The available tensile strength for the limit state of tensile rupture of the gusset plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(109 \text{ kips})$ $= 81.8 \text{ kips}$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{109 \text{ kips}}{2.00}$ $= 54.5 \text{ kips}$

Block shear rupture of the gusset plate

From AISC *Specification* Section J4.3, the nominal strength for the limit state of block shear rupture of the gusset plate is:

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (Spec.\ \text{Eq. J4-5})$$

where

$$F_u = F_{up}$$

$$F_y = F_{yp}$$

$$U_{bs} = 1 \qquad \text{(uniform stress)}$$

$$\begin{aligned} A_{gv} &= 2(1.50 \text{ in.} + 3.00 \text{ in.})(0.375 \text{ in.}) \\ &= 3.38 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - 2\left[\text{deduction for } (1 + \tfrac{1}{2}) \text{ bolt holes}\right] t_p \\ &= 3.38 \text{ in.}^2 - 2(1.5)(\tfrac{7}{8} \text{ in.} + \tfrac{1}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})(0.375 \text{ in.}) \\ &= 2.26 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[3.00 \text{ in.} - 2(0.5)(\tfrac{7}{8} \text{ in.} + \tfrac{1}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](0.375 \text{ in.}) \\ &= 0.750 \text{ in.}^2 \end{aligned}$$

The left side of the inequality given in AISC *Specification* Equation J4-5 is:

$$\begin{aligned} 0.6F_u A_{nv} + U_{bs} F_u A_{nt} &= 0.6(58 \text{ ksi})(2.26 \text{ in.}^2) + 1.00(58 \text{ ksi})(0.750 \text{ in.}^2) \\ &= 122 \text{ kips} \end{aligned}$$

The right side of the inequality given in AISC *Specification* Equation J4-5 is:

$$\begin{aligned} 0.6F_y A_{gv} + U_{bs} F_u A_{nt} &= 0.6(36 \text{ ksi})(3.38 \text{ in.}^2) + 1.00(58 \text{ ksi})(0.750 \text{ in.}^2) \\ &= 117 \text{ kips} \end{aligned}$$

Because 122 > 117 kips, use $R_n = 117$ kips. The available strength of the gusset plate for the limit state of block shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(117 \text{ kips})$ $= 87.8 \text{ kips}$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{117 \text{ kips}}{2.00}$ $= 58.5 \text{ kips}$

Shear of the weld metal and shear of the HSS and gusset plate base metals

Because the HSS slot is 1⁄16 in. wider than the gusset thickness, the gap for either weld will not exceed 1⁄16 in., and no adjustment of the weld size is required. Therefore, the effective weld size is the full weld: $w_{eff} = w =$ 3⁄16 in. From Table 2-3, $t_{min} = 0.166$ in. for ASTM A500 Grade B HSS with a 3⁄16 in. weld size. The design thickness of the HSS is $t = 0.233$ in., which is greater than $t_{min} = 0.166$ in. Therefore, the shear strength of the weld metal controls over the shear strength of the HSS base metal.

The gusset plate has two shear planes. Because $F_{yp}/F_{up} = 0.621 < 0.750$, the limit state of shear yielding controls over the limit state of shear rupture. As derived in Section 2.3, the effective weld size is:

$$D_{eff} = 30.2\left(\frac{F_y}{F_{EXX}}\right)t_{min} \tag{2-3a}$$

$$= 3.00 \text{ sixteenths-of-an-inch}$$

Solve for t_{min}:

$$t_{min} = \frac{D_{eff}}{30.2}\left(\frac{F_{EXX}}{F_y}\right)$$

$$= \frac{3.00}{30.2}\left(\frac{70 \text{ ksi}}{36 \text{ ksi}}\right)$$

$$= 0.193 \text{ in.}$$

$$t_p = 0.375 \text{ in.} < 2t_{min}$$

$$2t_{min} = 2(0.193 \text{ in.})$$

$$= 0.386 \text{ in.}$$

$$0.375 < 0.386$$

Therefore, shear yielding of the gusset plate base metal controls over the weld metal. From AISC *Specification* Section J2.4, the nominal strength of the base metal for the limit state of shear yielding is:

