HSS

ARTICLE

DEEP DIVE INTO HSS SHEAR AND MOMENT CONNECTION EXAMPLES

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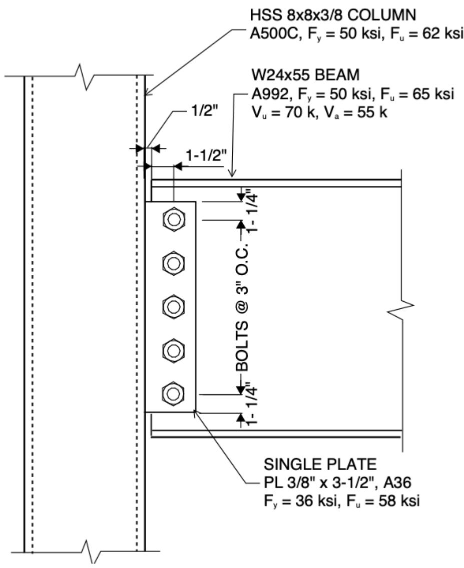
HSS is commonly utilized as a column or truss member due to its efficiency to transfer axial and torsional forces in a structure. To illustrate how to design connections to HSS members, we will dive into HSS connection examples for one common connection type and one less common connection that is difficult to find an example for in current AISC publications. Using AISC 360-2016 Specification and the 15th Edition Manual as a basis, we will step through an example for the HSS connections below. References are also made to the Steel Tube Institute's Limit State Tables that can be downloaded here. The Limit State Tables summarize HSS local limit states to consider when designing HSS connections and outline design equations in the AISC Specification and Manual with variable substitutions specific to HSS.

1. WF Beam to HSS Column Single Plate Shear Connection

2. HSS-to-HSS Connection With Shear, Axial and In-Plane Moment

WF BEAM TO HSS COLUMN SINGLE PLATE SHEAR CONNECTION EXAMPLE

The following example is a WF beam-to-HSS column single plate connection resisting shear. Design the connection per the AISC 360-16 Specification and AISC 15th Edition Manual. Loads shown are factored per LRFD load combinations.



HSS8x8x3/8 Column:	W24x55 Beam:
ASTM A500 Grade C	ASTM A992
F_y = 50 ksi	F_y = 50 ksi
F_u = 62 ksi	F_u = 65 ksi
H = 8 in	t_w = 3/8 in
B = 8 in	T = 20 ¾ in
t_{des} = 0.349 in	
A = 10.4 in^2	Bolts:
S_x = 24.9 in^3	¾" diameter A325-N
	STD holes in beam
	SSLH holes in plate

Figure 1: WF Beam-to-HSS Column Single Plate Shear Connection

Determine the capacity of the WF beam-to-HSS column single plate connection. Note there are a number of alternate shear connections that can be utilized and are discussed in this article. A through-plate shear connection is one other shear connection option in which the single plate extends through slots in the HSS column both near side and far side. However, the through-plate connection is a more costly connection to fabricate. The single plate is typically the most cost-effective connection to resist shear.

Per AISC 15th Edition Manual page 9-17, to ensure rotational ductility in the single plate connection, the geometry and thickness of the plate are configured so that the plate will yield, the bolt group will rotate, and/or the bolt holes will elongate prior to failure of welds or bolts. Per the AISC 15th Edition Manual page 10-88, check that the following dimensional limitations are met to ensure the single plate connection is consistent with a conventional configuration.

1. There is only a single row of bolts, $2 \le n \le 12$
2. The distance from the bolt line to the weld line $a = 2\ in \le 3\ 1/2\ in$ **OK**
3. Standard or short-slotted holes are used
4. $l_{ev} = 1.25\ in\ \ge 1\ in$ (from AISC 360-16 Table J3.4), $l_{eh} = 1.5\ in \ge 2d = 1.5\ in$ **OK**
5. $t_p = 0.375\ in \le d/2 + 1/16 = 0.4375\ in$ **OK**

Since all conditions are met, this connection can be designed as a conventional configuration.

