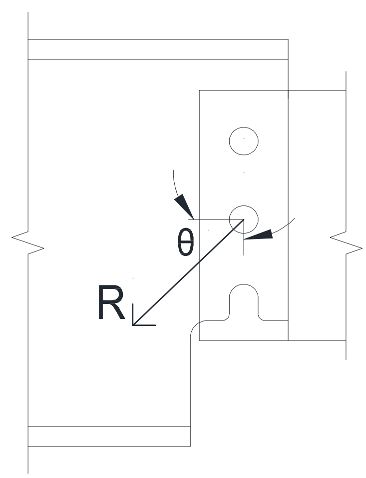
3.4.4.2 Bearing at the Bolt Holes

When the connection is subjected to eccentric loading, the available bolt bearing capacity of the connection depends on the angle of the resultant of F_H and F_V. The contribution of the dowel to the bearing capacity of the connection is ignored.



For the formulations that follow, the following definitions apply:

F_H = Horizontal Component
F_V = Vertical Component
R = Resultant of F_H and F_V
θ = Angle that the resultant load makes with respect to F_V.



As can be seen on the right, the angle of the resultant force is measured clockwise from the downward direction.

The angle that the resultant R makes with respect to F_V is determined as follows:

$$\theta = \arctan\left(\frac{F_H}{F_V}\right)$$

The resultant load is determined as follows:

$$R = \sqrt{F_H^{\ 2} + F_V^{\ 2}}$$

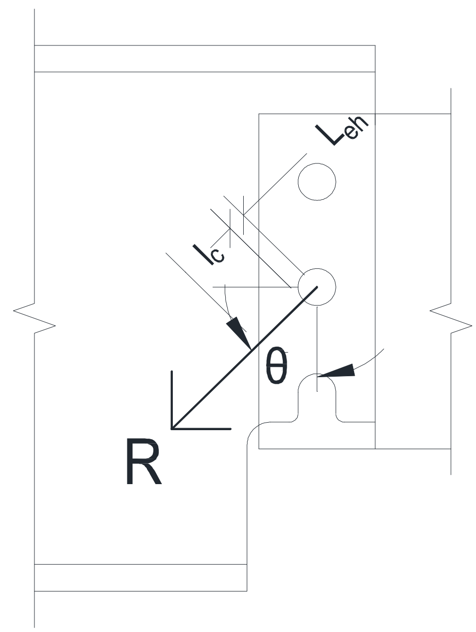
The available bearing strength at the bolt holes, in the direction of the resultant load, is determined in accordance with Chapter J of the *Specification*, as shown below.

$$R_n = 1.5 l_c t F_u \ \leq \ 3.0 d t F_u \quad \text{(J3-6b)}$$

where

l_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm).

Using n to denote the number of fasteners in the connection the value of the clear distance, l_c, is determined as follows for a given angle θ (see figure on the right).



For the upper (n-1) fasteners, the clear distance can be expressed as follows:

$$\left(r + l_c\right)\sin\left(\theta\right) = L_{eh}$$

And therefore,

$$l_c = \frac{L_{eh}}{\sin\left(\theta\right)} - r$$

where

L_{eh} = Edge distance in the horizontal direction.

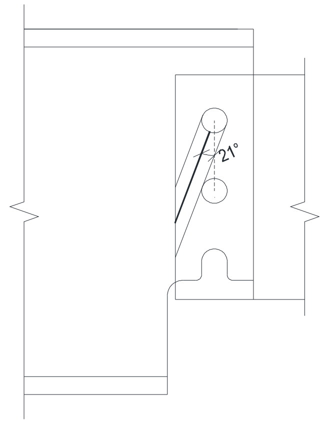
r = Radius of bolt hole.

For ¾ in. diameter ASTM A325-N high strength bolts,

$$r = \frac{d_h}{2} = \frac{13/16\,in.}{2} = \frac{13}{32}\,in.$$

As can be seen in the figure on the right, when the angle $\theta \leq 21°$, the clear distance of the upper bolt encroaches on the bolt hole below it.

For such cases, the value of the clear distance, l_c, is limited to the bolt spacing less the diameter of one hole, d_h.



The calculations below determine the angle θ and the clear distance l_c.

Determine the angle, θ:

$$\begin{aligned}\theta &= \arctan\left(\frac{e}{s} \times n\right)\\ &= \arctan\left(\frac{6.4375\,in.}{2.3125\,in.} \times 3\right)\\ &= 83.17°\end{aligned}$$

Determine the clear distance for the beam, l_c:

$$\begin{aligned}l_c &= \frac{2.0\,in.}{\sin(83.17°)} - \frac{13/16\,in.}{2}\\ &= 1.60\,in.\end{aligned}$$

Determine the clear distance for the angle, l_c:

$$\begin{aligned}l_c &= \frac{1.25\,in.}{\sin(83.17°)} - \frac{13/16\,in.}{2}\\ &= 0.85\,in.\end{aligned}$$

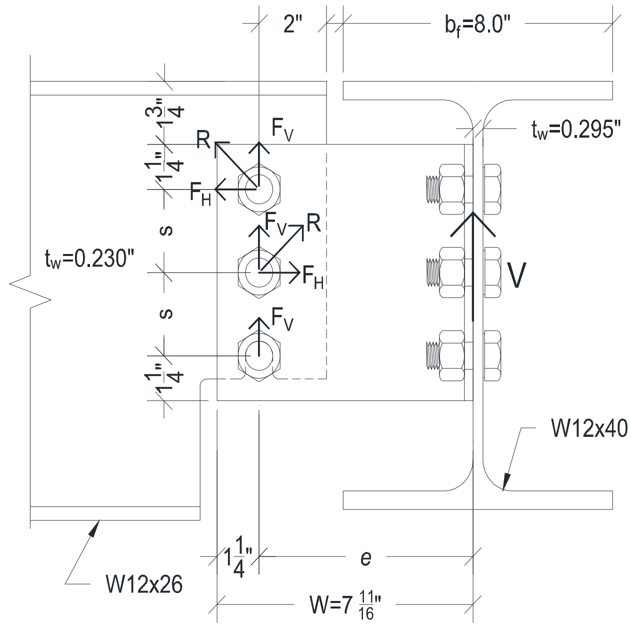
Bolt Bearing in the Beam

The available bearing strength at bolt holes shall be determined for the limit state of bearing as follows:

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force when deformation at the bolt hole at service load is not a design consideration the nominal bearing strength of the connected material is determined as follows.

$$R_n = 1.5 l_c t F_u \le 3.0 d t F_u \qquad \text{(J3-6b)}$$



NOTE: F_H bearing for the dowel is ignored, but F_V bearing is included.

The ASTM A992 beam has the following material specifications:

$$F_y = 50\ ksi$$
$$F_u = 65\ ksi$$

The capacity of a single bolt bearing in the beam is calculated as shown below:

$$l_c = 1.60\,in.$$
$$R_n = 1.5(1.60\,in.)(0.230\,in.)65\,ksi = 35.88\,kips$$
$$\le 3.0\left(\frac{3}{4}\,in.\right)(0.230\,in.)65\,ksi = 33.64\,kips \qquad \therefore R_n = 33.64\,kips$$

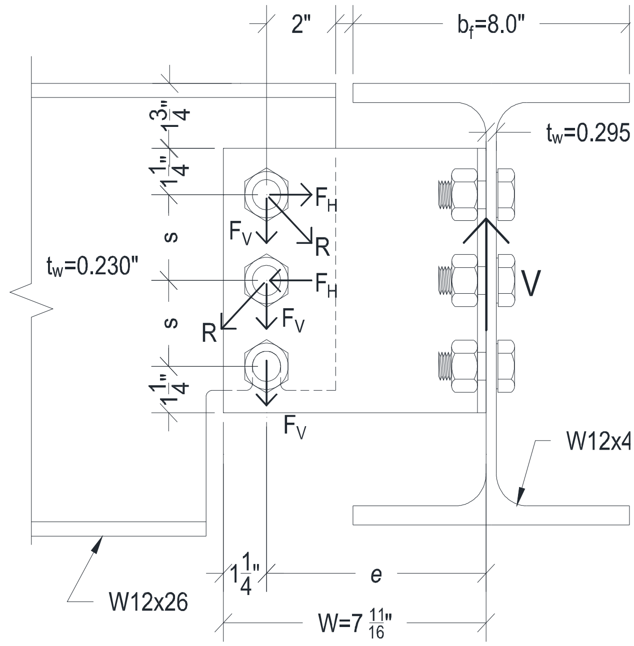
The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(33.64\ kips)$ $= 25.23\ kips\ (112.23\ \text{kN})$ $V = \dfrac{(\phi R_n)}{\sqrt{\left(\frac{e}{s}\right)^2 + \left(\frac{1}{n}\right)^2}}$ $= \dfrac{25.23\ kips}{\sqrt{\left(\frac{6.4375\ in.}{2.3125\ in.}\right)^2 + \left(\frac{1}{3}\right)^2}}$ $= 8.99\ kips\ (39.98\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{33.64\ kips}{2.00}$ $= 16.82\ kips\ (74.81\ \text{kN})$ $V = \dfrac{(R_n / \Omega)}{\sqrt{\left(\frac{e}{s}\right)^2 + \left(\frac{1}{n}\right)^2}}$ $= \dfrac{16.82\ kips}{\sqrt{\left(\frac{6.4375\ in.}{2.3125\ in.}\right)^2 + \left(\frac{1}{3}\right)^2}}$ $= 5.99\ kips\ (26.64\ \text{kN})$

NOTE: THIS IS THE GOVERNING LIMIT STATE OF THE CONNECTION.

