# Fillet Weld Design for Rectangular HSS Connections

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## ABSTRACT

The 2010 AISC *Specification for Structural Steel Buildings* has expanded the scope in Chapter K ("Design of HSS and Box Member Connections") with a new Section K4, "Welds to Branches." This paper describes the background to this development and examines the structural reliability of the weld effective length provisions contained therein. The latter is achieved by using a database of 31 welded square/rectangular hollow structural section (HSS) K-, T- and cross- (or X-) connections in which all test specimen failures were reached by fracture of the welded joints. The potential inclusion (or exclusion) of the sin θ effect, whereby fillet weld capacity is increased for loading not parallel to the axis of the weld, has also been investigated. In conclusion, some design examples are given to illustrate the weld design method, performed to the 2010 AISC *Specification* in both LRFD and ASD.


**Keywords:** hollow structural sections, connections, joints, welding, trusses, effective weld lengths.

## WELD DESIGN PHILOSOPHY FOR HSS CONNECTIONS

AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (Miller, 2006), provides excellent general advice for engineers on welding, including some particular remarks on welding hollow structural sections (HSS). It has been well known, however, that the welds at the footprint of an HSS branch member, like the four walls of the HSS branch itself, are generally loaded in a highly nonuniform manner. In particular, the branch walls (and adjacent welds) transverse to the direction of a HSS truss are liable to have a very high concentration of strain and stress at the branch corners but low strain and stress in the middle of the branch transverse wall. The lower the stress in the middle portion of a branch transverse wall, the less effective that location is in resisting load.

With HSS-to-HSS welded connections, there are currently two design philosophies that can be used for weld design, as described in AISC Design Guide 24, *Hollow Structural Section Connections* (Packer et al., 2010):

1. The weld can be proportioned so that it develops the yield strength of the connected branch wall at all locations around the branch. This will represent an upper limit on the weld size and hence be a conservative design procedure. This approach is particularly appropriate if there is low confidence in the design forces acting on the branch, or if there is uncertainty regarding method 2, or if plastic stress redistribution is required in the connection. The same effective weld size should be maintained all around the attached branch. The one exception to this latter rule is for the hidden weld in HSS-to-HSS partially overlapped K- (or N-) connections, which may be left unwelded (usually just tacked) provided the force components of the two branches normal to the chord do not differ by more than 20%. The hidden weld refers to the weld along the hidden toe of the overlapped branch, which is hidden in the final connection by the overlapping branch. This is particularly an issue with square/rectangular partially overlapped HSS-to-HSS connections where the typical fabrication procedure is to tack all the branches into place and perform final welding afterward (Packer et al., 2010).
2. The weld can be proportioned so that it resists the applied forces in the branch. This approach may be appropriate where there is high confidence in the design forces acting on the branch, or if the branch forces are particularly low relative to the branch member capacity. The latter situation often arises if the branches are sized for aesthetic reasons. For example, the branches of a simply supported truss may be the same throughout the truss, yet at the center of the truss the web member forces will be very low under uniformly distributed load. If this design philosophy is adopted, then weld effective lengths must be taken into account, because HSS connections are usually very flexible. The same effective weld size should still be maintained all around the attached branch, with the entire branch perimeter welded (including the hidden toe, if applicable).

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## HISTORICAL TREATMENT OF WELD DESIGN IN HSS CONNECTIONS

In 1981, Subcommission XV-E of the International Institute of Welding (IIW) produced its first design recommendations for statically loaded HSS connections, which were updated and revised with a second edition later that decade (IIW, 1989). These recommendations are still the basis for nearly all current design rules around the world dealing with statically loaded connections in onshore HSS structures, including those in Europe (CEN, 2005), Canada (Packer and Henderson, 1997) and the United States (AISC, 2010). The IIW (1989) recommendations and welding requirements are discussed next.

Fillet welds that are automatically prequalified for any member loads should be designed to give a resistance that is not less than the member capacity. This results in the following minimum throat thickness for fillet welds, assuming matched electrodes:

$$t_w \geq 0.95 t_b \text{, for Fe 360 } (F_{yb} = 235 \text{ MPa or } 34.1 \text{ ksi}) \tag{1a}$$

$$t_w \geq 1.00 t_b \text{, for Fe 430 } (F_{yb} = 275 \text{ MPa or } 39.9 \text{ ksi}) \tag{1b}$$

$$t_w \geq 1.07 t_b \text{, for Fe 510 } (F_{yb} = 355 \text{ MPa or } 51.5 \text{ ksi}) \tag{1c}$$

This requirement may only be waived where departure from them can be justified in terms of strength and deformational and/or rotational capacity.

Research at the University of Toronto (Frater and Packer, 1992a, 1992b) on fillet-welded rectangular HSS branches in large-scale Warren trusses with gapped K-connections (Figure 1) showed that fillet welds in that context can be proportioned on the basis of the loads in the branches, thus resulting in relatively smaller weld sizes compared to IIW (1989). It was concluded simplistically that the welds along all four sides of the HSS branch could be taken as fully effective when the chord-to-branch angle is 50° or less, but that the weld along the heel should be considered as completely ineffective when the angle is 60° or more. A linear interpolation was recommended when the chord-to-branch angle is between 50° and 60°. Based on this research, the formulas for the effective length of branch member welds in planar, gapped, rectangular HSS K- and N-connections, subject to predominantly static axial load, were taken in Packer and Henderson (1992) as:

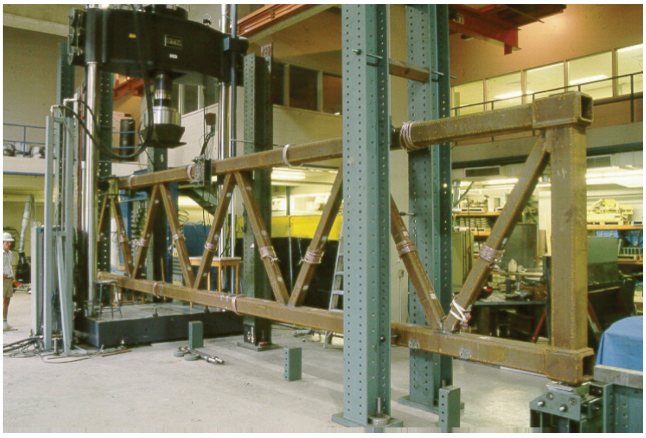
When $\theta \leq 50°$:

$$L_e = \frac{2H_b}{\sin\theta} + 2B_b \tag{2a}$$

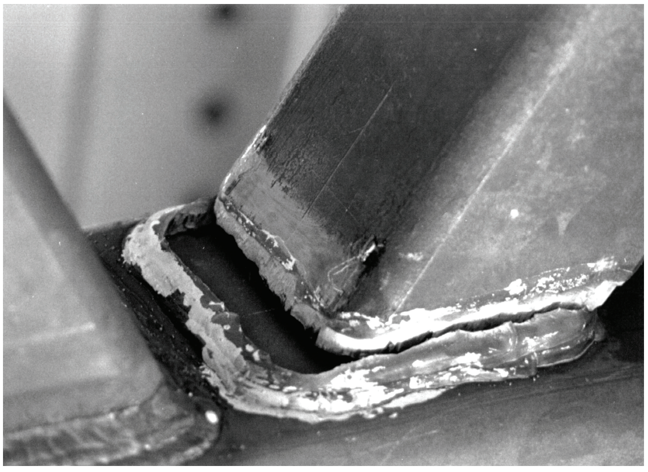
When $\theta \leq 60°$:

$$L_e = \frac{2H_b}{\sin\theta} + B_b \tag{2b}$$

In 1992, the American Welding Society (AWS) adopted these recommendations, and a linear interpolation was recommended between 50° and 60°. For the welds in planar T-, Y- and cross- (or X-) connections, AWS (1992) also implemented the following effective-length formulas based on the conclusions from the research on gapped rectangular HSS K-connections:



*(a)*



*(b)*

*Fig. 1. Large-scale, laboratory testing of rectangular HSS K-connections: (a) welded HSS truss with gapped and overlapped K-connections; (b) weld failure in a gapped K-connection.*

When $\beta \leq 0.85$:

$$L_e = \frac{2H_b}{\sin\theta} + 2B_b \tag{3a}$$

When $\beta \leq 0.85$:

$$L_e = \frac{2H_b}{\sin\theta} \tag{3b}$$

However, Packer and Henderson (1992) wrote that the addition of the $2B_b$ term in Equation 3a was likely nonconservative for $\theta \geq 60°$. In a further study by Packer and Cassidy (1995), by means of 16 large-scale connection tests that were designed to be weld-critical (Figure 2), new weld effective length formulas for T-, Y- and cross- (or X-) connections were developed. It was found that more of the weld perimeter was effective for lower branch member inclination angles for T-, Y- and cross (or X-) connections. Thus, the formulas for the effective length of branch member welds in planar T-, Y- and cross- (or X-) rectangular HSS connections, subjected to predominantly static axial load, were revised in Packer and Henderson (1997) to:

When $\theta \leq 50°$:

$$L_e = \frac{2H_b}{\sin\theta} + B_b \tag{4a}$$

When $\theta \leq 60°$:

$$L_e = \frac{2H_b}{\sin\theta} \tag{4b}$$

A linear interpolation was recommended between 50° and 60°.

The latest (third) edition of the International Institute of Welding recommendations (IIW, 2009) requires that the design resistance of HSS connections be based on failure modes that do not include weld failure, with the latter being avoided by satisfying either of the following criteria:

1. Welds are to be proportioned to be "fit for purpose" and to resist forces in the connected members, taking account of joint deformation/rotation capacity and considering weld effective lengths.
2. Welds are to be proportioned to achieve the capacity of the connected member walls.

This IIW (2009) document thus specifically acknowledges the effective-length concept for weld design. The most recent guidance on this topic (Packer et al., 2009) again notes that for ISO steel grades and matched electrodes, the minimum throat thickness for fillet welds to achieve branch member yield capacity is (CEN, 2005):

$$t_w \geq 0.92t_b \text{ , for S235 } (F_{yb} = 235 \text{ MPa or } 34.1 \text{ ksi}) \tag{5a}$$

$$t_w \geq 0.96t_b \text{ , for S275 } (F_{yb} = 275 \text{ MPa or } 39.9 \text{ ksi}) \tag{5b}$$

$$t_w \geq 1.10t_b \text{ , for S355 } (F_{yb} = 355 \text{ MPa or } 51.5 \text{ ksi}) \tag{5c}$$

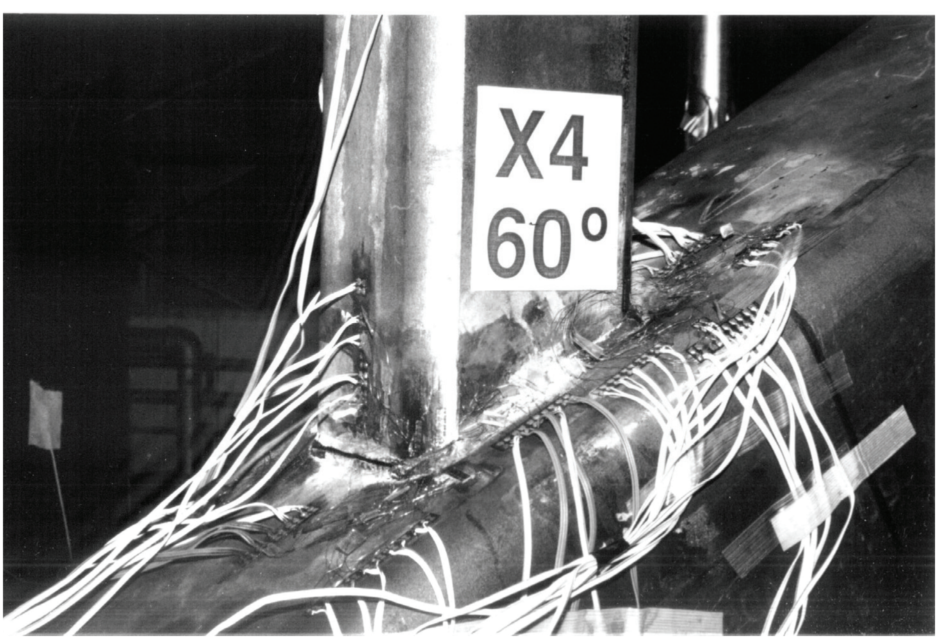



*Fig. 2. Laboratory testing of rectangular HSS cross- (or X-) connections, with failure of the welds.*

$$t_w \geq 1.42 t_b \text{, for S420 } (F_{yb} = 420 \text{ MPa or } 60.9 \text{ ksi}) \tag{5d}$$

$$t_w \geq 1.48 t_b \text{, for S460 } (F_{yb} = 460 \text{ MPa or } 66.7 \text{ ksi}) \tag{5e}$$

## 2010 AISC SPECIFICATION, SECTION K4

In Section K4 of the 2010 AISC *Specification* (AISC, 2010), a more detailed design method considering weld effective lengths ($L_e$) is given for different types of RHS connections and branch loadings, as described next.