$$R_n = F_{BM}A_{BM} \qquad \text{(\textit{Spec.} Eq. J2-2)}$$

AISC *Specification* Table J2.5 stipulates that the available shear strength of the base metal is governed by Section J4. From Section J4.2(a), the nominal shear strength for the limit state of shear yielding of the gusset plate is:

$$R_n = 0.60F_yA_g \qquad \text{(\textit{Spec.} Eq. J4-3)}$$

Therefore, the variables in Equation J2-2 are defined as follows:

$$F_{BM} = 0.6F_{yp}$$

$$= 0.6(36 \text{ ksi})$$

$$= 21.6 \text{ ksi}$$

The cross-sectional area of the base metal at the weld is:

$$A_{BM} = 2lt_p$$

$$= 2(6.00 \text{ in.})(0.375 \text{ in.})$$

$$= 4.50 \text{ in.}^2$$

Therefore, the nominal strength of the base metal for the limit state of shear yielding is:

$$R_n = 21.6 \text{ ksi}\left(4.50 \text{ in.}^2\right)$$

$$= 97.2 \text{ kips}$$

The available strength of the base metal for the limit state of shear yielding is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(97.2 \text{ kips})$ $= 97.2 \text{ kips}$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{97.2 \text{ kips}}{1.50}$ $= 64.8 \text{ kips}$

Bolt shear

From AISC *Manual* Table 7-1, the available shear strength for four ⅞-in.-diameter ASTM A325-N bolts is determined as follows:

LRFD	ASD
$\phi_v r_n = 21.6 \text{ kips/bolt}$ $\phi_v R_n = 4(\phi_v r_n)$ $\phi_v R_n = 4(21.6 \text{ kips/bolt})$ $= 86.4 \text{ kips}$	$\dfrac{r_n}{\Omega_v} = 14.4 \text{ kips/bolt}$ $\dfrac{R_n}{\Omega_v} = 4\left(\dfrac{r_v}{\Omega_v}\right)$ $= 4(14.4 \text{ kips/bolt})$ $= 57.6 \text{ kips}$

Bolt bearing

Assume the gusset plate is thinner than the plate member it is bolted to and will therefore control the strength. From AISC *Specification* Section J3.10(a)(i):

$$R_n = 1.2L_c tF_u \leq 2.4dtF_u \qquad (Spec. \text{ Eq. J3-6a})$$

where

$t = t_p$
$= 0.375 \text{ in.}$
$F_u = F_{up}$
$= 58 \text{ ksi}$

For the end bolts:

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the material, in.
$= 1.50 - d_h/2$
$= 1.50 \text{ in.} - {}^{15}\!/_{16} \text{ in.}/2$
$= 1.03 \text{ in.}$

Therefore, the nominal bearing strength at the end bolt holes is:

$$R_n = 1.2(1.03 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})$$
$$= 26.9 \text{ kips} < 2.4(\tfrac{7}{8} \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})$$
$$= 45.7 \text{ kips}$$

Use $R_n = 26.9$ kips.

The available bearing strength for the four bolts is determined as follows:

LRFD	ASD
For the end bolts $\phi = 0.75$ $\phi R_n = 0.75(26.9 \text{ kips})$ $= 20.2 \text{ kips}$ From AISC *Manual* Table 7-5, the available bearing strength for the interior bolts is: $\phi_v r_n = 91.4 \text{ kips per inch of thickness}$ $\phi R_n = 91.4 \text{ kips/in.}(0.375 \text{ in.})$ $= 34.3 \text{ kips}$ The available bearing strength for the four bolts is: $\phi R_n = 2(20.2 \text{ kips}) + 2(34.3 \text{ kips})$ $= 109 \text{ kips}$	For the end bolts $\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{26.9 \text{ kips}}{2.00}$ $= 13.5 \text{ kips}$ From AISC *Manual* Table 7-5, the available bearing strength for the interior bolts is: $\dfrac{r_n}{\Omega_v} = 60.9 \text{ kips per inch of thickness}$ $\dfrac{R_n}{\Omega} = 60.9 \text{ kips/in.}(0.375 \text{ in.})$ $= 22.8 \text{ kips}$ The available bearing strength for the four bolts is: $\dfrac{R_n}{\Omega} = 2(13.5 \text{ kips}) + 2(22.8 \text{ kips})$ $= 72.6 \text{ kips}$

The available strength in tension is controlled by the limit state of gusset plate tensile rupture.