We refer to the Steel Tube Institute's Limit State Tables to determine the limit states to check per AISC 360-16 Specification and the 15th Edition Manual. Figure 2 shows an excerpt from the Shear Limit State Tables Single Plate Connection.

Element		Limit State	Single Plate (Conventional and Extended Configurations)	
Element	AISC Specification and Manual References		AISC 360-10 and 14th Ed. Manual pp. 10-102 to 10-107	AISC 360-16 and 15th Ed. Manual pp. 10-87 to 10-91
	AISC Manual Considerations Specific to HSS Elements		14th Ed. Manual pp. 10-167	15th Ed. Manual pp. 10-153
	ROW NO. / COL NO.		A	B
HSS Column	16	Wall Slenderness	Manual pp. 10-167	Manual pp. 10-153
	17	HSS Shear Rupture at Weld	Manual Eq. (9-2), and DG24 First Edition Eq. (2-4)	
	18	HSS Shear Yielding at Weld	–	–
	19	Shear Yielding (Punching) of the HSS Connecting Face	Spec Eq. (K1-3) Subject to limits in Table K1.1A or Table K1.2A	Manual Eq. (10-7)

Figure 2: STI Limit State Table – Shear Single Plate (Excerpt)

1. BOLT SHEAR (LIMIT STATE TABLE CELL B2, SHEAR)

¾" A325N Bolts

ϕr_n = 17.9 k/bolt *[AISC 15th Edition Manual Table 7-1]*
a = 2 *in*, e = a/2 = 2 *in*/2 = 1 *in*
5 bolts
C = 4.77
$\phi R_n = C\phi r_n$ = 4.77(17.9) *[AISC 15th Edition Manual Table 7-6]*
= 85.4 k > 70 k **OK**

The "T" dimension of a W24x55 is 20 ¾"
AISC 15th Edition Manual page 10-7 recommends a minimum connection length of T/2
Plate vertical dimension is L_c = (2)(1¼")+(4)(3") = 14½" < 20 ¾" **OK**
L_c = 14½" > T/2 = 20.75"/2 = 10.375" **OK**

2. BOLT BEARING AND TEAROUT IN THE PLATE AND BEAM WEB (LIMIT STATE TABLE CELL B4, SHEAR)

Since the connection geometry meets the requirements for the conventional configuration, the plate bearing and tear-out checks may be calculated with the load applied as concentric per AISC 15th Edition Manual page 10-89. Consider deformation at the bolt hole at service loads. Since the plate and W24x55 beam web thickness are both 3/8 inch, the plate calculations below can also be applied to the W24 beam web.

Bearing: $R_n = 2.4dtF_u$ *[AISC 360-16 Equation J3-6a]*
$R_n = 2.4 \times 0.75\ in \times 0.375\ in \times 58\ ksi = 39.1\ k/bolt$

Tear-out: $R_n = 1.2l_ctF_u$ *[AISC 360-16 Equation J3-6c]*

At bottom bolt:
$l_c = l_{ev} - \frac{d_h}{2} = 1.25\ in - \frac{0.75\ in + 0.0625\ in}{2} = 0.844\ in$
$R_n = 1.2 \times 0.844\ in \times 0.375\ in \times 58\ ksi = 22.0\ k/bolt$

At other bolts:
$l_c = s - d_h = 3\ in - (0.75\ in + 0.0625\ in) = 2.19\ in$
$R_n = 1.2 \times 2.19\ in \times 0.375\ in \times 58\ ksi = 57.0\ k/bolt$

At the bottom bolt, tear-out controls; at the other bolts, bearing controls. Therefore:
$\phi R_n = 0.75 \times (22.0\ k/bolt + 4 \times 39.1\ \ k/bolt) = 134\ k > R_u = 70\ k$ **OK**

3. PLATE SHEAR LIMIT STATES (LIMIT STATE TABLE CELLS B5, B6 AND B10, SHEAR)

Shear Yield

$R_n = 0.60F_yA_{gv}$ *[AISC 360-16 Equation J4-3]*

$A_{gv} = 14.5\ in \times 0.375\ in = 5.44\ in^2$

$\phi R_n = 1.0 \times 0.60 \times 36\ ksi \times 5.44\ in^2 = 117\ k > R_u = 70\ k$ **OK**