### T-, Y- and Cross- (or X-) Connections under Branch Axial Load or Bending

Effective weld properties are given by:

$$L_e = \frac{2H_b}{\sin\theta} + 2b_{eoi} \tag{6}$$

$$S_{ip} = \frac{t_w}{3}\left(\frac{H_b}{\sin\theta}\right)^2 + t_w b_{eoi}\left(\frac{H_b}{\sin\theta}\right) \tag{7}$$

$$S_{op} = t_w\left(\frac{H_b}{\sin\theta}\right)B_b + \frac{t_w}{3}\left(B_b^2\right) - \frac{(t_w/3)\left(B_b - b_{eoi}\right)^3}{B_b} \tag{8}$$

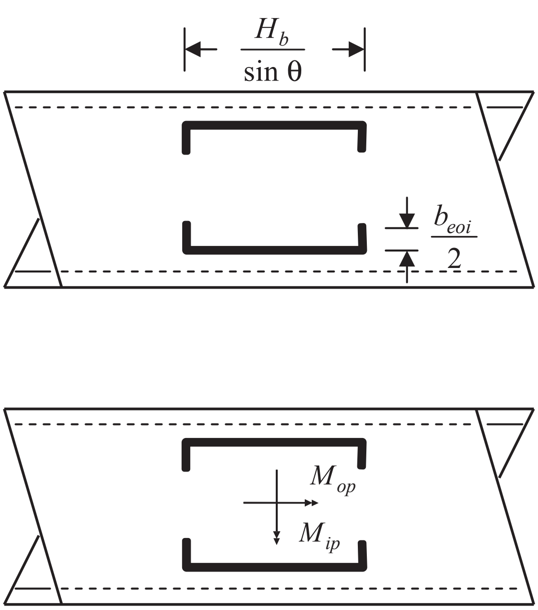



*Fig. 3. Weld effective-length terminology for T-, Y- and cross- (or X-) connections: (a) weld effective length; (b) in-plane and out-of-plane bending moment directions.*

$$b_{eoi} = \frac{10}{B/t}\left(\frac{F_y t}{F_{yb} t_b}\right)B_b \leq B_b \tag{9}$$

When $\beta > 0.85$ or $\theta > 50°$, $b_{eoi}/2$ shall not exceed $2t$. This limitation represents additional engineering judgment.

In contrast to Equations 4a and 4b, the weld effective length in Equation 6 was—for consistency—made equivalent to the branch wall effective lengths used in Section K2.3 of the *Specification* for the limit state of local yielding of the branch(es) due to uneven load distribution, which in turn is based on the IIW *Design Recommendations* (1989). The effective length of the weld transverse to the chord, $b_{eoi}$, is illustrated in Figure 3a. This term, $b_{eoi}$, was empirically derived on the basis of laboratory tests in the 1970s and 1980s (Davies and Packer, 1982). The weld effective elastic section moduli for in-plane bending and out-of-plane bending, $S_{ip}$ and $S_{op}$, respectively (Equations 7 and 8), apply in the presence of bending moments $M_{ip}$ and $M_{op}$, as shown in Figure 3b. Equation 7 can be determined by deriving:

$$\begin{aligned} I_{ip} &= 2 \times \frac{1}{12} t_w \left(\frac{H_b}{\sin\theta}\right)^3 + 4t_w\left(\frac{b_{eoi}}{2}\right)\left(\frac{H_b}{2\sin\theta}\right)^2 \\ &= \frac{t_w}{6}\left(\frac{H_b}{\sin\theta}\right)^3 + \frac{t_w b_{eoi}}{2}\left(\frac{H_b}{\sin\theta}\right)^2 \end{aligned} \tag{10}$$

and substituting into:

$$S_{ip} = \frac{I_{ip}}{\left(\dfrac{H_b}{2\sin\theta}\right)} \tag{11}$$

Similarly, Equation 8 can be determined by deriving:

$$\begin{aligned} I_{op} &= 2t_w\left(\frac{H_b}{\sin\theta}\right)\left(\frac{B_b}{2}\right)^2 + 2 \times \frac{1}{12} t_w\left(B_b^3\right) - 2 \times \frac{1}{12} t_w\left(B_b - b_{eoi}\right)^3 \\ &= \frac{t_w}{2} B_b^2 \frac{H_b}{\sin\theta} + \frac{t_w}{6}\left(B_b^3\right) - \frac{t_w}{6}\left(B_b - b_{eoi}\right)^3 \end{aligned} \tag{12}$$

| Table 1. Measured Properties of 15 Gapped K-Connections Exhibiting Weld Failure | | | | | | |
|---|---|---|---|---|---|---|
| **Truss 1** | | | | | | |
| **Connection Number** | **HSS Chord Member** | | **HSS Branch Member** | | **θ (degrees)** | **$F_{uw}$ (ksi)** |
| | **$H \times B \times t$ (in. × in. × in.)** | **$F_y$, $F_u$ (ksi)** | **$H_b \times B_b \times t_b$ (in. × in. × in.)** | **$F_{yb}$, $F_{ub}$ (ksi)** | | |
| K1 | 12.02 × 8.05 × 0.472 | $F_y$ = 62.6<br>$F_u$ = 74.1 | 5.03 × 5.03 × 0.465 | $F_{yb}$ = 57.6<br>$F_{ub}$ = 73.2 | 60 | 98.3 |
| K2 | | | | | | |
| K3 | | | | | | |
| K4 | | | | | | |
| K5 | | | | | | |
| K6 | | | | | | |
| K7 | | | | | | |
| K8 | | | | | | |
| K9 | | | | | | |
| **Truss 2** | | | | | | |
| **Connection Number** | **HSS Chord Member** | | **HSS Branch Member** | | **θ (degrees)** | **$F_{uw}$ (ksi)** |
| | **$H \times B \times t$ (in. × in. × in.)** | **$F_y$, $F_u$ (ksi)** | **$H_b \times B_b \times t_b$ (in. × in. × in.)** | **$F_{yb}$, $F_{ub}$ (ksi)** | | |
| K10 | 8.03 × 8.03 × 0.476 | $F_y$ = 58.7<br>$F_u$ = 74.8 | 5.04 × 5.04 × 0.472 | $F_{yb}$ = 61.3<br>$F_{ub}$ = 75.7 | 60 | 78.3 |
| K11 | | | | | | |
| K12 | | | | | | |
| K13 | | | | | | |
| K14 | | | | | | |
| K15 | | | | | | |

and substituting into:

$$S_{op} = \frac{I_{op}}{\left(\frac{B_b}{2}\right)} \tag{13}$$

### For Gapped K- and N-Connections under Branch Axial Load

Effective weld lengths are given by:

When θ ≤ 50°:

$$L_e = \frac{2\left(H_b - 1.2t_b\right)}{\sin\theta} + 2\left(B_b - 1.2t_b\right) \tag{14a}$$

When θ ≥ 60°:

$$L_e = \frac{2\left(H_b - 1.2t_b\right)}{\sin\theta} + \left(B_b - 2t_b\right) \tag{14b}$$

When 50° < θ < 60°, a linear interpolation is to be used to determine $L_e$.