LRFD	ASD
$\phi R_n = 81.8 \text{ kips}$	$\dfrac{R_n}{\Omega} = 54.5 \text{ kips}$

Chapter 6
Branch Loads on HSS—An Introduction

This brief chapter is intended to serve as an introduction to Chapters 7, 8 and 9, in which connection nominal strengths are tabulated, along with applicable limits of validity for the formulas. These design procedures are in accordance with Chapter K of the AISC *Specification* (AISC, 2005a) with some minor modifications. Classic failure modes for HSS welded connections, wherein the main (or through) HSS member is loaded by attached HSS branches or plates, are described in this chapter. This provides a physical understanding of the limit states that are to be checked in the following chapters and, importantly, allows the user of this Design Guide to understand HSS connection behavior and extrapolate "engineering judgment" to other connection types that are beyond the scope of this Design Guide. The chapter also provides some generic design guidance for HSS connections. Information on the connection classification for HSS-to-HSS truss connections can be found in Section 8.3, which discusses K-, N-, Y-, T- and X-connections.

6.1 PRINCIPAL LIMIT STATES

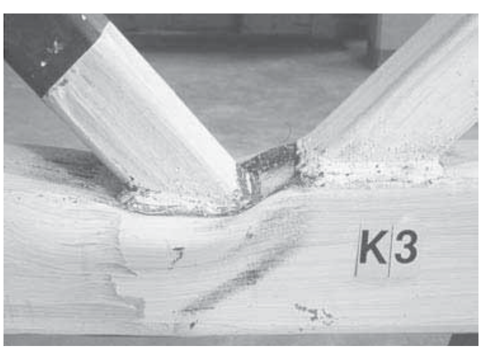
6.1.1 Chord or Column Wall Plastification

This failure mode is particularly prevalent in HSS connections due to the flexible nature of the connecting HSS face, which readily distorts under loading normal to the chord connecting surface. If the main member is a square or rectangular HSS, then the chord connecting face acts like a flat plate under transverse load, supported by two remote webs. The HSS connecting face acts relatively independent of the other three sides. Plastification of the connecting HSS face is the most common failure mode for gapped K- and N-connections with small to medium ratios of branch (or web) member widths to chord width (β). In the case of gapped K-connections, the actions of the compression branch and tension branch develop a "push–pull" mechanism on the chord connecting face, usually resulting in large deformations of the connecting face as shown in Figure 6-1. This limit state is the one specifically covered by AISC *Specification* Equation K2-20.

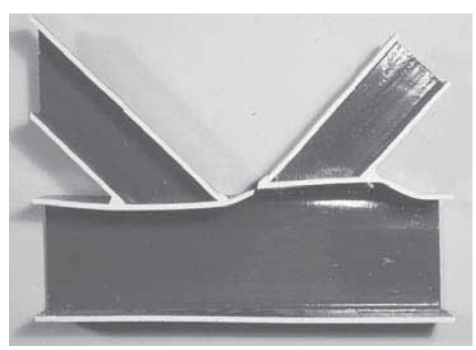
If the main member is a round HSS, then the chord behaves like a closed ring under transverse load and chord plastification results in distortion of the entire chord cross-section. The failure mode in Figure 6-2(a) is the one specifically covered by AISC *Specification* Equations K2-6 and K2-8. Chord plastification applies to many other HSS connection types, including T-, Y- and cross-connections under branch axial loading (AISC *Specification* Equations K2-3, K2-5 and K2-13); T-, Y- and cross-connections under branch moment loading (AISC *Specification* Equations K3-3, K3-5, K3-11 and K3-15); and branch plate-to-HSS connections (AISC *Specification* Equations K1-1, K1-8 and K1-9). Longitudinal plate-to-HSS connections under branch axial load are particularly susceptible to chord plastification, and Figure 6-2(b) shows the gross deformations that are achieved at the ultimate capacity of such a connection. As a result, the connection nominal strength for this limit state also includes a connection deformation control.

6.1.2 Chord Shear Yielding (Punching Shear)

This failure mode, often termed simply "punching shear," may govern in various HSS connections, particularly for medium to high branch-to-chord width ratios. With this failure mechanism, a patch of chord material pulls out (or punches in) around the footprint of a branch member, at the toe of



(a) Balanced K-connection at ultimate load.