Shear Rupture

$R_n = 0.60F_uA_{nv}$ *[AISC 360-16 Equation J4-4]*

$A_{nv} = 0.375 \times [14.5\ in - 5 \times (0.75\ in + 0.125\ in)] = 3.80\ in^2$

$\phi R_n = 0.75 \times 0.60 \times 58\ ksi \times 3.80\ in^2 = 99.1\ k > R_u = 70\ k$ **OK**

Block Shear Rupture

$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$ *[AISC 360-16 Equation J4-5]*

$A_{gv} = (14.5\ in - 1.25\ in) \times 0.375\ in = 4.97\ in^2$

$A_{nv} = [(14.5\ in - 1.25\ in) - 4.5\ holes \times 0.875\ in] \times 0.375\ in = 3.49\ in^2$

$A_{nt} = \left(1.5\ in - \frac{0.875\ in}{2}\right) \times 0.375\ in = 0.398\ in^2$ $U_{bs} = 1.0$

$0.60F_uA_{nv} = 0.60 \times 58\ ksi \times 3.49\ in^2 = 121\ k$

$0.60F_yA_{gv} = 0.60 \times 36\ ksi \times 4.97\ in^2 = 107\ k$ controls

$\phi R_n = 0.75(107\ k + 1.0 \times 58\ ksi \times 0.398\ in^2) = 97.8\ k > R_u = 70\ k$ **OK**

4. WELD STRENGTH (LIMIT STATE TABLE CELL B14, SHEAR)

AISC 15th Edition Manual page 10-87 offer guidance on sizing the weld between the single plate and the supporting member, our HSS column, to be 5/8t_p. This develops the strength of the plate to ensure the weld is not the critical limit state. For our 3/8 inch thick single plate:

Weld size, $w = 5/8\ t_p = 0.625 \times 0.375\ in = 0.234\ in$

Therefore, try 1/4" fillet welds, each side of plate.

Check weld shear capacity (fillet weld on each side of the plate):

$\phi R_n = (1.392\ \ k/in)\ Dl$ *[AISC 15th Edition Manual Equation 8-2a]*

$$D = \frac{70\ k}{1.392 \times 2\ sides \times 14.5\ in} = 1.73\ sixteenths$$

Therefore, 1/4" fillet weld each side is **OK** in weld shear.

5. HSS COLUMN SLENDERNESS (LIMIT STATE TABLE CELL B16, SHEAR)

Confirm the HSS8x8x3/8 column selected is not classified as a slender element per AISC 360-16 Specification Table B4.1a Case 6 and 15th Edition Manual page 10-153.

For rectangular HSS sections:

$b/t \leq 1.40\sqrt{E/F_y} = 33.7$

$b = B - 2(1.5t) = 8\ in - 2(1.5)(0.349\ in) = 6.95\ in$

$b/t = 6.95\ in/0.349\ in = 19.9 < 33.7$ **OK**

Note that the resulting b/t ratio is consistent with the AISC 15th Edition Manual Table 1-12 tabulated value for b/t for the HSS8x8x3/8 section.

6. SHEAR YIELDING (PUNCHING SHEAR) OF THE HSS WALL (LIMIT STATE TABLE CELL B19, SHEAR)

$R_ue \leq \frac{\phi F_u t l_p{}^2}{5}$ *[AISC 15th Edition Manual Equation 10-7a]*

$e = a = 2\ in$

$\phi = 0.75$

$$\phi R_n = \frac{0.75 \times 62\ ksi \times 0.349\ in \times (14.5\ in)^2}{5 \times 2\ in} = 341\ k > R_u = 70\ k\ \textbf{OK}$$

7. SHEAR RUPTURE AT WELD (LIMIT STATE TABLE CELL B17, SHEAR)

In welded joints, the AISC Specification requires that both the weld metal and base metal be checked. Specifically for HSS, we must check that the HSS wall thickness is sufficient to adequately transfer the shear along the length of the weld, which is 14.5 inches.