No provisions are given for branch moment loading because the recommended K- and N- connection design procedure in *Specification* Section K2.3 is based on axial loads only in the branches of such connections. This results from the truss design method recommended, in which all branch members are considered as pin-ended. Equations 14a and 14b are very similar to Equations 2a and 2b, but the former incorporate a reduction to allow for a typical HSS corner radius. For gapped K- and N-connections, the simplified nature of these effective-length formulas (Equations 14a and 14b) was preferred to the more complex ones that would result if the branch effective widths of the HSS walls in the *Specification* Section K2.3 were adopted. Effective weld length provisions for overlapped HSS K- and N-connections were also provided in *Specification* Section K4 (AISC, 2010), based on branch effective widths of the HSS walls in *Specification* Section K2.3; however, in this case, no research data on weld-critical overlapped HSS K- and N-connections were available.

| Table 2. Measured Properties of 16 T- and X-Connections Exhibiting Weld Failure | | | | | | |
|---|---|---|---|---|---|---|
| Connection Number | HSS Chord Member | | HSS Branch Member | | θ (degrees) | $F_{uw}$ (ksi) |
| | $H \times B \times t$ (in. × in. × in.) | $F_y$, $F_u$ (ksi) | $H_b \times B_b \times t_b$ (in. × in. × in.) | $F_{yb}$, $F_{ub}$ (ksi) | | |
| T1 | 9.99 × 9.99 × 0.476 | $F_y = 59.5$ $F_u = 61.5$ | 5.00 × 5.00 × 0.480 | $F_{yb} = 79.0$ $F_{ub} = 87.4$ | 90 | 83.2 |
| T2 | | | | | | |
| T3 | | | 7.99 × 7.99 × 0.474 | $F_{yb} = 64.5$ $F_{ub} = 66.8$ | | |
| T4 | | | | | | |
| X1 | | | 5.00 × 5.00 × 0.480 | $F_{yb} = 79.0$ $F_{ub} = 87.4$ | 30 | |
| X2 | | | | | 40 | |
| X3 | | | | | 50 | |
| X4 | | | | | 60 | |
| X5 | | | | | 70 | |
| X6 | | | | | 80 | |
| X7 | | | 7.99 × 7.99 × 0.474 | $F_{yb} = 64.5$ $F_{ub} = 66.8$ | 30 | |
| X8 | | | | | 40 | |
| X9 | | | | | 50 | |
| X10 | | | | | 60 | |
| X11 | | | | | 70 | |
| X12 | | | | | 80 | |

| Table 3. Measured Weld Sizes of Gapped K-Connections and Failure Loads of Welded Joints | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Connection Number | Weld 1 (in.) | | | Weld 2 (in.) | | | Weld 3 (in.) | | | Weld 4 (in.) | | | Joint Failure Load (kips) |
| | $w_b$ | $w_c$ | $t_w$ | $w_b$ | $w_c$ | $t_w$ | $w_b$ | $w_c$ | $t_w$ | $w_b$ | $w_c$ | $t_w$ | |
| K1 | 0.236 | 0.260 | 0.154 | 0.217 | 0.220 | 0.154 | 0.205 | 0.224 | 0.106 | 0.268 | 0.343 | 0.240 | 171.1 |
| K2 | 0.220 | 0.264 | 0.165 | 0.228 | 0.272 | 0.161 | 0.197 | 0.248 | 0.110 | 0.264 | 0.295 | 0.224 | 177.6 |
| K3 | 0.256 | 0.311 | 0.173 | 0.256 | 0.287 | 0.189 | 0.201 | 0.291 | 0.118 | 0.311 | 0.406 | 0.276 | 175.8 |
| K4 | 0.295 | 0.283 | 0.197 | 0.272 | 0.299 | 0.201 | 0.224 | 0.291 | 0.126 | 0.350 | 0.409 | 0.311 | 198.9 |
| K5 | 0.382 | 0.425 | 0.256 | 0.402 | 0.433 | 0.260 | 0.378 | 0.382 | 0.189 | 0.441 | 0.496 | 0.370 | 217.4 |
| K6 | 0.402 | 0.413 | 0.268 | 0.374 | 0.398 | 0.240 | 0.366 | 0.378 | 0.185 | 0.472 | 0.476 | 0.390 | 271.8 |
| K7 | 0.531 | 0.433 | 0.323 | 0.492 | 0.469 | 0.319 | 0.492 | 0.406 | 0.224 | 0.559 | 0.500 | 0.445 | 362.6 |
| K8 | 0.500 | 0.476 | 0.331 | 0.457 | 0.476 | 0.327 | 0.465 | 0.429 | 0.228 | 0.579 | 0.543 | 0.449 | 353.4 |
| K9 | 0.496 | 0.551 | 0.343 | 0.555 | 0.484 | 0.346 | 0.587 | 0.567 | 0.287 | 0.606 | 0.622 | 0.520 | 368.0 |
| K10 | 0.205 | 0.283 | 0.146 | 0.209 | 0.331 | 0.154 | 0.213 | 0.248 | 0.110 | 0.299 | 0.394 | 0.244 | 186.8 |
| K11 | 0.287 | 0.417 | 0.197 | 0.307 | 0.421 | 0.217 | 0.260 | 0.394 | 0.150 | 0.390 | 0.516 | 0.307 | 239.0 |
| K12 | 0.339 | 0.429 | 0.248 | 0.346 | 0.445 | 0.252 | 0.382 | 0.445 | 0.205 | 0.504 | 0.555 | 0.386 | 304.2 |
| K13 | 0.512 | 0.634 | 0.362 | 0.457 | 0.650 | 0.350 | 0.461 | 0.528 | 0.244 | 0.567 | 0.760 | 0.492 | 333.4 |
| K14 | 0.469 | 0.630 | 0.358 | 0.528 | 0.598 | 0.370 | 0.555 | 0.606 | 0.287 | 0.634 | 0.724 | 0.528 | 308.9 |
| K15 | 0.650 | 0.815 | 0.484 | 0.669 | 0.913 | 0.508 | 0.760 | 0.713 | 0.350 | 0.783 | 1.094 | 0.681 | 369.6 |