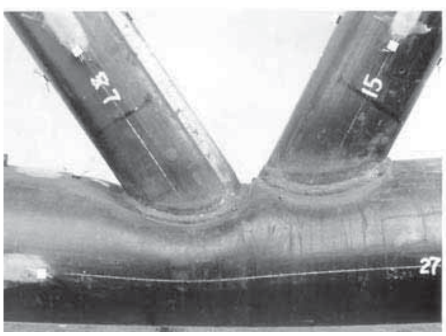
(b) Longitudinal cross-section through an HSS K-connection at ultimate load.

Fig. 6-1. Chord wall plastification for rectangular HSS gapped K-connections.

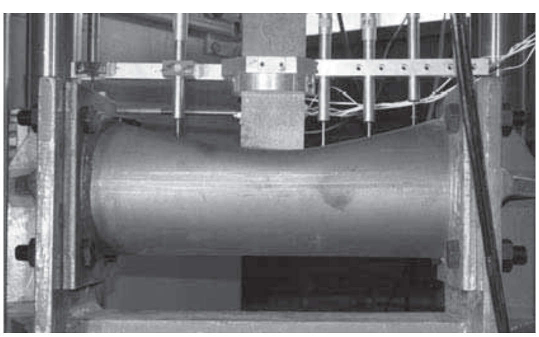
the weld to the chord. Such a failure can occur under either a tension branch or a compression branch member, providing the branch is physically capable of shearing through the chord wall [i.e., the branch outside width (or diameter) is less than the inside width (or diameter) of the chord]. Shear failure strength is typically calculated in steel design codes based on either the ultimate shear stress ($F_u/\sqrt{3} \approx 0.6F_u$), with a relatively low resistance factor on the order of 0.75, or the shear yield stress ($F_y/\sqrt{3} \approx 0.6F_y$), with a resistance factor close to unity. The AISC *Specification* uses both of these approaches in Section J4 for "block shear," checking the limit states of shear yielding and shear rupture. In Chapter K, however, the AISC *Specification* checks simply for shear yielding with a shear yield stress of $0.6F_y$ and a resistance factor of 0.95. The chord shear yielding limit state check applies to a considerable number of HSS connections: T-, Y- and cross-connections under branch axial loading (AISC *Specification* Equations K2-4 and K2-14); gapped K-connections (AISC *Specification* Equations K2-9 and K2-21); T-, Y- and cross-connections under branch moment loading (AISC *Specification* Equations K3-4 and K3-6); and transverse branch plate-to-HSS connections (AISC *Specification* Equation K1-3). One should note that the shear yield stress of $0.6F_y$ is not always applied to the entire perimeter around the footprint of a branch member—for transverse plates or transverse walls of a rectangular HSS across a rectangular HSS chord member, just an *effective* width is used (see AISC *Specification* Equations K1-3, K2-14 and K2-21). This is because the transverse plate (or transverse wall of an HSS branch) is not uniformly loaded across its width; it is very highly stressed at the outer portions of its width adjacent to the (stiff) sidewalls of the rectangular HSS chord member. As a consequence, the transverse plate (or transverse wall of an HSS branch) punches out the chord connecting face prematurely in these highly loaded regions [see Figure 6-3(a)]. For round HSS connections under branch axial loading, on the other hand, uniform punching shear is assumed around the footprint of the branch, commensurate with the failure mode shown in Figure 6-3(b).

6.1.3 Local Yielding Due to Uneven Load Distribution

This failure mode applies to transverse plates, or transverse walls of a rectangular HSS, across a rectangular HSS chord member. It is analogous to the plate effective punching shear width concept described for the previous failure mode, except now the effective width is applied to the transverse element itself rather than the HSS chord, resulting in premature failure of that element. In tension, local yielding and then premature failure of the transverse element occurs, as shown



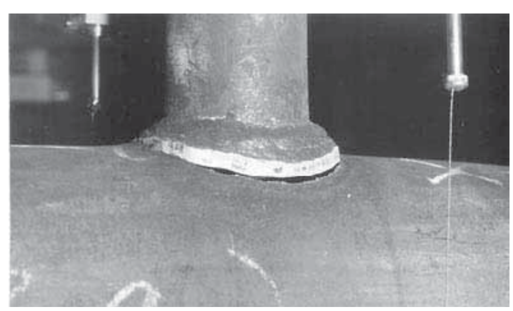
(a) Balanced gapped K-connection at ultimate load.