$R_n = 0.6F_u A_{nv}$ *[AISC 360-16 Equation J4-4]*
$\phi = 0.75$
$\phi R_n = 0.75(0.6)(62\ ksi)(2\ sides)(0.349\ in)(14.5\ in) = 282\ k > R_u = 70\ k$ **OK**

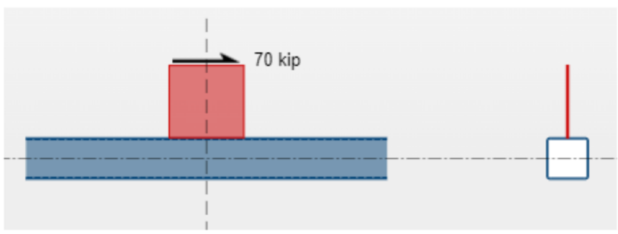
Alternately, determine the minimum HSS column thickness required to match the 1/4-inch fillet weld strength.

$t_{min} = 3.09\frac{D}{F_u} = 3.09\frac{4}{62\ ksi} = 0.2\ in < \ t_{des} = 0.349\ in$ **OK**

Therefore, the HSS8x8x3/8 column is adequate to transfer shear V_u = 70 kips at the connection.

The Steel Tube Institute's [HSS Connex tool] performs the above HSS limit state checks #5 and #6. HSS Connex does not check the plate, bolts or welds, but it can assist in ensuring the HSS wall is adequate for its local limit states. Figure 3 shows output from HSS Connex for the connection in this example.

HSS CONNEX Output Report

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++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
HSS CONNEX Version 2.0
Steel Tube Institute
++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
LIMITS OF APPLICABILITY - Plate
------------------------------------------------------------------
HSS Wall Slenderness: (B-3t)/t or (H-3t)/t of loaded wall = 19.923 <= 1.40(E/Fy)^(1/2) =33.72

TABLE FUNCTIONS - - Plate
------------------------------------------------------------------
Qf = 1, k = 0.5 in

OUTPUT SUMMARY
====================================================================

OUTPUT CALCULATION - Plate - Plate-to-Rectangular HSS Longitudinal Plate Shear Load
------------------------------------------------------------------
LS: Wall Slenderness: Rectangular b/t or Round D/t meets slenderness limits of Manual pp. 10-153.
LS: HSS Punching Shear: phi = 0.75, Rn = 454.9
Governing Shear Force Capacity: phiRn = 341.2 kip
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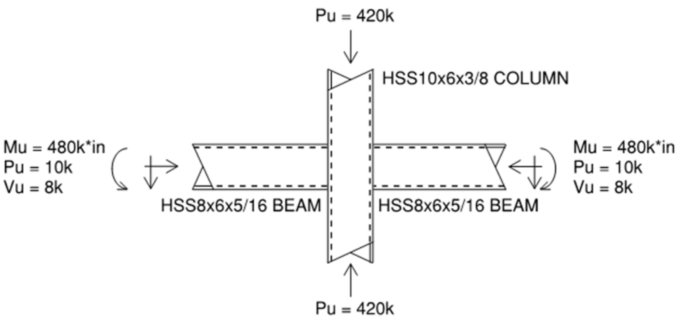
HSS Connex assumes $L_c/r < 25$ for the plate per AISC 360-16 Section J4.4

Connection OK

Figure 3: HSS Connex Output for WF Beam-to-HSS Column Shear Connection Example

HSS-TO-HSS CONNECTION WITH SHEAR, AXIAL AND IN-PLANE MOMENT EXAMPLE

With the increasing prevalence of HSS branches connecting to HSS chords in structural applications – such as trusses, girt connections, cantilevers, and exposed steel structures – finding clear examples for HSS-to-HSS connections, especially for moment transfer, can be challenging. This example illustrates an HSS column supporting a cantilever HSS beam with a back span and an axial lateral pass-through force. The connection design follows AISC 360-16 Specification and 15th Edition Manual, with factored per LRFD load combinations.