| Table 4. Measured Weld Sizes of T- and X-Connections and Failure Loads of Welded Joints | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Connection Number | Weld 1 (in.) | | | Weld 2 (in.) | | | Weld 3 (in.) | | | Weld 4 (in.) | | | Joint Failure Load (kips) |
| | $w_b$ | $w_c$ | $t_w$ | $w_b$ | $w_c$ | $t_w$ | $w_b$ | $w_c$ | $t_w$ | $w_b$ | $w_c$ | $t_w$ | |
| T1 | 0.205 | 0.264 | 0.157 | 0.205 | 0.256 | 0.142 | 0.189 | 0.260 | 0.142 | 0.244 | 0.220 | 0.154 | 118.5 |
| T2 | 0.244 | 0.319 | 0.185 | 0.232 | 0.291 | 0.169 | 0.280 | 0.319 | 0.209 | 0.244 | 0.287 | 0.181 | 154.4 |
| T3 | 0.213 | 0.248 | 0.146 | 0.197 | 0.185 | 0.130 | 0.154 | 0.220 | 0.126 | 0.201 | 0.244 | 0.134 | 203.9 |
| T4 | 0.252 | 0.287 | 0.181 | 0.256 | 0.299 | 0.185 | 0.256 | 0.291 | 0.161 | 0.220 | 0.299 | 0.165 | 195.1 |
| X1 upper | 0.256 | 0.272 | 0.177 | 0.291 | 0.319 | 0.205 | 0.307 | 0.315 | 0.213 | 0.398 | 0.465 | 0.295 | did not fail |
| X1 lower | 0.213 | 0.291 | 0.154 | 0.228 | 0.295 | 0.173 | 0.291 | 0.343 | 0.224 | 0.457 | 0.539 | 0.378 | 310.2 |
| X2 upper | 0.217 | 0.276 | 0.165 | 0.220 | 0.276 | 0.165 | 0.244 | 0.315 | 0.189 | 0.248 | 0.287 | 0.193 | did not fail |
| X2 lower | 0.232 | 0.299 | 0.177 | 0.256 | 0.283 | 0.185 | 0.307 | 0.358 | 0.213 | 0.295 | 0.311 | 0.244 | 200.7 |
| X3 upper | 0.185 | 0.260 | 0.146 | 0.213 | 0.252 | 0.165 | 0.232 | 0.311 | 0.173 | 0.244 | 0.260 | 0.165 | 149.0 |
| X3 lower | 0.209 | 0.264 | 0.150 | 0.189 | 0.280 | 0.146 | 0.224 | 0.299 | 0.185 | 0.209 | 0.157 | 0.114 | did not fail |
| X4 upper | 0.157 | 0.244 | 0.134 | 0.189 | 0.256 | 0.146 | 0.209 | 0.244 | 0.138 | 0.280 | 0.291 | 0.224 | 104.3 |
| X4 lower | 0.213 | 0.283 | 0.157 | 0.169 | 0.252 | 0.138 | 0.209 | 0.217 | 0.122 | 0.276 | 0.287 | 0.224 | did not fail |
| X5 upper | 0.189 | 0.232 | 0.126 | 0.213 | 0.244 | 0.150 | 0.189 | 0.232 | 0.126 | 0.217 | 0.283 | 0.177 | did not fail |
| X5 lower | 0.177 | 0.236 | 0.138 | 0.154 | 0.240 | 0.126 | 0.161 | 0.181 | 0.106 | 0.232 | 0.280 | 0.193 | 95.3 |
| X6 upper | 0.181 | 0.205 | 0.130 | 0.177 | 0.217 | 0.130 | 0.213 | 0.201 | 0.134 | 0.177 | 0.256 | 0.150 | 96.9 |
| X6 lower | 0.181 | 0.224 | 0.134 | 0.209 | 0.256 | 0.157 | 0.181 | 0.201 | 0.130 | 0.185 | 0.236 | 0.142 | did not fail |
| X7 upper | 0.291 | 0.283 | 0.201 | 0.299 | 0.311 | 0.217 | 0.374 | 0.402 | 0.260 | 0.559 | 0.567 | 0.469 | did not fail |
| X7 lower | 0.287 | 0.307 | 0.201 | 0.303 | 0.299 | 0.213 | 0.276 | 0.429 | 0.224 | 0.512 | 0.531 | 0.425 | 571.2 |
| X8 upper | 0.295 | 0.287 | 0.205 | 0.311 | 0.311 | 0.224 | 0.315 | 0.366 | 0.236 | 0.295 | 0.315 | 0.232 | 380.4 |
| X8 lower | 0.311 | 0.303 | 0.217 | 0.283 | 0.295 | 0.201 | 0.315 | 0.346 | 0.224 | 0.295 | 0.319 | 0.240 | did not fail |
| X9 upper | 0.295 | 0.272 | 0.201 | 0.307 | 0.327 | 0.220 | 0.319 | 0.307 | 0.220 | 0.287 | 0.319 | 0.224 | did not fail |
| X9 lower | 0.307 | 0.299 | 0.217 | 0.327 | 0.327 | 0.232 | 0.256 | 0.311 | 0.201 | 0.260 | 0.299 | 0.205 | 310.2 |
| X10 upper | 0.232 | 0.248 | 0.169 | 0.228 | 0.236 | 0.165 | 0.232 | 0.240 | 0.142 | 0.283 | 0.327 | 0.236 | did not fail |
| X10 lower | 0.240 | 0.276 | 0.189 | 0.232 | 0.248 | 0.173 | 0.205 | 0.264 | 0.130 | 0.276 | 0.350 | 0.228 | 218.5 |
| X11 upper | 0.224 | 0.236 | 0.161 | 0.260 | 0.295 | 0.197 | 0.236 | 0.268 | 0.165 | 0.264 | 0.295 | 0.213 | 197.8 |
| X11 lower | 0.256 | 0.248 | 0.177 | 0.248 | 0.260 | 0.181 | 0.252 | 0.260 | 0.161 | 0.264 | 0.287 | 0.220 | did not fail |
| X12 upper | 0.220 | 0.244 | 0.169 | 0.236 | 0.228 | 0.161 | 0.173 | 0.224 | 0.130 | 0.205 | 0.244 | 0.154 | 196.3 |
| X12 lower | 0.236 | 0.228 | 0.161 | 0.248 | 0.240 | 0.173 | 0.165 | 0.244 | 0.130 | 0.220 | 0.248 | 0.169 | did not fail |

Note: “upper” and “lower” denote the upper or the lower branch members.