(b) Longitudinal plate connection at ultimate load.

Fig. 6-2. Chord plastification for round HSS connections.

(a) Transverse plate-to-rectangular HSS connection at ultimate load. Note that rupture initiates at the plate extremities.



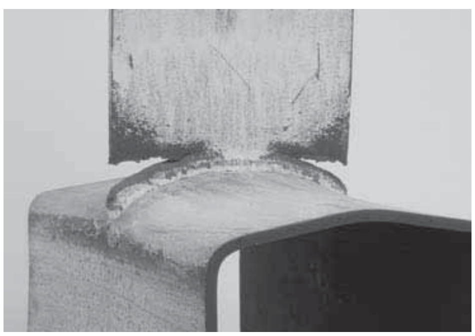
(b) Round HSS T-connection at ultimate load.

Fig. 6-3. Chord shear yielding (punching) for HSS connections.

in Figure 6-4(a). In compression, local yielding usually results in premature local buckling of the element. This is the most common failure mode for rectangular HSS overlapped K-connections, wherein local buckling of the compression branch occurs [see Figure 6-4(b)]. The local yielding due to uneven load distribution limit state check is applied to many HSS connections with rectangular chord members in the AISC *Specification*: T-, Y- and cross-connections under branch axial loading (AISC *Specification* Equation K2-18); gapped and overlapped K-connections (AISC *Specification* Equations K2-22, K2-24, K2-25 and K2-26); T- and cross-connections under branch moment loading (AISC *Specification* Equations K3-13 and K3-17); and transverse branch plate-to-HSS connections (AISC *Specification* Equation K1-2). An inspection of the latter equation (K1-2) shows that the effective width of a transverse element depends highly on the slenderness of the main HSS chord member—if the chord connecting face is thin and flexible (high *B/t*), then the effective width will be low. Conversely, maximum transverse element effective width is achieved for stocky chords (low *B/t*), but with an upper limit of the actual element width. This underlines the design principle (see Section 6.2) of achieving high HSS connection strengths by using stocky (thick-walled) chord members.

6.1.4 Chord or Column Sidewall Failure

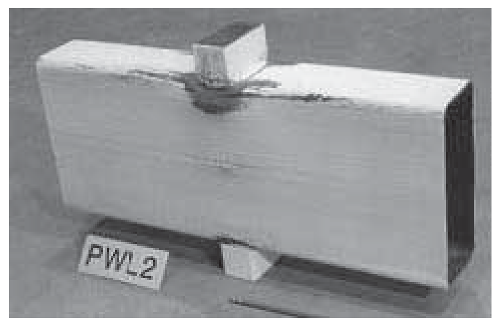
Failure of the chord member sidewalls, rather than the connecting face, may occur in rectangular HSS connections when the branch width is close to, or equals, the chord member width (i.e., $\beta \approx 1.0$). Such connections are also often termed "matched box connections" [e.g., by AWS D1.1 Chapter 2 (AWS, 2006)]. If the branch is in tension, the failure mode of the chord is sidewall local yielding (AISC *Specification* Equations K1-4, K2-15, K3-12 and K3-16) over a dispersed load width. The same failure mechanism is also possible if the branch is in compression—particularly if the chord side walls are relatively stocky (low *H/t*) or if the bearing length is low [see Figure 6-5(a)]. Under branch compression loading, sidewall local crippling or sidewall local buckling are



(a) Transverse plate-to-rectangular HSS connection at ultimate load. Note that rupture initiates at the plate extremities.

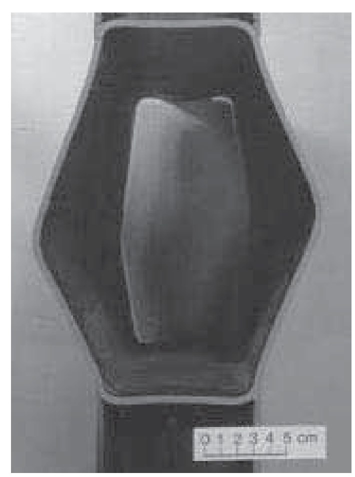
(b) Rectangular HSS overlapped K-connection at ultimate load.