Bolt Bearing in the Double-Angle

The available bearing strength at bolt holes shall be determined for the limit state of bearing as follows:



$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force when deformation at the bolt hole at service load is not a design consideration the nominal bearing strength of the connected material is determined as follows:

$$R_n = 1.5 l_c t F_u \le 3.0 d t F_u \qquad \text{(J3-6b)}$$

The ASTM A36 angles have the following material specifications:

$$F_y = 36\ ksi$$
$$F_u = 58\ ksi$$

The capacity of a single bolt bearing in the double angle is calculated as shown below:

$$l_c = 0.85\,in.$$

$$R_n = 1.5(0.85\,in.)\left(\frac{1}{4}\,in.\right)(58\,ksi)\,x2\ Angles = 36.97\ kips$$

$$\le 3.0\left(\frac{3}{4}\,in.\right)\left(\frac{1}{4}\,in.\right)(58\,ksi)\,x2\ Angles = 65.25\ kips \qquad \therefore R_n = 36.97\ kips$$

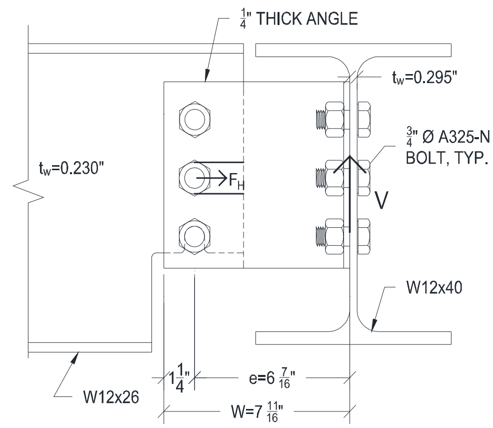
The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(36.97\ kips)$ $= 27.72\ kips\ \ (123.30\ \text{kN})$ $V = \dfrac{(\phi R_n)}{\sqrt{\left(\dfrac{e}{s}\right)^2 + \left(\dfrac{1}{n}\right)^2}}$ $= \dfrac{27.72\ kips}{\sqrt{\left(\dfrac{6.4375\ in.}{2.3125\ in.}\right)^2 + \left(\dfrac{1}{3}\right)^2}}$ $= 9.88\ kips\ \ (43.94\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{36.97\ kips}{2.00}$ $= 18.48\ kips\ \ (82.20\ \text{kN})$ $V = \dfrac{(R_n / \Omega)}{\sqrt{\left(\dfrac{e}{s}\right)^2 + \left(\dfrac{1}{n}\right)^2}}$ $= \dfrac{18.48\ kips}{\sqrt{\left(\dfrac{6.4375\ in.}{2.3125\ in.}\right)^2 + \left(\dfrac{1}{3}\right)^2}}$ $= 6.59\ kips\ \ (29.31\ \text{kN})$

3.4.4.3 Shear Rupture on Effective Area

The connection eccentricity generates vertical and horizontal component forces in the bolts. As can be seen in the figures on the right, the horizontal component, F_H, could cause shear rupture* through the effective area of either the beam or the angles.

*The limit states of shear yielding and shear rupture through the full beam and full angle depth are checked in Section 3.4.4.5.

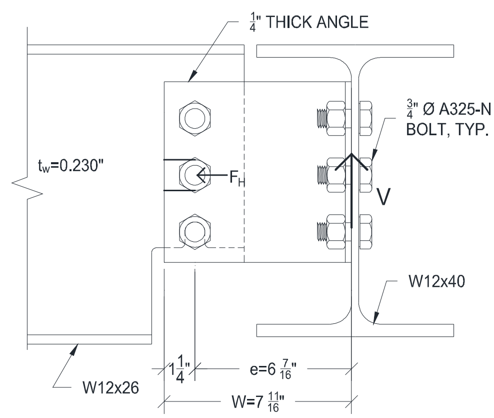


In accordance with Chapter D of the *Specification*, the design shear rupture strength, ϕP_n, and the allowable shear rupture strength, P_n/Ω, is determined as follows:

For shear rupture on the effective area:

$$P_n = 0.6\, F_u A_{sf} \qquad \text{(J4-4)}$$

$$\phi_{sf} = 0.75 \text{ (LRFD)} \qquad \Omega_{sf} = 2.00 \text{ (ASD)}$$

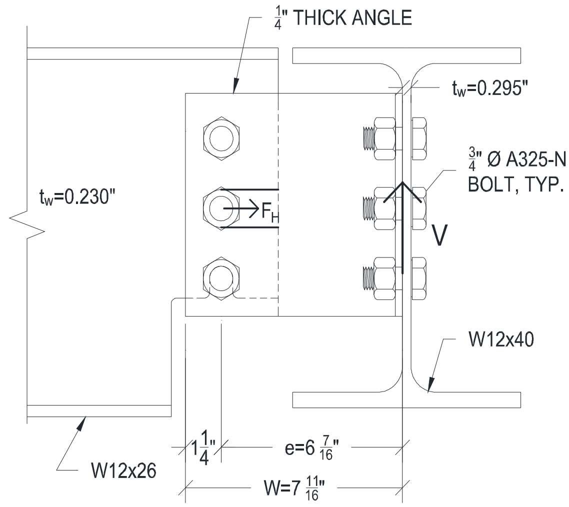


Shear Rupture in the Beam

For shear rupture on the effective area:

$$A_{sf} = 2\left(L_{eh}\right)t_{Beam}$$
$$= 2\left(2\ in.\right)\left(0.230\ in.\right) = 0.92\ in.^2$$

$$P_n = 0.6\left(65\ ksi\right)\left(0.92\ in.^2\right) = 35.88\ kips$$



Recall that $Ve = F_H s \quad \rightarrow \quad V = F_H\left(\frac{s}{e}\right) = P_n\left(\frac{s}{e}\right)$

The available design strength or allowable strength is:

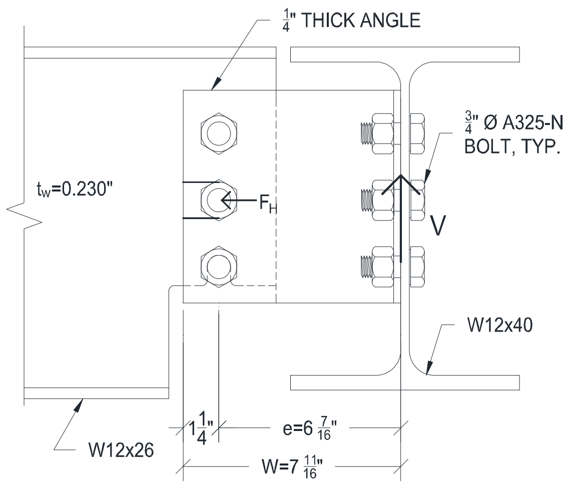
LRFD	ASD
$\phi_{sf} = 0.75$	$\Omega_{sf} = 2.00$
$\phi_{sf}P_n = 0.75\left(35.88\ kips\right)$ $= 26.91\ kips\ \left(119.7\ \text{kN}\right)$	$\frac{P_n}{\Omega_{sf}} = \frac{35.88\ kips}{2.00}$ $= 17.94\ kips\ \left(79.80\ \text{kN}\right)$
$V = \phi_{sf}P_n\left(\frac{s}{e}\right)$ $= 26.91\ kips\left(\frac{2.3125\ in.}{6.4375\ in.}\right)$ $= 9.66\ kips\ \left(42.96\ \text{kN}\right)$	$V = \frac{P_n}{\Omega_{sf}}\left(\frac{s}{e}\right)$ $= 17.94\ kips\left(\frac{2.3125\ in.}{6.4375\ in.}\right)$ $= 6.44\ kips\ \left(28.64\ \text{kN}\right)$

Shear Rupture in the Angles

For shear rupture on the effective area:

$$A_{sf} = 2L_{eh}t_{Plate} \times 2\ Angles$$
$$= 2\left(1\frac{1}{4}\ in.\right)\left(\frac{1}{4}\ in.\right) \times 2\ Angles$$
$$= 1.25\ in.^2$$

$$P_n = 0.6(58\ ksi)(1.25\ in.^2) = 43.50\ kips$$



Recall that $Ve = F_H s \quad \rightarrow \quad V = F_H\left(\frac{s}{e}\right) = P_n\left(\frac{s}{e}\right)$

The available design strength or allowable strength is:

LRFD	ASD
$\phi_{sf} = 0.75$	$\Omega_{sf} = 2.00$
$\phi_{sf}P_n = 0.75(43.50\ kips)$ $= 32.63\ kips\ \ (145.14\ \text{kN})$	$\frac{P_n}{\Omega_{sf}} = \frac{43.50\ kips}{2.00}$ $= 21.75\ kips\ \ (96.74\ \text{kN})$
$V = \phi_{sf}P_n\left(\frac{s}{e}\right)$ $= 32.63\ kips\left(\frac{2.3125\ in.}{6.4375\ in.}\right)$ $= 11.72\ kips\ \ (52.13\ \text{kN})$	$V = \frac{P_n}{\Omega_{sf}}\left(\frac{s}{e}\right)$ $= 21.75\ kips\left(\frac{2.3125\ in.}{6.4375\ in.}\right)$ $= 7.81\ kips\ \ (34.74\ \text{kN})$

3.4.4.4 Block Shear

The available strength for the limit state of block shear rupture along a shear failure path and a perpendicular tension failure path shall be taken as

$$R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(J4-5)}$$

$\phi = 0.75$ (LRFD) $\qquad \Omega = 2.00$ (ASD)

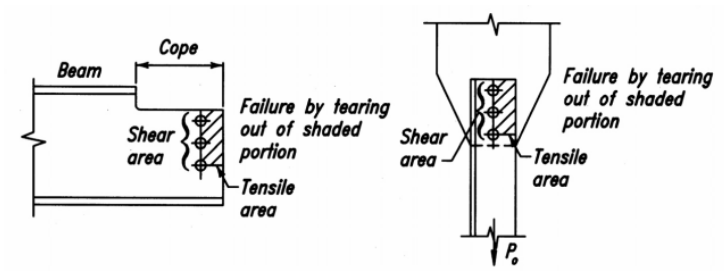


Figure 114: Failure Surfaces for Block Shear Rupture Limit State.

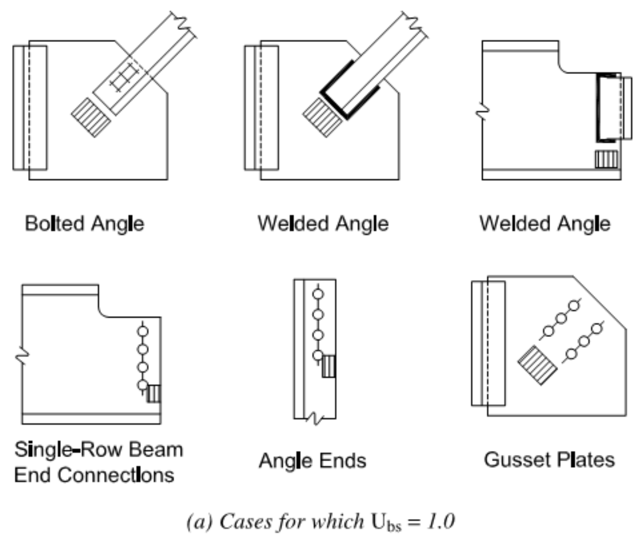
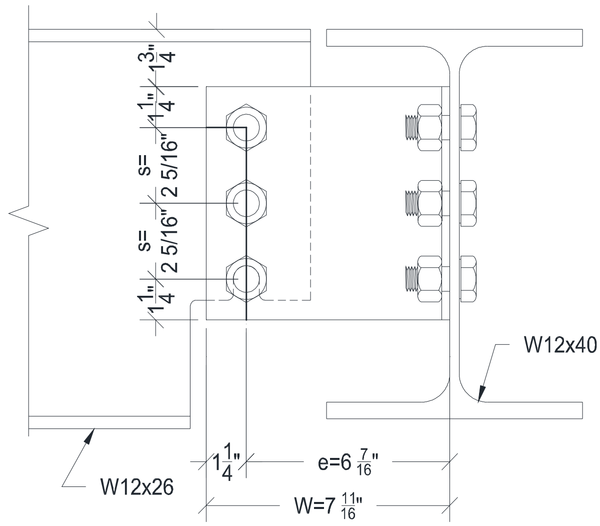


Figure 115: Block shear tensile stress distributions.

The double-angle shear connection can be categorized as a case where the tensile stress for block shear is uniform (see Figure 115). Therefore, the reduction factor $U_{bs} = 1.0$. The following calculations determine the block shear strength of the double-angle and the beam.

Block Shear Strength in the Double-Angle

Using n to denote the number of fasteners in the connection, the net shear, net tension and gross shear areas (for both angles) are calculated as follows:



$$U_{bs} = 1.0$$

$$d_{h\_net} = d_h + \frac{1}{16}\,in. + \frac{1}{16}\,in. = \frac{3}{4}\,in. + \frac{1}{16}\,in. + \frac{1}{16}\,in. = \frac{7}{8}\,in.$$

$$A_{nv} = \left(L - L_{ev} - (n-0.5)\,d_{h\_net}\right)t_{Plate} \times 2\ Angles$$

$$A_{nv} = \left(7\frac{1}{8}\,in. - 1\frac{1}{4}\,in. - 2.5 \times \frac{7}{8}\,in.\right)\left(\frac{1}{4}\,in.\right) \times 2\ Angles = 1.84\ in^2$$

$$A_{nt} = \left(L_{eh} - \frac{1}{2}d_{h\_net}\right)t_{Plate} \times 2\ Angles$$

$$= \left(1\frac{1}{4}\,in. - \frac{1}{2} \times \frac{7}{8}\,in.\right)\left(\frac{1}{4}\,in.\right) \times 2\ Angles$$

$$= 0.41\,in.^2$$

$$A_{gv} = \left(L - L_{ev}\right)t_{Plate} \times 2\ Angles$$

$$= \left(7\frac{1}{8}\,in. - 1\frac{1}{4}\,in.\right)\left(\frac{1}{4}\,in.\right) \times 2\ Angles$$

$$= 2.93\,in.^2$$

The block shear strength (for both angles) is calculated as follows:

$$R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt}$$
$$= 0.60\left(58\ ksi\right)1.84\ in.^2 + \left(1.0\right)\left(58\ ksi\right)0.41\ in.^2 = 87.81\ kips$$

$$\leq 0.60F_y A_{gv} + U_{bs} F_u A_{nt}$$
$$= 0.60\left(36\ ksi\right)2.93\ in.^2 + \left(1.0\right)\left(58\ ksi\right)0.41\ in.^2 = 87.06\ kips$$

$$\therefore R_n = 87.06\ kips$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(87.06\ kips\right)$ $= 65.29\ kips \quad \left(290.42\ \text{kN}\right)$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{87.06\ kips}{2.00}$ $= 43.53\ kips \quad \left(193.63\ \text{kN}\right)$

Block Shear Strength in the Beam

The limit state of block shear is not applicable to the beam because of the presence of the saddle on the beam cope. The failure mechanisms in the beam are shear yielding in the gross section and shear rupture in the net section. See Section 3.4.4.5 for Shear Yielding and Shear Rupture in the beam web.

3.4.4.5 Shear Yielding and Shear Rupture

The available shear strength of the affected and connecting elements in shear shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

For shear yielding of the element:

$$R_n = 0.60 F_y A_{gv} \qquad \text{(J4-3)}$$

$\phi = 1.00$ (LRFD) $\qquad \Omega = 1.50$ (ASD)

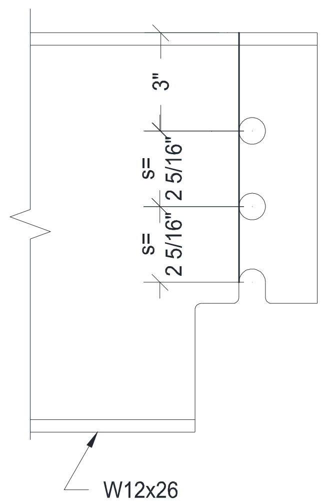
For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \qquad \text{(J4-4)}$$

$\phi = 0.75$ (LRFD) $\qquad \Omega = 2.00$ (ASD)

Shear Yielding in the Beam

In the figure below, the thickened vertical line represents the shear plane of the gross section.



For shear yielding in the gross section:
Using n to denote the number of fasteners in the connection,

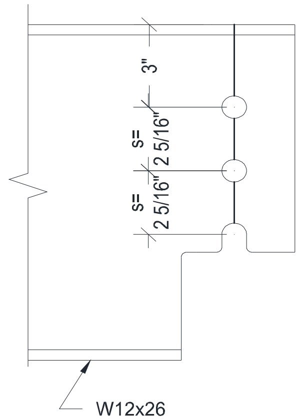
$$\begin{aligned} A_{gv} &= \left((n-1)s + 1.25\,in. + 1.5\,in.\right)t_{Beam} \\ &= \left(2\times 2.3125\,in. + 1.25\,in. + 1.75\,in.\right)(0.230\,in.) \\ &= 1.75\,in.^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_yA_{gv} \\ &= 0.60(50\,ksi)\left(1.75\,in.^2\right) \\ &= 52.5\,kips \end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(52.5\,kips)$ $= 52.5\,kips \quad (233.53\text{ kN})$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{52.5\,kips}{1.50}$ $= 35.0\,kips \quad (155.68\text{ kN})$

In the figure below, the thickened vertical line represents the shear plane of the net section.