## EXPERIMENTS ON HSS WELDS

Two large-scale, 39.4-ft (12.0-m) and 40.0-ft (12.2-m) span, simply supported, fillet-welded, rectangular HSS Warren trusses, comprised of 60° gapped and overlapped K-connections, were tested by Frater and Packer (1992a, 1992b). Quasi-static loading was performed in a carefully controlled manner (Figure 1a) to produce sequential failure of the tension-loaded, fillet-welded joints (rather than connection failures). Measured geometric properties of these two trusses are given in Table 1, along with measured mechanical properties of both the (as-deposited) weld metal and HSS members. In addition, a series of weld-critical tests have been performed (Packer and Cassidy, 1995) on four T-connections and 12 cross- (or X-) connections, with the branches loaded in quasi-static tension. The measured geometric and mechanical properties of these test specimens are given in Table 2.

Weld sizes in all of the preceding connections were determined by making a negative mold of each test specimen and then measuring the leg sizes and throat sizes of each weld at multiple points. The effective leg sizes of the welds, measured along the branch member and chord member, respectively ($w_b$ and $w_c$), plus the throat sizes ($t_w$), are listed in

| Table 5. Values of $\phi$, $\Omega$, $F_{nBM}$, $F_{nw}$, $A_{BM}$ and $A_{we}$ for Fillet Welds | | | | |
|---|---|---|---|---|
| **Load Type** | **Pertinent Metal** | **$\phi$ and $\Omega$** | **$F_{nBM}$ or $F_{nw}$** | **$A_{BM}$ or $A_{we}$** |
| Shear | Base metal | $\phi = 0.75$<br>$\Omega = 2.00$ | $F_{nBM} = 0.60F_{ub}$ or<br>$F_{nBM} = 0.60F_u$ | $A_{BM} = L_e w_b$ or<br>$A_{BM} = L_e w_c$ |
| | Weld metal | $\phi = 0.75$<br>$\Omega = 2.00$ | $F_{nw} = 0.60F_{EXX}$ | $A_{we} = L_e t_w$ |

Tables 3 and 4. Each separate weld size is reported as an average for weld 1, weld 2, weld 3 and weld 4, according to the notation in Figure 4. Tables 3 and 4 also list the failure loads of all welded joints, which are subsequently used to evaluate nominal weld strengths and predicted weld design strengths according to the 2010 AISC *Specification*.

## WELD DESIGN PROCEDURE TO AISC 2010 SPECIFICATION

Section J2.4 of the AISC *Specification* (AISC, 2010) requires that the design strength, $\phi R_n$, and the allowable strength, $R_n/\Omega$, of welded joints be the lower value of the base material strength (determined according to the limit states of tensile rupture and shear rupture) and the weld metal strength (determined according to the limit state of rupture) as follows:

For the base metal

$$R_n = F_{nBM} A_{BM} \tag{15a}$$

For the weld metal

$$R_n = F_{nw} A_{we} \tag{15b}$$

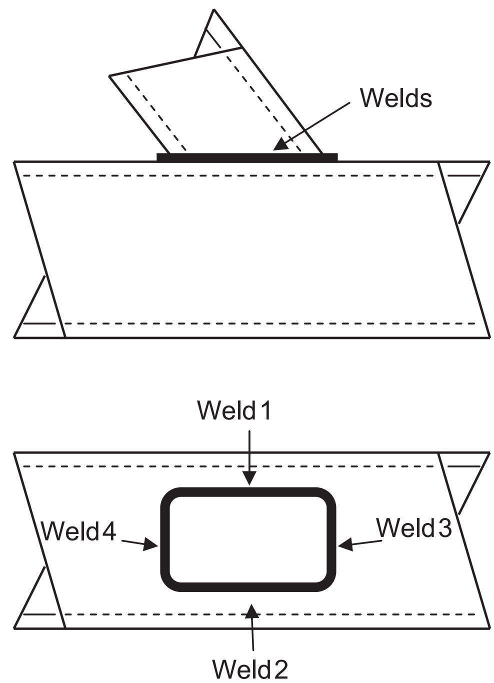



*Fig. 4. Weld notation.*

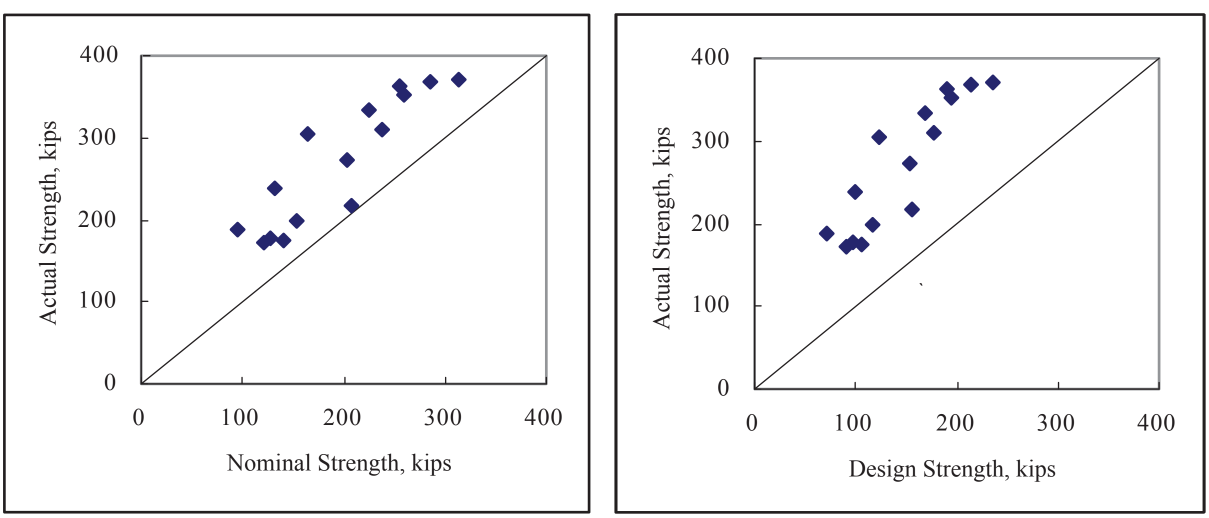



*Fig. 5. Correlation with test results for gapped K-connection test specimens, without inclusion of the (1.00 + sin$^{1.5}$ θ) term: (a) actual strength vs. predicted nominal strength; (b) actual strength vs. predicted LRFD design strength.*