Fig. 6-4. Local yielding of branch due to uneven load distribution.



(a) Sidewall local yielding.

(b) Sidewall local buckling.



(c) Cross-section through connection in (b).

Fig. 6-5. Rectangular HSS sidewall failure under branch compression loading.

also potential failure modes (AISC *Specification* Equations K1-5, K1-6, K2-16 and K2-17). The equations representing both of these limit states have been adapted (for two webs in compression) from concentrated loading to a W-section beam flange, elsewhere in the AISC *Specification*. Sidewall local buckling in a rectangular HSS cross-connection is shown in Figure 6-5(b), where the dark lines in the connection indicate regions of large plastic strain causing flaking of the whitewash. Figure 6-5(c) shows the overall web buckling mechanism, which is assumed to occur over a buckling length of $H - 3t$ with welded branches. [Note that $1.5t$ is the minimum outside corner radius that hence produces the maximum flat length for the HSS wall, according to AISC *Specification* Section B4.2(d).]

Some other connection limit states are analyzed in Chapter 7 (for special connection types), Chapter 8 (shear failure of the chord member side walls for rectangular HSS), and Chapter 9 (chord distortional failure for rectangular HSS connections under out-of-plane bending) but these are much less common. One known failure mode for rectangular HSS K-connections, and in particular overlapped K-connections, is local buckling of the chord connecting face behind the heel of the tension branch, as illustrated in Figure 6-6. Such failure is caused by a shear lag effect, because a disproportionate amount of the chord axial load is carried by the chord connecting face in this region. This limit state check is omitted in Chapter 8 as such failure can be precluded by placing strict wall slenderness limits on the chord connecting face. (Hence, in Table 8-2A, B/t has an upper limit of 35 for gapped K-connections and 30 for overlapped K-connections.)

6.2 DESIGN TIPS

Whenever possible, HSS connections should be designed to be unreinforced, for reasons of both economy and aesthetics. To do so, the members need to be selected astutely at the member selection stage in order to avoid subsequent problems at the connection design stage. The design engineer responsible for member selection must perform checks on the adequacy of critical connections, to be confident with the selected members. Usually the goal is to achieve a hollow section steel structure without any stiffeners or reinforcement; clearly this would entail very careful design even with open steel sections. Some golden rules to optimize welded HSS connection design follow.

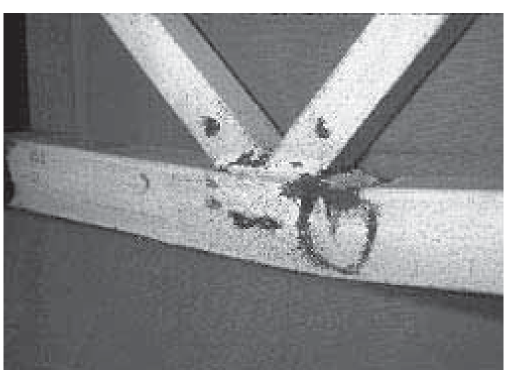


Fig. 6-6. Local yielding of chord due to uneven load distribution.

- *Select relatively stocky chord (or column) main members.*
  The static strength of nearly all HSS connections is enhanced by using stocky chord members, so this choice will maximize connection capacity. To achieve a high buckling resistance, there is a natural design tendency to select compression chord members that are thin-walled and have a high radius of gyration; this, however, is the classic cause of connection design problems later on. The conflicting requirements of axial load capacity, corrosion protection (a smaller surface area is obtained with stocky sections which reduces painting costs), and tube wall slenderness need to all be considered. In general, the cross-section slenderness of chord members is usually within the range $15 \le D/t \le 30$ for round HSS and within the range $15 \le B/t \le 25$ for square and rectangular HSS.

- *Select relatively thin branch members.*
  Converse to the preceding rule (where the chord members are ideally relatively thick), maximum connection efficiency will typically be realized with relatively thin branch members. The branches, however, still have an axial force requirement, so this implies using branches with large outside dimensions. Thus, a good connection design strategy is to make t_b/t as low as possible and B_b/B as high as possible, but one should still try to sit the branch on the "flat" of the chord member with rectangular HSS chord members. With $B_b < B - 4t$, fillet welding of the branch is usually possible, and difficult, expensive flare-groove welds (arising when $B_b \approx B$) can be avoided.