For shear rupture in the net section:
Using n to denote the number of fasteners in the connection,

$$d_{h\_net} = d_h + \frac{1}{8}\,in. = \frac{3}{4}\,in. + \frac{1}{8}\,in. = \frac{7}{8}\,in.$$

$$\begin{aligned}
A_{nv} &= \left((n-1)2\,in. + 1.25\,in. + 1.5\,in. - (n-0.5)\,d_{h_net}\right)t_{Beam} \\
&= \left(2\times 2.3125\,in. + 1.25\,in. + 1.75\,in. - 2.5\times\frac{7}{8}\,in.\right)(0.230\,in.) \\
&= 1.25\,in.^2
\end{aligned}$$

$$\begin{aligned}
R_n &= 0.60F_uA_{nv} \\
&= 0.60(65\,ksi)(1.25\,in.^2) \\
&= 48.75\,kips
\end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(48.75\ kips)$ $= 36.56\ kips\ (162.62\ \text{kN})$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{48.75\ kips}{2.00}$ $= 24.37\ kips\ (108.42\ \text{kN})$

Note: Based on the preceding calculations, shear rupture is the governing limit state for shear in the beam web.

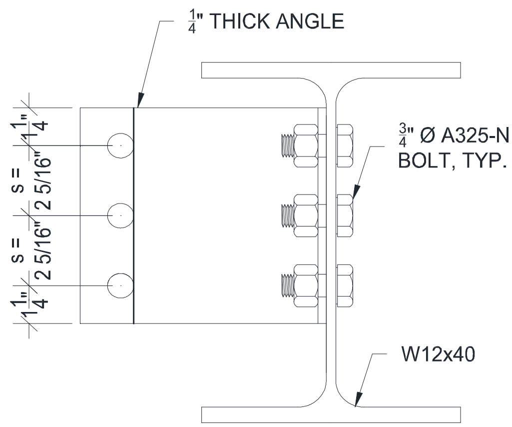
Shear Yielding in the Double-Angle

The thickened vertical line in the figure below shows the shear plane of the gross section.

For shear yielding in the gross section:

$$A_{gv} = Lt_{Plate} \times 2\ Angles$$
$$= 7.125\ in.\left(\frac{1}{4}\ in.\right) \times 2\ Angles$$
$$= 3.56\ in.^2$$

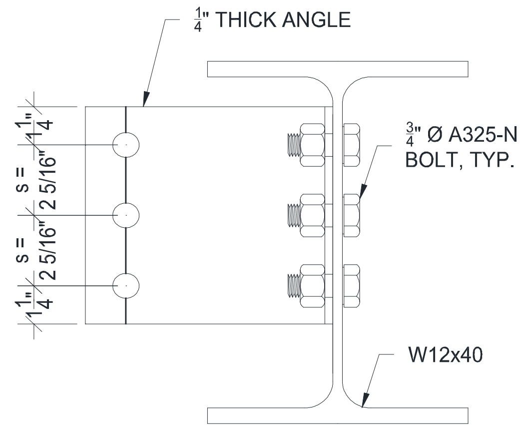
$$R_n = 0.60 F_y A_{gv}$$
$$= 0.60(36\ ksi)(3.56\ in.^2)$$
$$= 76.89\ kips$$



The available design strength or allowable strength is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(76.89\ kips)$ $= 76.89\ kips\ \ (342.02\ \text{kN})$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{76.89\ kips}{1.50}$ $= 51.26\ kips\ \ (228.01\ \text{kN})$

The thickened vertical lines in the figure on the right show the shear plane of the net section.



For shear rupture in the net section:
Using n to denote the number of fasteners in the connection,

$$d_{h\_net} = d_h + \frac{1}{8}\,in. = \frac{3}{4}\,in. + \frac{1}{8}\,in. = \frac{7}{8}\,in.$$

$$\begin{aligned} A_{nv} &= \left(L - n d_{h\_net}\right) t_{Plate} \times 2\ Angles \\ &= \left(7.125\,in. - 3\times\frac{7}{8}\,in.\right)\left(\frac{1}{4}\,in.\right)\times 2\ Angles \\ &= 2.25\,in.^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_u A_{nv} \\ &= 0.60\left(58\,ksi\right)\left(2.25\,in.^2\right) \\ &= 78.30\,kips \end{aligned}$$

The available design strength or allowable strength is:

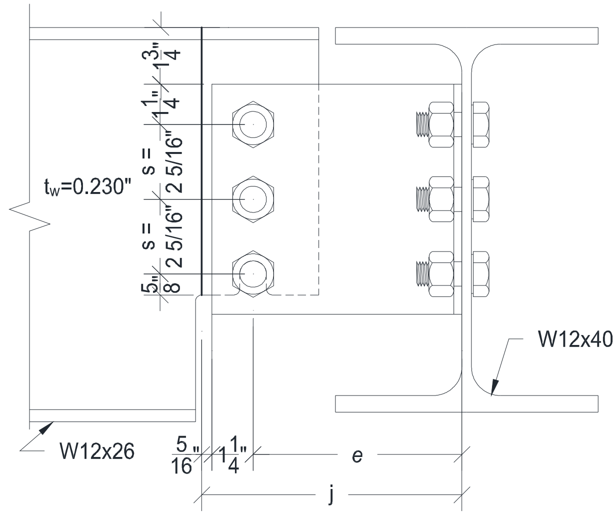
LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(78.30\,kips\right)$ $= 58.72\,kips\ \left(261.19\ \text{kN}\right)$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{78.30\,kips}{2.00}$ $= 39.15\,kips\ \left(174.14\ \text{kN}\right)$

Note: Based on the preceding calculations, shear rupture is the governing limit state for shear in the angles.

3.4.4.6 Flexural Bending in the Beam (Reduced Tee Section)

The beam in the connection experiences flexural bending due to the eccentricity of the shear connection. Flexure is checked based on the cross-section where the bottom cope begins.

Note: For the purpose of this analysis, the flange of the incoming beam is ignored in calculating the section properties of the cross-section.



Determine the lever arm j:

$$j = e + 1\frac{1}{4}\,in. + \frac{5}{16}\,in.$$

$$= 6.4375\,in. + 1\frac{1}{4}\,in. + \frac{5}{16}\,in. = 8.0\,in.$$

Determine the length of the beam flange L:

$$L = s(n-1) + 1\frac{1}{4}\,in. + 1\frac{3}{4}\,in. + \frac{5}{8}\,in.$$

$$= 2.3125\,in. \times 2 + 1\frac{1}{4}\,in. + 1\frac{3}{4} + \frac{5}{8}\,in. = 8\frac{1}{4}\,in.$$

Determine the section modulus S:

$$S = \frac{(0.23\,in.)\left(8\frac{1}{4}\,in.\right)^2}{6} = 2.60\,in.^3$$

Determine the plastic modulus Z:

$$Z = \frac{(0.23\,in.)\left(8\frac{1}{4}\,in.\right)^2}{4} = 3.91\,in.^3$$

Per Chapter F in the *Specification*, determine the nominal flexural strength M_n:

$$M_n = M_p = F_y Z \le 1.6 F_y S \qquad \text{(F11-1)}$$

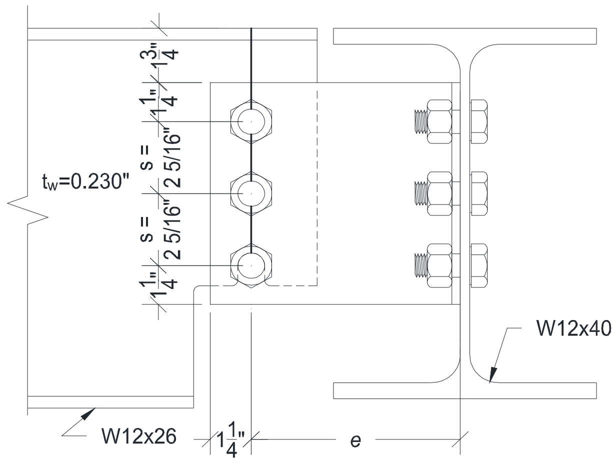
$$M_n = F_y Z = (50\ ksi)(3.91\ in.^3) = 195.50\ kip-in.$$
$$\le 1.6 F_y S = 1.6(50\ ksi)(2.60\ in.^3) = 208.00\ kip-in. \qquad \therefore M_n = 195.50\ kip-in.$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(195.50\ kip-in)$ $= 175.95\ kip-in$ $V = \dfrac{\phi M_n}{j}$ $= \dfrac{175.95\ kip-in}{8\ in.}$ $= 21.99\ kips\ (97.81\ \mathrm{kN})$	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{195.50\ kip-in}{1.67}$ $= 117.06\ kip-in$ $V = \dfrac{M_n/\Omega}{j}$ $= \dfrac{117.06\ kip-in}{8\ in.}$ $= 14.63\ kips\ (65.07\ \mathrm{kN})$

3.4.4.7 Flexural Bending in the Beam (Net Section)

The beam in the connection experiences flexural bending due to the eccentricity of the shear connection. Flexure is checked based on the cross-section through the centerline of the bolts.



Note: For the purpose of this analysis, the flange of the incoming beam is ignored in calculating the section properties of the cross-section.