| Table 6. Actual Strengths, Nominal Strengths ($R_n$ ) and Predicted Resistances ($\phi R_n$ ) of Welded Joints in Gapped K-Connection Test Specimens, without Inclusion of the $(1.00 + \sin^{1.5}\theta)$ Term | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Connection Number | $R_{nbb}$ (kips) | $R_{nbc}$ (kips) | $R_{nw}$ (kips) | $R_n$ (kips) | $\phi R_n$ (kips) | Actual (kips) | Actual/$R_n$ | Actual/ ($\phi R_n$) |
| K1 | 142.9 | 154.8 | 121.5 | 121.5 | 91.2 | 171.1 | 1.407 | 1.877 |
| K2 | 140.5 | 172.2 | 128.6 | 128.6 | 96.4 | 177.6 | 1.381 | 1.842 |
| K3 | 155.5 | 195.3 | 141.4 | 141.4 | 106.1 | 175.8 | 1.243 | 1.657 |
| K4 | 172.7 | 191.6 | 154.3 | 154.3 | 115.7 | 198.9 | 1.289 | 1.719 |
| K5 | 252.0 | 272.9 | 206.9 | 206.9 | 155.2 | 217.4 | 1.051 | 1.401 |
| K6 | 247.8 | 261.3 | 203.5 | 203.5 | 152.6 | 271.8 | 1.336 | 1.781 |
| K7 | 328.9 | 287.5 | 254.6 | 254.6 | 191.0 | 362.6 | 1.424 | 1.899 |
| K8 | 308.3 | 304.0 | 260.5 | 260.5 | 195.3 | 353.4 | 1.357 | 1.809 |
| K9 | 353.7 | 350.4 | 285.6 | 285.6 | 214.2 | 368.0 | 1.288 | 1.718 |
| K10 | 140.1 | 192.1 | 95.7 | 95.7 | 71.8 | 186.8 | 1.952 | 2.603 |
| K11 | 192.1 | 273.3 | 131.6 | 131.6 | 98.7 | 239.0 | 1.815 | 2.420 |
| K12 | 238.1 | 291.8 | 164.2 | 164.2 | 123.2 | 304.2 | 1.852 | 2.470 |
| K13 | 320.5 | 403.2 | 224.0 | 224.0 | 168.0 | 333.4 | 1.488 | 1.984 |
| K14 | 346.1 | 406.2 | 236.9 | 236.9 | 177.7 | 308.9 | 1.304 | 1.738 |
| K15 | 463.4 | 543.4 | 314.1 | 314.1 | 235.6 | 369.6 | 1.177 | 1.569 |

The values of $\phi$, $\Omega$, $F_{nBM}$, $F_{nw}$, $A_{BM}$ and $A_{we}$ for different types of welds are given in Table J2.5 of the AISC *Specification* (AISC, 2010). Because all the welded joints in the HSS connection test specimens listed in Tables 1 through 4 were considered to be fillet-welded and loaded in shear, the part for fillet welds is abstracted from Table J2.5 and Section J4 and is given for both Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) in Table 5.

It should be noted that the LRFD weld and base metal resistances for the AISC *Specification* (AISC, 2010), determined using Equations 15a and 15b (in conjunction with Table 5), are the same as the Limit States Design (LSD) weld and base metal resistances calculated using CSA (2009), because the $\phi F_{nw}$ terms ($\phi$ = 0.75)(0.60$F_{EXX}$) and ($\phi$ = 0.67)(0.67$F_{EXX}$) both equal 0.45$F_{EXX}$. Thus, the use of either AISC or CSA formulations results in the same design outcome.

## EVALUATION OF AISC 2010 *SPECIFICATION* WITH EXPERIMENTS ON HSS WELDS

Table 5 and Equations 6, 9, 14 and 15 were used to calculate the nominal strengths (excluding the resistance factor) of the 31 welded joints in Tables 1 through 4. Although weld metal strength may be expected to typically govern in a design situation, the analysis of laboratory tests—where weld and base metal strengths vary considerably—requires the strength calculation for both weld metal failure and base metal failure. The predicted strength of each welded joint, without consideration of the $(1.00 + \sin^{1.5}\theta)$ effect, was determined by the summation of the strengths of the four separate welds (see Figure 4)—that together add up to the perimeter around the branch footprint—and is given as a predicted nominal strength, $R_n$, in Table 6 (for gapped K-connections) and Table 7 (for T- and X-connections). The predicted nominal strength, $R_n$, is taken as the lowest of the values for the branch base metal ($R_{nbb}$ ), the chord base metal ($R_{nbc}$ ) and the weld metal ($R_{nw}$ ). Here, $R_{nw}$ governed over $R_{nbb}$ and $R_{nbc}$ in all cases. These tables also show the predicted welded joint design strength, $\phi R_n$, using $\phi$ = 0.75.

The correlations between the actual strengths and predicted nominal strengths, and between the actual strengths and predicted design strengths, are shown in Figure 5 (for gapped K-connection test specimens) and Figure 6 (for T-and X-connection test specimens). For gapped K-connections, the Mean of the (Actual/$R_n$) ratios given in Table 6, and shown in Figure 5a, is 1.424 with a coefficient of variation (COV) of 0.180. For T- and X-connections, the mean of the (actual/$R_n$ ) ratios given in Table 7, and shown in Figure 6a, is 1.153 with a COV of 0.136.