- *Consider using gapped K-connections.*
  With truss-type construction involving K- or N-connections, gapped connections are easier and less expensive to fabricate than overlapped connections, so gapped connections are much more popular with fabricators. This is particularly the case with round-to-round HSS welded connections, where branch member ends require complex profiling and the fit-up of members requires special attention. On the other hand, overlapped K-connections—relative to gapped K-connections—generally do have a higher static (and even fatigue) strength and produce a stiffer truss with reduced truss deflections.

Chapter 7
Line Loads and Concentrated Forces on HSS

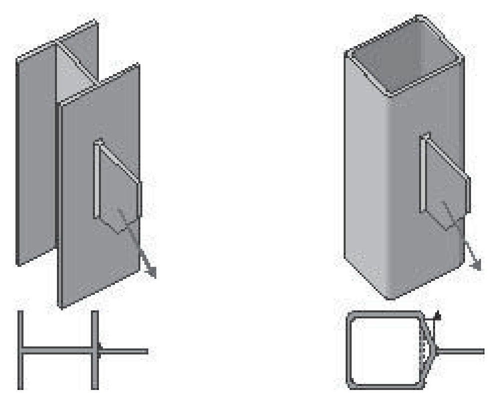
7.1 SCOPE AND BASIS

The scope of this chapter follows AISC *Specification* Section K1 and is intended for local "line loads" applied to the face of an HSS or to the end of an HSS member via a cap plate. The line loads are typically applied by a welded plate, oriented either longitudinal or transverse to the main HSS member axis, with the plate generally loaded in axial tension or compression. Strong-axis bending moments applied to a plate-to-round HSS connection are also covered, as is the case of shear loading on a longitudinal plate-to-HSS connection. Aside from the cap-plate case, these local loads are assumed to be applied away from the ends of an HSS member; hence, the concentrated load can be dispersed to either side of the connection. If the local line load (transverse or longitudinal) occurs near the end of the member, then it is assumed that the member end would be capped, thus regaining a similar connection strength. Wide-flange beam-to-HSS column directly welded moment connections are also indirectly covered by the scope of this chapter. The flexural capacity of such connections can be determined by ignoring the beam web and considering the beam flanges as a pair of transverse plates welded to the HSS. The moment capacity of the connection—which would be semi-rigid or partially restrained (PR)—can then be computed by multiplying the capacity of one plate-to-HSS connection by the lever arm between the centroids of the two flanges (the beam depth minus one flange thickness). When plates are oriented longitudinal to the HSS member axis, it is assumed that the plate is aligned with the HSS member axis. However, if a longitudinal plate is slightly offset from the centerline—perhaps so that a beam or diagonal bracing member centerline can coincide with the HSS column centerline—the difference in connection capacity is small and can be ignored.

Research on plate connections dates back to the 1960s (Rolloos, 1969), with numerous studies specifically applied to HSS (Kurobane, 1981; Wardenier et al., 1981; Davies and Packer, 1982; Makino et al., 1991; Cao et al., 1998; Kosteski and Packer, 2003). These form the basis of the design recommendations in this chapter; most equations (after application of appropriate resistance or safety factors) conform to CIDECT Design Guides 1 (Wardenier et al., 1991) or 3 (Packer et al., 1992) with updates in accordance with CIDECT Design Guide 9 (Kurobane et al., 2004). The latter includes revisions for longitudinal plate-to-rectangular HSS connections (AISC *Specification* Equation K1-9) based on extensive experimental and numerical studies reported in Kosteski and Packer (2003). Still further revisions to the plate-to-round HSS design criteria have again been recently recommended (Wardenier et al., 2008).