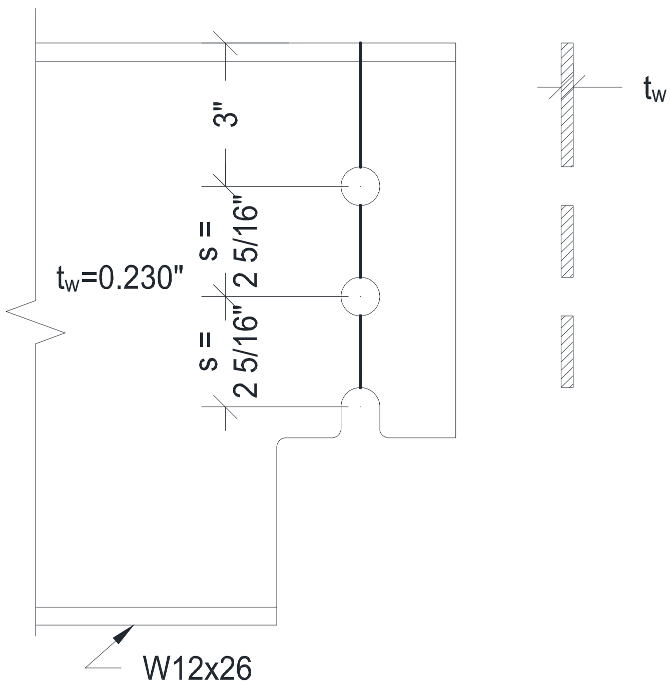
Per AISC 360-16 *Specification* Chapter F

$$M_n = M_p = F_y Z \leq 1.6 F_y S \qquad (F11-1)$$

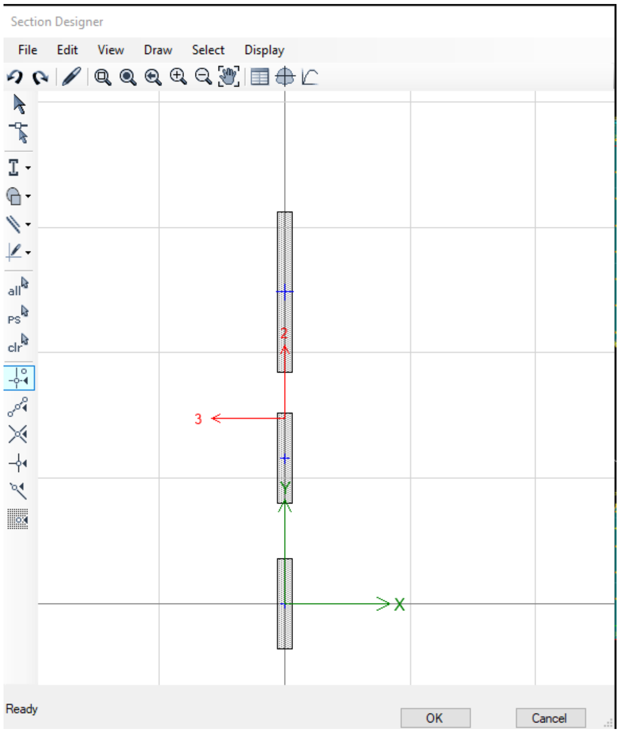
Determine the lever arm j:

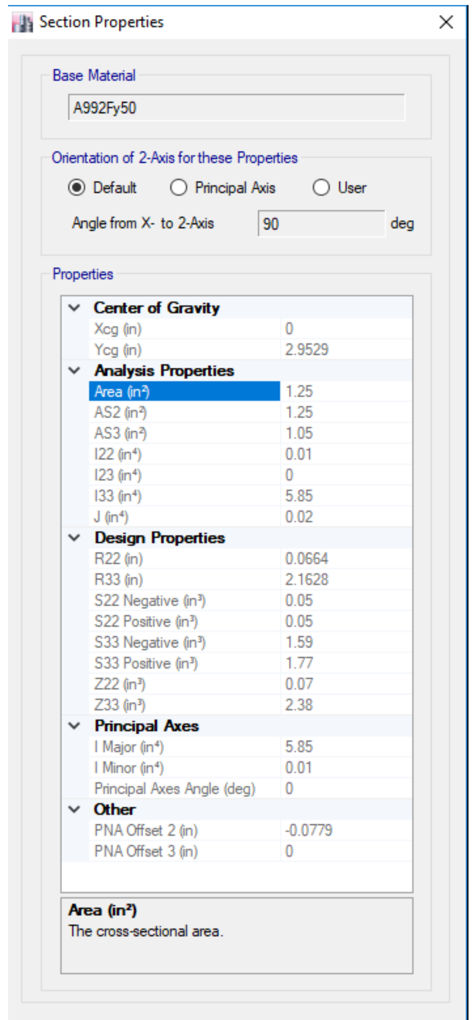
$$j = e = 6.4375\, in.$$

The figure below shows the side and cross-section view of the tee-section in the beam.



CSI ETABS' Section Designer is used to determine the cross-sectional properties of the section to be analyzed.





As can be seen in the figure on the right,

$$S^{+} = 1.77\ in.^3$$
$$Z = 2.38\ in.^3$$

Determine M_n:

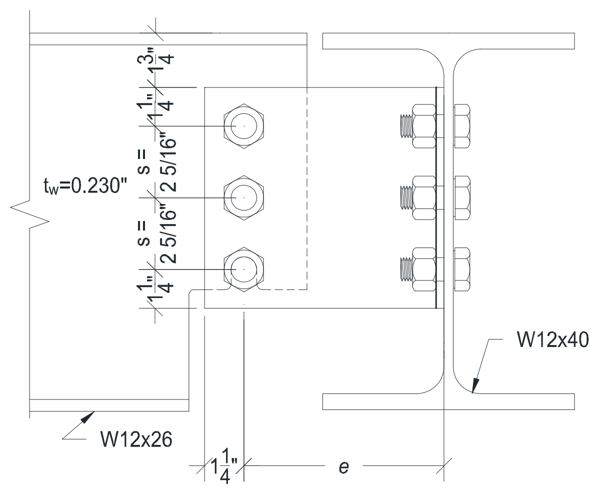
$$M_n = (50\ ksi)(2.38\ in^3) = 119\ kip-in$$
$$\leq 1.6(50\ ksi)(1.77\ in^3) = 141.6\ kip-in$$
$$\therefore M_n = 119\ kip-in$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi M_n = 0.90(119\,kip-in)$ $= 107.1\,kip-in$	$\dfrac{M_n}{\Omega} = \dfrac{119\,kip-in}{1.67}$ $= 71.25\,kip-in$
$V = \dfrac{\phi M_n}{j}$ $= \dfrac{107.1\,kip-in}{6.4375\,in.}$ $= 16.63\,kips\ (73.97\text{ kN})$	$V = \dfrac{M_n/\Omega}{j}$ $= \dfrac{71.25\,kip-in}{6.4375\,in.}$ $= 11.06\,kips\ (49.19\text{ kN})$

3.4.4.8 Flexural Bending in the Double-Angle

The double-angle in the connection experiences flexural bending due to the eccentricity of the shear connection.



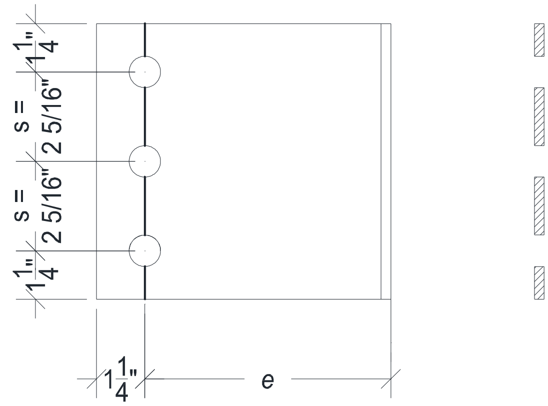
Per the *Specification* Chapter F

$$M_n = M_p = F_y Z \leq 1.6 F_y S \qquad \text{(F11-1)}$$

Determine the lever arm j:

$$j = e = 6.4375\,in.$$

The figure below shows the side and cross-section view of the tee-section in the beam.



Determine the section modulus S:

$$S=\frac{(0.25\,in.)\left((7.125in.)^3-(5.5in.)^3\right)}{6(7.125in.)}+\frac{(0.25\,in.)\left((3.75in.)^3-(0.875in.)^3\right)}{6(3.75in.)}=1.72\,in.^3$$

Determine the plastic modulus Z:

$$Z=\frac{(0.25\,in.)\left((7.125in.)^2-(5.5in.)^2+(3.75in.)^2-(0.875in.)^2\right)}{4}=2.11\,in.^3$$

Determine M_n:

$$\begin{aligned}M_n&=(36\,ksi)(2.11\,in.^3)=75.96\,kip-in.\\&\le 1.6(36\,ksi)(1.72\,in.^3)=99.07\,kip-in.\\&\therefore M_n=75.96\,kip-in.\end{aligned}$$

The available design strength or allowable strength is:

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(75.06\,kip-in)2$ $= 135.10\,kip-in$ $V = \dfrac{\phi M_n}{j}$ $= \dfrac{135.10\,kip-in}{6.4375in.}$ $= 20.98\ kips\ (93.32\ \text{kN})$	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{2(75.06\,kip-in)}{1.67}$ $= 89.89\,kip-in$ $V = \dfrac{M_n/\Omega}{j}$ $= \dfrac{89.89\,kip-in}{6.4375\,in.}$ $= 13.96\ kips\ (62.09\ \text{kN})$

The governing limit state of the connection is bolt bearing in the beam per Section 3.4.4.2. Therefore, the maximum shear that can be applied to the example connection is 8.99 kips.