HSS10x6x3/8 Column/Chord:
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi
H = 10 in
B = 6 in
t_{des} = 0.349 in
A = 10.4 in^2
S_x = 27.4 in^3
Z_x = 33.8 in^3

HSS8x6x5/16 Branch:
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi
H_b = 8 in
B_b = 6 in
$t_{b,des}$ = 0.291 in
A_b = 7.59 in^2
$S_{b,x}$ = 17.1 in^3
$Z_{b.x}$ = 20.6 in^3

Figure 4: HSS-to-HSS Moment Connection Geometry & Loading

Based on geometry, this connection is classified as a cross-connection with an in-plane moment. The column is the through member and is considered the "chord," while the beams are considered the "branches." Since the width of the chord and branches are the same, this is a matched width connection, i.e., β = 1.0 ($B_b = B$). This can be considered a fully restrained (FR) moment connection per AISC terminology. FR moment connections are considered sufficiently rigid, which means the change in angle between members when load is applied is negligible. In contrast, partially restrained (PR) connections can transfer moment, but the change in angle is not negligible and must be considered in the frame analysis per AISC section B3.4b(b).

AXIAL FORCE LIMIT STATES

1. HSS Chord Plastification (Limit State Table cell L1, Truss)

This is checked with AISC 15th Edition Manual Equation 9-30.

$$R_n = \frac{t^2 F_y}{2}\left[\frac{(a+b)\left(4\sqrt{\frac{Tab}{a+b}}+L\right)}{ab}\right]Q_f$$

Refer to AISC 15th Edition Manual Figure 9-5(a). Since both the chord and the branch have a width of 6 inches, both a and b will equal zero, and therefore equation 9-30 is undefined. This makes sense since plastification occurs within the chord wall, but in this case, the branch has the same width. Therefore, the axial load transfers directly into the stiffer chord sidewalls instead of through the chord face, and this limit state is not applicable.

2. HSS Chord Shear Yielding (Punching) (Limit State Table Cell L2, Truss)

This is checked with AISC 15th Edition Manual equation 9-29.

$$R_n = 0.6 F_y t_p (2c_{eff} + 2L)$$

This limit state occurs when the branch punches through the connecting face of the chord as a shear failure. Similar to plastification, the chord and branch have the same width in this example, which means a shear failure through the chord cannot occur since the load will pass directly into the sidewalls. Therefore, this limit state is not applicable.

3. Local Yielding of HSS Chord Sidewalls (Limit State Table Cell L3, Truss)

$R_n = F_{yw} t_w (5k + l_b)$ *[AISC 360-16 Equation J10-2]*

$R_n = P_n \sin\theta$

$t_w = 2t_{des} = 2 \times 0.349\ in = 0.698\ in$

$k = 1.5t_{des} = 1.5 \times 0.291\ in = 0.523\ in$

$l_b = \frac{H_b}{\sin\theta} = \frac{8\ in}{\sin 90} = 8\ in$

$$\phi P_n = \frac{1.0 \times 50\ ksi \times 0.698\ in \times (5 \times 0.523\ in + 8\ in)}{\sin 90} = 370\ k > 10\ k$$

OK

4. Local Crippling of HSS Chord Sidewalls (Limit State Table Cell L4, Truss)

$R_n = 0.80 t_w{}^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$ *[AISC 360-16 Equation J10-4]*

$R_n = P_n \sin\theta$

$t_w = t_f = t_{des} = 0.349\ in$

$l_b = H_b/\sin\theta = 8\ in/\sin 90 = 8\ in$

$d = H - 3t_{des} = 10\ in - 3 \times 0.349\ in = 8.953\ in$

R_n shall be doubled for two HSS sidewalls.

