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HSS

ARTICLE

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

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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

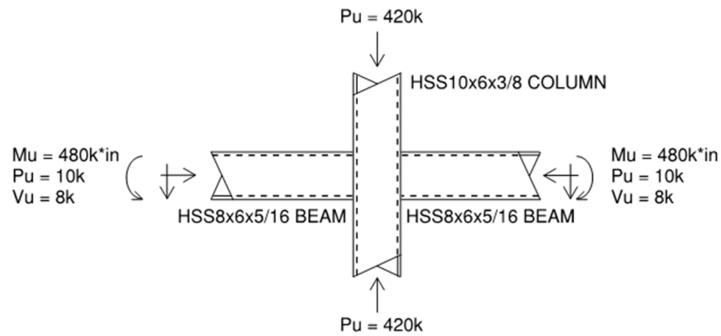

Figure 4: HSS-to-HSS Moment Connection Geometry & Loading

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.

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

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

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.

$U = \left|\frac{P_{ro}}{F_cA_g} + \frac{M_{ro}}{F_cS_x}\right| = \left|\frac{420\ k}{50\ ksi \times 10.4\ in^2} + \frac{0\ k-in}{50\ ksi \times 27.4\ in^3}\right| = 0.808$ *[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:

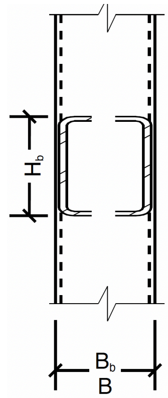

Figure 5: Branch Wall Effective Width

$R_n = F_y A_g$ *[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]*
$Z = Z_{net}$

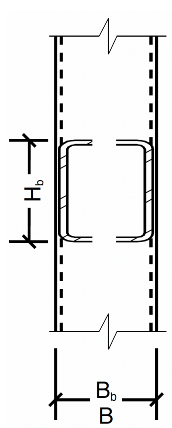

Figure 6: Branch Wall Effective Width for Plastic Section Modulus

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.)

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