3.4.5 Design Aid

The following tables report the available shear strength of bolted/bolted double-angle beam-to-beam connections that consist of seven (7) bolts or fewer in a single row.

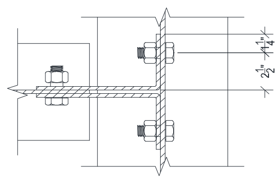
Table 10: Available Strength of Connection *(W = 5.6250 in.)*

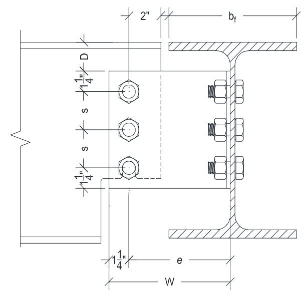
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, of 4 in.

3/4-in. A325-STD BOLTS

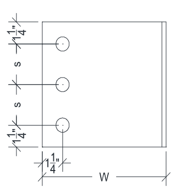
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

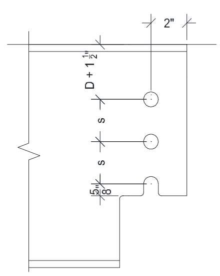
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 5 5/8 in
e = 4 3/8 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	38	57	7	10	9	14	12	18	12	18
4	9 7/16	74	112	14	22	19	29	24	36	24	36
5	11 3/4	142	213	30	45	40	60	46	69	46	69
6	14 1/16	190	285	46	69	61	92	66	100	66	100
7	16 3/8	218	327	66	100	89	133	93	140	93	140

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

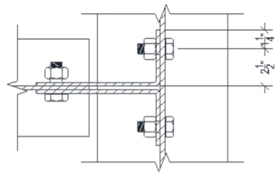
Table 11: Available Strength of Connection *(W = 6.2500 in.)*

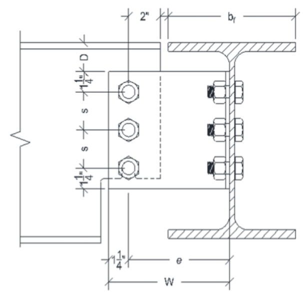
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 5 in. and 5.25 in.

3/4-in. A325-STD BOLTS

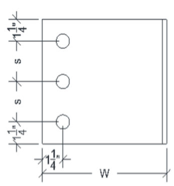
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

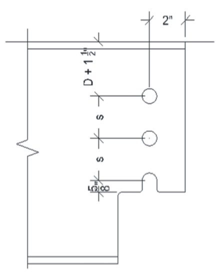
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 6 1/4 in
e = 5 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	33	50	6	9	8	12	10	16	10	16
4	9 7/16	65	98	12	19	17	25	21	32	21	32
5	11 3/4	126	190	26	39	35	52	41	62	41	62
6	14 1/16	184	276	39	59	53	79	60	90	60	90
7	16 3/8	218	327	61	91	81	122	85	127	85	127

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

CXT-SS-001
Beam to Beam Gravity Connection Spreadsheet
Rev 1.0

Release Date: 5 February 2019
Created by: Mary Nunneley
Checked by: Matthew Bosch-Willett

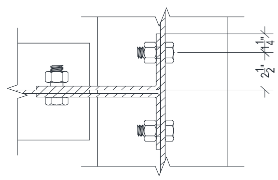
Table 12: Available Strength of Connection *(W = 6.5625 in.)*

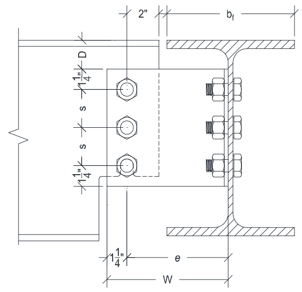
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 5.5 in. and 6 in.

$\frac{3}{4}$-in.
A325-STD BOLTS

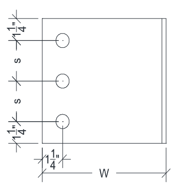
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

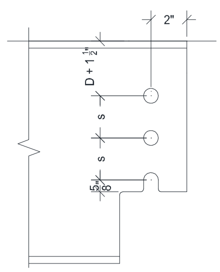
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 6 9/16 in
e = 5 5/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips — Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	31	47	6	9	8	12	10	15	10	15
4	9 7/16	62	93	12	18	16	24	20	30	20	30
5	11 3/4	120	180	24	37	32	49	39	58	39	58
6	14 1/16	174	262	37	56	49	74	57	85	57	85
7	16 3/8	218	327	57	86	76	114	81	122	81	122

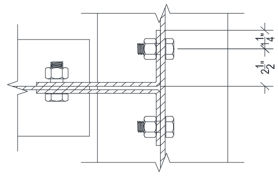
Notes:

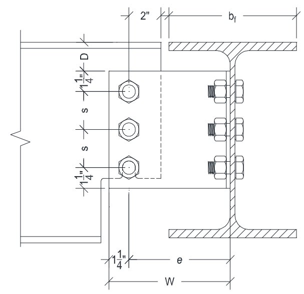
1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

Table 13: Available Strength of Connection *(W = 6.8750 in.)*

Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, of 6.5 in.

$\frac{3}{4}$-in.
A325-STD
BOLTS

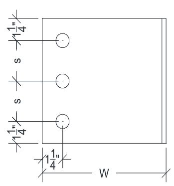


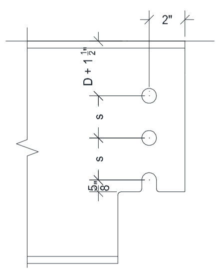


Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

W36, 33, 30, 27, 24, 21, 18, 16, 14, 12

s = 2 5/16 in
D = 1 3/4 in
W = 6 7/8 in
e = 5 5/8 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	29	44	5	8	7	11	9	14	9	14
4	9 7/16	58	88	11	17	15	22	19	28	19	28
5	11 3/4	114	171	23	34	30	46	37	55	37	55
6	14 1/16	166	250	35	53	47	70	54	81	54	81
7	16 3/8	218	327	53	80	71	107	77	116	77	116

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

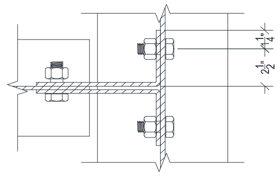
Table 14: Available Strength of Connection *(W = 7.1250 in.)*

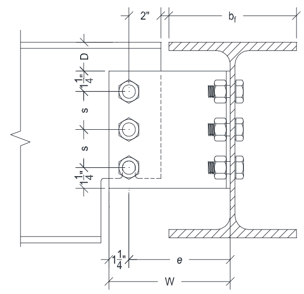
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 6.75 in. and 7.125 in.

3/4-in. A325-STD BOLTS

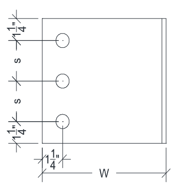
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

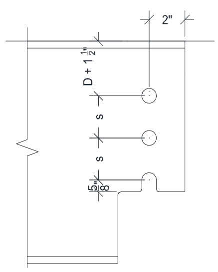
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 7 1/8 in
e = 5 7/8 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	28	42	5	8	7	10	9	13	9	13
4	9 7/16	56	84	10	16	14	21	18	27	18	27
5	11 3/4	109	164	22	33	29	44	35	53	35	53
6	14 1/16	160	241	33	50	44	67	52	78	52	78
7	16 3/8	218	327	51	76	68	102	75	113	75	113

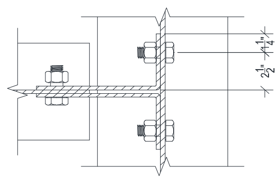
Notes:

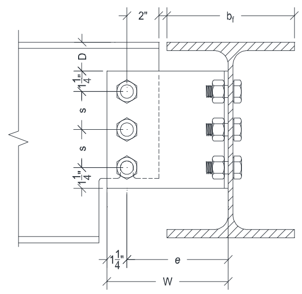
1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

Table 15: Available Strength of Connection *(W = 7.3125 in.)*

Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 7.5 in. and 7.625 in.

$\frac{3}{4}$ – in.
A325-STD
BOLTS

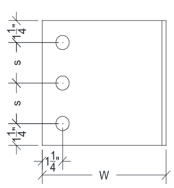


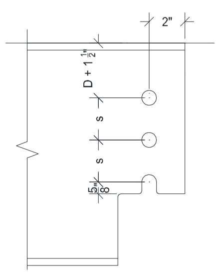


Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

W36, 33, 30, 27, 24, 21, 18, 16, 14, 12

s = 2 5/16 in
D = 1 3/4 in
W = 7 5/16 in
e = 6 1/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	27	41	5	7	7	10	8	13	9	13
4	9 7/16	54	82	10	15	14	21	17	26	17	26
5	11 3/4	106	160	21	32	28	42	34	52	34	52
6	14 1/16	156	234	32	48	43	64	50	76	50	76
7	16 3/8	218	327	49	74	66	99	73	110	73	110

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

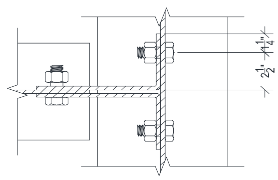
Table 16: Available Strength of Connection *(W = 7.6875 in.)*

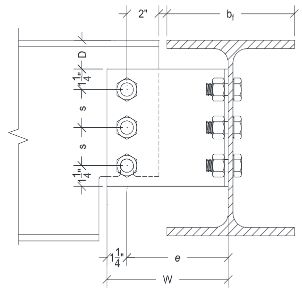
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 8 in. and 8.375 in.