$U = \left|\frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S_x}\right| = \left|\frac{420\ k}{50\ ksi \times 10.4\ in^2} + \frac{0\ k-in}{50\ ksi \times 27.4\ in^3}\right| = 0.808$ *[AISC 360-16 Equation K2-4]*

$Q_f = 1.3 - 0.4\frac{U}{\beta} = 1.3 - 0.4\frac{0.808}{1} = 0.977$ *[AISC 360-16 Equation K3-14]*

$$\phi P_n = 0.75 \times 2 \times 0.80 \times (0.349\ in)^2 \times \left[1 + 3 \times \frac{8\ in}{8.953\ in} \times \left(\frac{0.349\ in}{0.349\ in}\right)^{1.5}\right] \times \sqrt{\frac{29000\ ksi \times 50\ ksi \times 0.349\ in}{0.349\ in}} \times \frac{0.977}{\sin 90}$$

$\phi P_n = 633\ k > 10\ k$ **OK**

5. Local Buckling of HSS Chord Sidewalls (Limit State Table Cell L5, Truss)

$R_n = \left(\frac{24 t_w{}^3\sqrt{EF_{yw}}}{h}\right)Q_f$ *[AISC 360-16 Equation J10-8]*

$R_n = P_n \sin\theta$

$t_w = t_{des} = 0.349\ in$

$h = H - 3t_{des} = 10\ in - 3 \times 0.349\ in = 8.953\ in$

R_n shall be doubled for two HSS sidewalls.

$Q_f = 0.977$ per check 4 above.

$$\phi P_n = 0.90 \times 2 \times \left(\frac{24 \times (0.349\ in)^3\sqrt{29000\ ksi \times 50\ ksi}}{8.953\ in}\right) \times \frac{0.977}{\sin 90} = 241\ k > 10\ k$$

OK

6. Local Yielding of HSS Branches Due to Uneven Load Distribution (Limit State Table Cell L6, Truss)

This limit state occurs because the chord face that the branch is attaching to has higher stiffness near the chord sidewalls than at the middle of the chord face. To account for this, an effective width is used for the branch walls perpendicular to the chord. Only the effective area will be counted on as shown in figure 5.

Effective area:

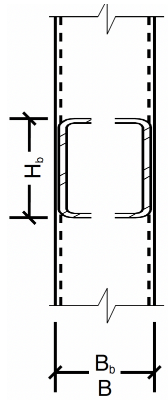


Figure 5: Branch Wall Effective Width

$R_n = F_y A_g$ [AISC 360-16 Equation J4-1]

$A_g = A_e = t_b(2H_b + 2B_e - 4t_b)$ xx

$B_e = \left(\frac{10t}{B}\right)\left(\frac{F_y t}{F_{yb} t_b}\right) B_b \leq B_b$ [AISC 360-16 Equation K1-1]

$$B_e = \left(\frac{10 \times 0.349\ in}{6\ in}\right)\left(\frac{50\ ksi \times 0.349\ in}{50\ ksi \times 0.291\ in}\right) \times 6\ in = 4.19\ in$$

$$A_g = 0.291\ in \times (2 \times 8\ in + 2 \times 4.19\ in - 4 \times 0.291\ in) = 6.75\ in^2$$

$$\phi R_n = 0.90 \times 50\ ksi \times 6.75\ in^2 = 304\ k > 10\ k$$

OK

7. Shear Yielding of HSS Chord Sidewalls (Limit State Table Cell L7, Truss)

This limit state applies only for inclined branches. Since $\theta = 90\ degrees$ in this example, there is no projected gap, and thus this limit state is not applicable.

Controlling Axial Limit State:

Local buckling of chord sidewalls: $\phi R_n = 241\ k$

MOMENT FORCE LIMIT STATES

1. Chord Wall Plastification (Limit State Table Cell F1, Moment)

This is checked with AISC 15th Edition Manual Equation 9-32.

$$M_n = \frac{t^2 F_y}{4}\left(\frac{2T}{L} + \frac{4L}{T-c} + 8\sqrt{\frac{T}{T-c}}\right) L Q_f$$

Referring to AISC 15th Edition Manual Figure 9-5(b), $T = B$ and $c = B_b$. Since $T - c = 0$ will be in the denominator, equation 9-32 is undefined. Similar to item A1 under the axial section above, the force will transfer directly into the chord sidewalls, and therefore this limit state does not apply.

2. HSS Chord Shear Yielding (Punching) (Limit State Table Cell F2, Moment)

This is another limit state that checks the connecting face of the HSS chord. Since this is a matched connection where the chord and branch widths are equal, this limit state does not apply. See item 2 in the axial section above for similar discussion.