7.2 LIMIT STATES

Many plate-to-HSS welded connections are extremely flexible. As shown in Figure 7-1, a concentric longitudinal plate welded to a wide flange section is inherently stiffened by the presence of the web immediately behind the plate applied load. The application of the same connection practice to an HSS member results in flexure of the connecting HSS face, because the plate force is transmitted to the two HSS webs remote from the point of load application. As a consequence, the limit state of HSS plastification is a failure mode that must be commonly checked, and this serves as a control on connection deformations. The design recommendations herein make no distinction between branch plate loading in axial tension and compression. For rectangular HSS connections, there is little difference in the connection behavior—and hence the design capacity—as the connecting HSS face behaves as a laterally loaded flat plate. For plate-to-round HSS connections, however, the ultimate connection strength is usually higher under plate tension loading than under plate compression loading. For tension loading, excessive connection deformations or premature cracking may be present, well before the connection ultimate load, so—following the example of round-to-round HSS welded T-, Y- and cross-connections (see Chapter 8)—the same design capacity is conservatively used for both plate axial



(a) To a wide flange. *(b) To a rectangular HSS.*

Fig. 7-1. Longitudinal-plate connections.

tension and compression load cases. Hence, because deformation limits frequently control the nominal strengths of these connections, the design capacity is often considerably less than the ultimate capacity that is recorded in laboratory tests. For diagonal bracing connections to HSS columns, where the axial load is transferred to a longitudinal gusset plate, the connection capacity can be checked under the bracing force component normal to the column axis (using Table 7-1 or Table 7-2), as this will govern.

Because the available strength of a longitudinal plate-to-HSS connection is rather low, there are instances in which the connection may need to be locally reinforced (e.g., in hanger connections, high loads from cables supporting roofs or bridges, etc.). A common means of reinforcement (although not favored by fabricators) is to pass the longitudinal plate through the complete cross-section, after slotting the HSS, and to weld the "through plate" to both the front and back sides of the HSS. If this is done for a rectangular HSS (Figure 7-2) the nominal strength can be taken as twice that given by AISC *Specification* Equation K1-9, as is also indicated in Table 7-2 (Kosteski and Packer, 2003). Caution should be exercised before applying this principle to plate-to-round HSS through-plate connections; recent research has suggested that the nominal strength is less than twice that of the corresponding branch-plate connection.

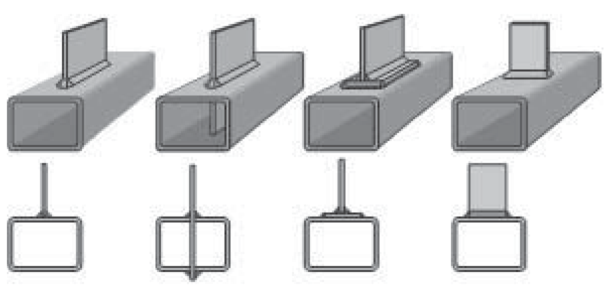


Fig. 7-2. Longitudinal-plate, through-plate, stiffened longitudinal plate and transverse-plate connections.

In addition to the limit state of HSS plastification (or HSS local yielding) described previously, other pertinent limit states for plate-to-HSS connections are (1) local yielding due to uneven load distribution in the loaded plate, (2) shear yielding (punching shear) of the HSS, and (3) sidewall strength (for rectangular HSS) by various failure modes. The provisions for the limit state of sidewall crippling of rectangular HSS (one of the sidewall failure modes), Equations K1-5 and K1-6 in the AISC *Specification,* conform to web crippling expressions elsewhere in the AISC *Specification,* and not to CIDECT or International Institute of Welding (IIW) recommendations. The pertinent limit states to be checked for plate-to-HSS connections are summarized in Table 7-1 (for round HSS) and Table 7-2 (for rectangular HSS). It is important to note that a number of potential limit states can often be precluded from the connection checking procedure because the corresponding failure modes are excluded—by virtue of the connection geometry and the limits of applicability of various parameters. The limits of applicability (given in Tables 7-1A and 7-2A) generally represent the parameter range over which the design equations have been verified in experiments or by numerical simulation. In Table 7-2 (and also in AISC *Specification* Section K1.3b), for transverse plate connections to rectangular HSS, it is evident that there is no check for the limit state of HSS wall plastification. This is omitted because this limit state will not govern design in practical cases. However, if there is a major compression load in the HSS—such as when it is used as a column—one should be aware that this compression load in the main member has a negative influence on the yield line plastification failure mode of the connecting HSS wall (via a Q_f factor). In such a case, the designer can utilize guidance in CIDECT Design Guide 9 (Kurobane et al., 2004).