$^{3}/_{4}$ – in.
A325-STD
BOLTS

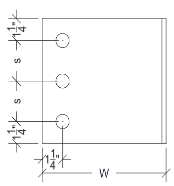
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

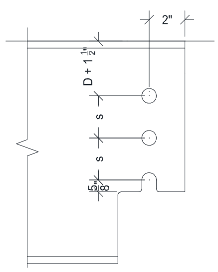
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 7 11/16 in
e = 6 7/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	26	39	4	7	6	9	8	12	8	12
4	9 7/16	51	77	9	14	13	19	16	24	16	25
5	11 3/4	101	151	20	30	26	40	32	49	32	49
6	14 1/16	148	222	30	45	40	60	48	72	48	72
7	16 3/8	214	321	46	69	61	92	69	104	69	104

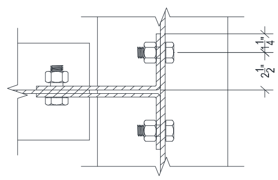
Notes:

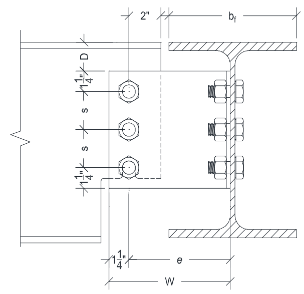
1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

Table 17: Available Strength of Connection *(W = 8.0625 in.)*

Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 9 in. and 9.125 in.

3/4-in. A325-STD BOLTS

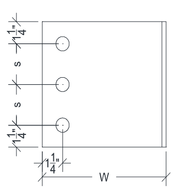


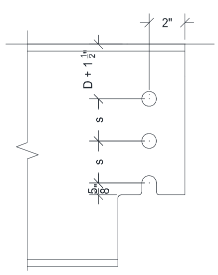


Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

W36, 33, 30, 27, 24, 21, 18, 16, 14, 12

s = 2 5/16 in
D = 1 3/4 in
W = 8 1/16 in
e = 6 13/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips — Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	24	36	4	7	6	9	7	11	8	12
4	9 7/16	48	73	9	14	12	18	15	23	15	23
5	11 3/4	95	143	19	28	25	38	31	46	31	46
6	14 1/16	140	211	28	42	38	57	45	68	45	68
7	16 3/8	204	307	43	65	58	87	66	100	66	100

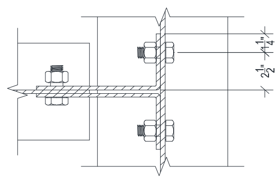
Notes:

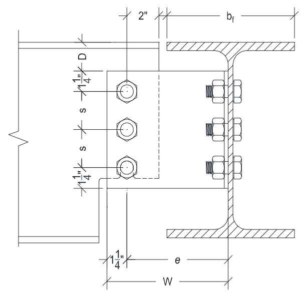
1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

Table 18: Available Strength of Connection *(W = 8.5625 in.)*

Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 10 in. and 10.125 in.

$\frac{3}{4}$-in.
A325-STD
BOLTS

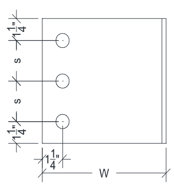


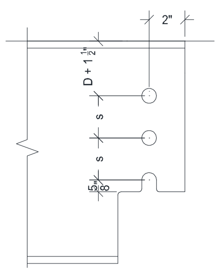


Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

W36, 33, 30, 27, 24, 21, 18, 16, 14, 12

s = 2 5/16 in
D = 1 3/4 in
W = 8 9/16 in
e = 7 5/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	23	34	4	6	5	8	7	10	7	11
4	9 7/16	45	68	8	13	11	17	14	21	14	22
5	11 3/4	89	134	17	26	23	35	29	43	29	43
6	14 1/16	132	198	26	39	35	53	43	64	43	64
7	16 3/8	192	289	40	60	53	80	62	94	62	94

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

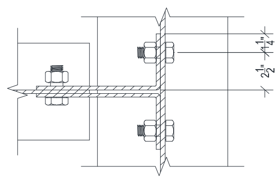
Table 19: Available Strength of Connection *(W = 8.7500 in.)*

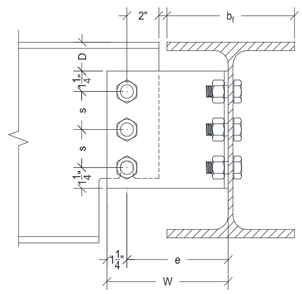
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 10.25 in. and 10.5 in.

$\frac{3}{4}$–in.
A325-STD
BOLTS

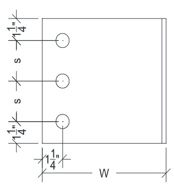
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

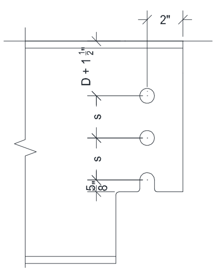
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 8 3/4 in
e = 7 1/2 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	22	33	4	6	5	8	7	10	7	10
4	9 7/16	44	66	8	12	11	17	14	21	14	21
5	11 3/4	87	131	17	25	22	34	28	42	28	42
6	14 1/16	129	193	25	38	34	51	42	63	42	63
7	16 3/8	188	282	39	59	52	79	61	92	61	92

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

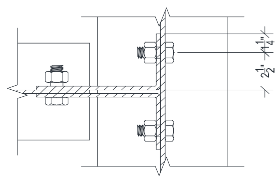
Table 20: Available Strength of Connection *(W = 9.0625 in.)*

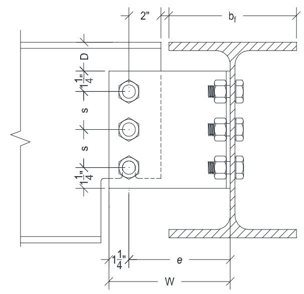
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 11 in. and 11.375 in.

3/4-in. A325-STD BOLTS

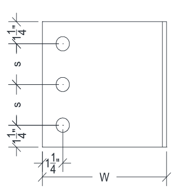
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

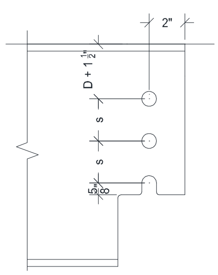
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 9 1/16 in
e = 7 13/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips — Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	21	32	4	6	5	8	6	10	7	10
4	9 7/16	42	64	8	12	10	16	13	20	13	20
5	11 3/4	84	126	16	24	22	33	27	41	27	41
6	14 1/16	124	186	25	37	33	50	40	60	40	60
7	16 3/8	181	272	37	56	50	75	59	89	59	89

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

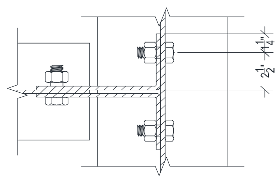
Table 21: Available Strength of Connection *(W = 9.3125 in.)*

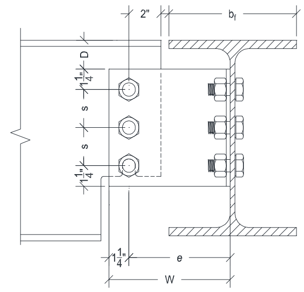
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 11.5 in. and 11.875 in.

3/4-in.
A325-STD
BOLTS

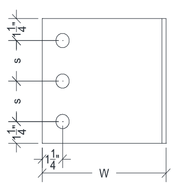
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

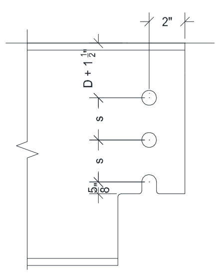
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 9 5/16 in
e = 8 1/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips (Angle Thickness, in.) 3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	20	31	3	5	5	7	6	9	6	10
4	9 7/16	41	62	7	11	10	15	13	19	13	20
5	11 3/4	81	122	16	24	21	32	26	40	26	40
6	14 1/16	120	181	24	36	32	48	39	59	39	59
7	16 3/8	177	265	36	55	48	73	57	86	57	86

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

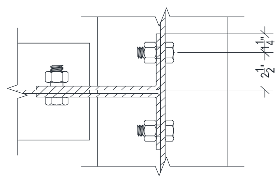
Table 22: Available Strength of Connection *(W = 9.625 in.)*

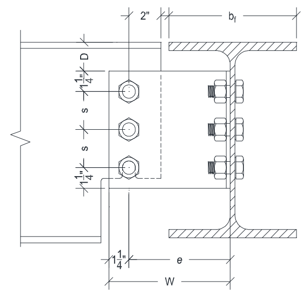
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 12 in. and 12.375 in.