3. Local Yielding of HSS Chord Sidewalls (Limit State Table Cell F3, Moment)

$R_n = F_{yw} t_w (5k + l_b)$ *[AISC 360-16 Equation J10-2]*

$t_w = k = t_{des}$
$l_b = H_b$
$M_n = R_n \times moment\ arm$
$moment\ arm = 0.5(H_b + 5k) = 0.5 \times (8\ in + 5 \times 0.349\ in) = 4.87\ in$

$\phi M_n = 1.0 \times 50\ ksi \times 0.349\ in \times (5 \times 0.349 + 8\ in) \times 4.87\ in = 829\ k - in > 480\ k - in$
OK

4. Local Buckling of HSS Chord Sidewalls (Limit State Table Cell F5, Moment)

Buckling of the sidewalls is a limit state that is only applicable to matched width (β=1.0) cross-connections when the moment is equilibrated by each branch of the cross-connection. The strength of the moment connection is increased by yielding in the adjacent tensile region. The hybrid compression buckling and tension-yielding behavior is accounted for with a local yielding check using a reduced yield strength of $0.8F_y$ with the same equation variable values as local yielding in moment section 3 above.

$R_n = F_{yw} t_w (5k + l_b)$ *[AISC 360-16 Equation 10-2]*

Use ϕM_n from moment check 3 above.
$\phi M_n = 0.8 \times 829\ k - in = 663\ k - in > 480\ k - in$
OK

5. Local Yielding of Branches Due to Uneven Load Distribution (Limit State Table Cell F6, Moment)

$M_n = M_p = F_y Z$ *[AISC 360-16 Equation F7-1]*
$Z = Z_{net}$

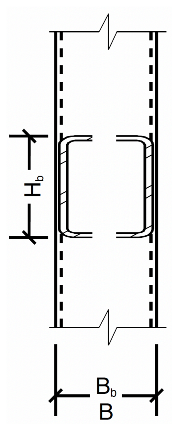


Figure 6: Branch Wall Effective Width for Plastic Section Modulus

This limit state occurs because the chord face that the branch is attaching to has higher stiffness near the chord sidewalls than at the middle of the chord face. Thus, the load is passed through the effective portions of the branch and an effective branch Z_b is used in this equation, $Z_{b,net}$ (see Figure 6). A modification and derivation of this can be found in the paper "Stepped HSS T- and Cross-Connections Under Branch In-Plane and Out-of-Plane Bending" by Dr. Jeffrey Packer, available on the STI website. The following equation can be found there:

$M_{n-ip} = F_{yb}\left[Z_b - \left(1 - \frac{B_e}{B_b}\right)\frac{B_b H_b t_b}{\sin\theta}\right]$ *[Packer, Equation 5]*

$B_e = 4.19\ in$ from Axial check 5 above.

$\phi M_{n-ip} = 0.95 \times 50\ ksi \times \left[20.6\ in^3 - \left(1 - \frac{4.19\ in}{6\ in}\right)\frac{6\ in \times 8\ in \times 0.291\ in}{\sin 90}\right] = 778\ k - in$
$\phi M_{n-ip} = 778\ k - in > 480\ k - in$
OK

CONTROLLING MOMENT LIMIT STATE

Local buckling of chord sidewalls: $\phi M_n = 663\ k - in > M_u = 480\ k - in$ **OK**

AXIAL AND MOMENT INTERACTION

$$\frac{P_u}{\phi P_n} + \frac{M_{u-ip}}{\phi M_{n-ip}} + \frac{M_{u-op}}{\phi M_{n-op}} \leq 1.0$$

[AISC 360-16 Equation K4-8]

$$\frac{10\ k}{241\ k} + \frac{480\ k - in}{663\ k - in} + 0 = 0.765 < 1.0$$

OK

Therefore, this HSS-to-HSS moment connection is adequate to transfer the shear, axial and in-plane moment demand.

(NOTE: The zero term in the equation above reflects that there is no out-of-plane moment component in this example.)

February 2023