3/4-in. A325-STD BOLTS

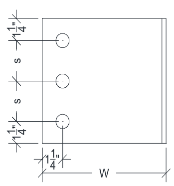
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

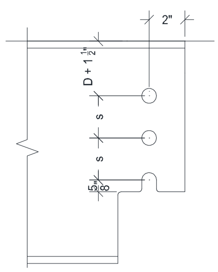
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 9 5/8 in
e = 8 3/8 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	20	30	3	5	5	7	6	9	6	9
4	9 7/16	40	60	7	11	10	15	12	19	13	19
5	11 3/4	78	118	15	22	20	30	25	38	25	38
6	14 1/16	116	175	23	34	30	46	38	57	38	57
7	16 3/8	171	256	35	52	46	70	55	83	55	83

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

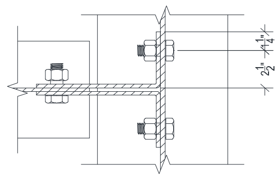
Table 23: Available Strength of Connection *(W = 9.7500 in.)*

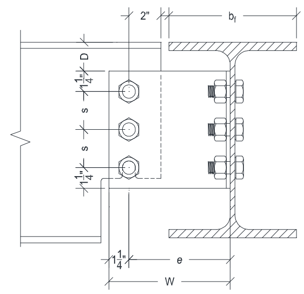
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 12.5 in. and 12.625 in.

3/4-in. A325-STD BOLTS

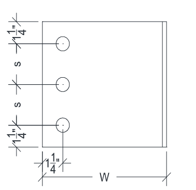
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

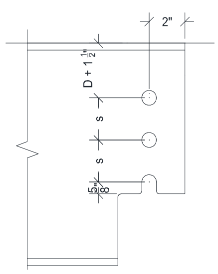
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 9 3/4 in
e = 8 1/2 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips — Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	19	29	3	5	5	7	6	9	6	9
4	9 7/16	39	59	7	11	10	15	12	18	12	19
5	11 3/4	77	116	15	22	20	30	25	37	25	38
6	14 1/16	115	172	22	34	30	45	37	56	37	56
7	16 3/8	168	253	34	52	46	69	55	82	55	82

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

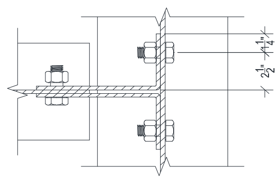
Table 24: Available Strength of Connection *(W = 9.875 in.)*

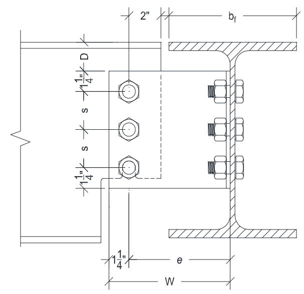
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 12.75 in. and 13 in.

3/4-in.
A325-STD
BOLTS

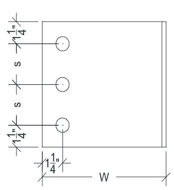
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

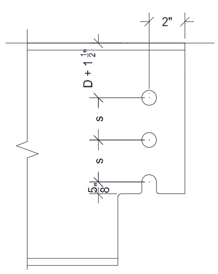
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 9 7/8 in
e = 8 5/8 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips (Angle Thickness, in.) 3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	19	29	3	5	4	7	6	9	6	9
4	9 7/16	38	58	7	11	9	14	12	18	12	19
5	11 3/4	76	114	14	22	19	29	24	37	25	37
6	14 1/16	113	170	22	33	30	45	37	55	37	55
7	16 3/8	166	250	34	51	45	68	54	81	54	81

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

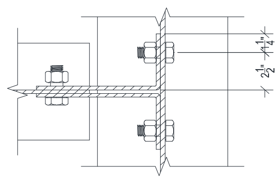
Table 25: Available Strength of Connection *(W = 10.5625 in.)*

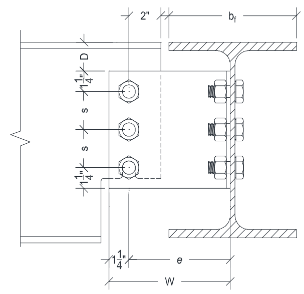
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 14 in. and 14.375 in.

3/4-in. A325-STD BOLTS

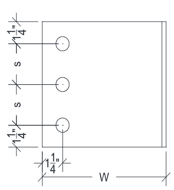
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

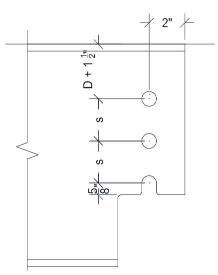
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 10 9/16 in
e = 9 5/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	18	27	3	5	4	6	5	8	5	8
4	9 7/16	36	54	6	10	9	13	11	17	11	17
5	11 3/4	71	106	13	20	18	27	23	34	23	34
6	14 1/16	105	158	20	31	27	41	34	51	34	51
7	16 3/8	155	233	31	47	42	63	50	76	50	76

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

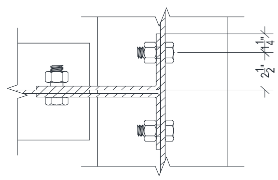
Table 26: Available Strength of Connection *(W = 10.9375 in.)*

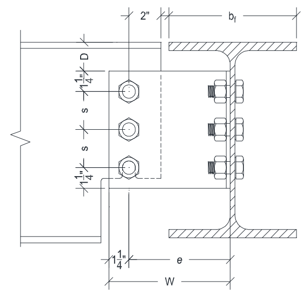
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 14.5 in. and 15.125 in.

$\frac{3}{4}$-in.
A325-STD
BOLTS

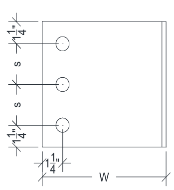
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

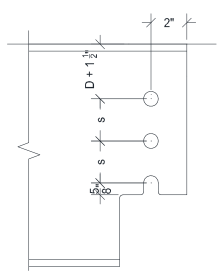
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 10 15/16 in
e = 9 11/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				$\frac{3}{16}$		$\frac{1}{4}$		$\frac{5}{16}$		$\frac{3}{8}$	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	17	26	3	4	4	6	5	8	5	8
4	9 7/16	34	52	6	9	8	13	10	16	11	16
5	11 3/4	68	102	13	19	17	26	22	33	22	33
6	14 1/16	101	152	20	29	26	39	33	49	33	49
7	16 3/8	150	225	30	45	40	60	48	73	48	73

Notes:

1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.

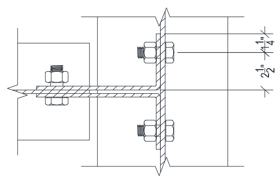
Table 27: Available Strength of Connection *(W = 11.1875 in.)*

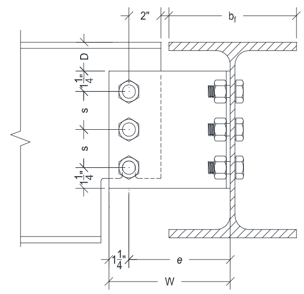
Available Strength of Bolted/Bolted Double-Angle Connection with Supporting Beam Flange Width, bf, between 15.5 in. and 15.75 in.

$\frac{3}{4}$-in. A325-STD BOLTS

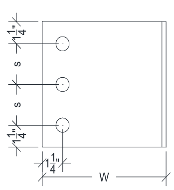
Beam	F_y =	50 ksi
	F_u =	65 ksi
Angle	F_y =	36 ksi
	F_u =	58 ksi

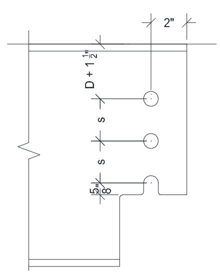
W36, 33, 30, 27, 24, 21, 18, 16, 14, 12





s = 2 5/16 in
D = 1 3/4 in
W = 11 3/16 in
e = 9 15/16 in





Number of Bolts	Length of Angle, in.	Beam Web Available Strength per Inch of Thickness, kips/in.		Bolt and Double-Angle Available Strength, kips							
				Angle Thickness, in.							
				3/16		1/4		5/16		3/8	
n	L	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	7 1/8	16	25	3	4	4	6	5	8	5	8
4	9 7/16	33	50	6	9	8	12	10	16	11	16
5	11 3/4	66	100	12	19	17	25	21	32	21	32
6	14 1/16	99	149	19	29	26	39	32	48	32	48
7	16 3/8	146	220	29	44	39	58	47	71	47	71

Notes:
1. All values have been calculated in accordance with the AISC Steel Construction Manual and the AISC 360-10 Specification. All the values tabulated above have been rounded down to the nearest whole number.
2. Limit states include bolt shear, bearing at bolt holes, shear rupture on the effective area, block shear, shear yielding, shear rupture, bending at reduced tee section, and bending of the angles.
3. Bolt shear strength governs when the angle thickness is greater than 1/4-in. All values assume Grade 36 Angles.
4. All values assume ASTM A325-N bearing type connections in STD holes with $L_{eh\text{-}angle}$ = 1.25 in., $L_{eh\text{-}beam}$ = 2.0 in., and L_{ev} = 1.25 in. For the values tabulated above, the saddle on the beam cope is treated as a long-slotted hole perpendicular to the direction of the load.
5. n = Number of bolts in connection.
6. L = Length of Angle, in.
7. s = Bolt spacing, in.