To assess whether adequate, or excessive, safety margins are inherent in the correlations shown in Figures 5a and 6a, one can check to ensure that a minimum safety index of $\beta^+$ = 4.0 (per Chapter B of the AISC *Specification* Commentary) is achieved, using a simplified reliability analysis in which the resistance factor $\phi$ is given by (Fisher et al., 1978; Ravindra and Galambos, 1978):

$$\phi = m_R \exp(-\alpha\beta^+ \text{COV}) \tag{16}$$

| Table 7. Actual Strengths, Nominal Strengths ($R_n$) and Predicted Resistances ($\phi R_n$) of Welded Joints in T- and X-Connection Test Specimens, without Inclusion of the ($1.00 + \sin^{1.5}\theta$) Term | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Connection Number | $R_{nbb}$ (kips) | $R_{nbc}$ (kips) | $R_{nw}$ (kips) | $R_n$ (kips) | $\phi R_n$ (kips) | Actual (kips) | Actual/ $R_n$ | Actual/ ($\phi R_n$) |
| T1 | 147.6 | 127.2 | 100.8 | 100.8 | 75.6 | 118.5 | 1.175 | 1.567 |
| T2 | 173.5 | 152.1 | 122.9 | 122.9 | 92.2 | 154.4 | 1.257 | 1.675 |
| T3 | 158.3 | 160.3 | 134.7 | 134.7 | 101.0 | 203.9 | 1.514 | 2.018 |
| T4 | 199.2 | 214.4 | 177.2 | 177.2 | 132.9 | 195.1 | 1.101 | 1.468 |
| X1 upper | 352.4 | 268.7 | 235.5 | 235.5 | 176.6 | did not fail | — | — |
| X1 lower | 300.7 | 273.9 | 216.4 | 216.4 | 162.3 | 310.2 | 1.434 | 1.912 |
| X2 upper | 224.0 | 197.4 | 162.2 | 162.2 | 121.6 | did not fail | — | — |
| X2 lower | 255.1 | 210.8 | 181.0 | 181.0 | 135.8 | 200.7 | 1.109 | 1.479 |
| X3 upper | 180.4 | 160.5 | 131.3 | 131.3 | 98.5 | 149.0 | 1.135 | 1.514 |
| X3 lower | 176.3 | 160.6 | 122.7 | 122.7 | 92.0 | did not fail | — | — |
| X4 upper | 150.3 | 141.4 | 112.6 | 112.6 | 84.4 | 104.3 | 0.926 | 1.235 |
| X4 lower | 160.6 | 146.9 | 115.7 | 115.7 | 86.8 | did not fail | — | — |
| X5 upper | 149.7 | 127.1 | 100.0 | 100.0 | 75.0 | did not fail | — | — |
| X5 lower | 128.9 | 123.6 | 96.5 | 96.5 | 72.4 | 95.3 | 0.987 | 1.317 |
| X6 upper | 131.6 | 108.7 | 90.9 | 90.9 | 68.2 | 96.9 | 1.066 | 1.421 |
| X6 lower | 137.8 | 118.5 | 97.9 | 97.9 | 73.4 | did not fail | — | — |
| X7 upper | 510.2 | 476.1 | 461.0 | 461.0 | 345.8 | did not fail | — | — |
| X7 lower | 489.7 | 482.1 | 444.0 | 444.0 | 333.0 | 571.2 | 1.286 | 1.715 |
| X8 upper | 388.4 | 362.8 | 348.7 | 348.7 | 261.6 | 380.4 | 1.091 | 1.454 |
| X8 lower | 382.5 | 360.8 | 340.7 | 340.7 | 255.5 | did not fail | — | — |
| X9 upper | 337.6 | 311.5 | 297.6 | 297.6 | 223.2 | did not fail | — | — |
| X9 lower | 338.0 | 320.1 | 305.1 | 305.1 | 228.8 | 310.2 | 1.017 | 1.356 |
| X10 upper | 209.9 | 204.7 | 190.2 | 190.2 | 142.6 | did not fail | — | — |
| X10 lower | 211.6 | 221.4 | 201.0 | 201.0 | 150.8 | 218.5 | 1.087 | 1.449 |
| X11 upper | 203.4 | 206.3 | 188.1 | 188.1 | 141.1 | 197.8 | 1.052 | 1.402 |
| X11 lower | 211.3 | 197.8 | 188.5 | 188.5 | 141.4 | did not fail | — | — |
| X12 upper | 177.5 | 174.4 | 161.0 | 161.0 | 120.7 | 196.3 | 1.219 | 1.625 |
| X12 lower | 187.1 | 174.8 | 164.1 | 164.1 | 123.1 | did not fail | — | — |
| Note: "upper" and "lower" denote the upper or the lower branch members. | | | | | | | | |

where $m_R$ = mean of the ratio: (actual element strength)/(nominal element strength = $R_n$); COV = associated coefficient of variation of this ratio; and $\alpha$ = coefficient of separation taken to be 0.55 (Ravindra and Galambos, 1978). Equation 16 neglects variations in material properties, geometric parameters and fabrication effects, relying solely on the so-called professional factor. In the absence of reliable statistical data related to welds, this is believed to be a conservative approach. Application of Equation 16 produced $\phi$ = 0.959 for welded joints in gapped K-connections and $\phi$ = 0.855 for T- and X-connections. Because both of these exceed 0.75, the effective weld length concepts advocated in Section K4 of the *Specification* can, on the basis of the available experimental evidence, be deemed adequately conservative.

## Introduction of the ($1.00 + 0.50 \sin^{1.5}\theta$) Term

The AISC *Specification* (AISC, 2010) and the recent Canadian *Standard* (CSA, 2009)—like their preceding editions—permit a greater available strength for a fillet weld, when loaded at an angle of $\theta$ relative to the longitudinal axis of the weld, to be taken into account in weld design, where applicable, by using Equation 17. Conservatively, one is allowed to set the function $(1.00 + 0.50 \sin^{1.5} \theta)$ to unity.

$$F_{nw} = 0.60\, F_{EXX}\, (1.00 + 0.50 \sin^{1.5} \theta) \tag{17}$$

The AISC *Specification* (AISC, 2010) notes, in Section J2.4, that Equation 17 is applicable only for fillet welds in a weld group that:

1. Is linear (a group in which all elements are in a line or are parallel), with a uniform leg size, and loaded through the center of gravity; or
2. Is loaded and analyzed using an instantaneous center of rotation method.

Thus, the strength enhancement of Equation 17 is not deemed appropriate for the simple design method for fillet welds to HSS branches, as described herein. This policy is also evident in the recent AISC *Design Guide* on HSS connections (Packer et al., 2010). The effect of including the $\sin \theta$ effect in fillet weld design formulas for rectangular HSS-to-HSS connections was last considered by Packer (1995), but since that time the AISC *Specification* design equations for weld effective length have evolved, as explained previously. If this $\sin \theta$ effect is taken into consideration for all branch fillet welds 1 through 4 (see Figure 4) in the analysis of the data presented in this paper, the statistical outcomes change to:

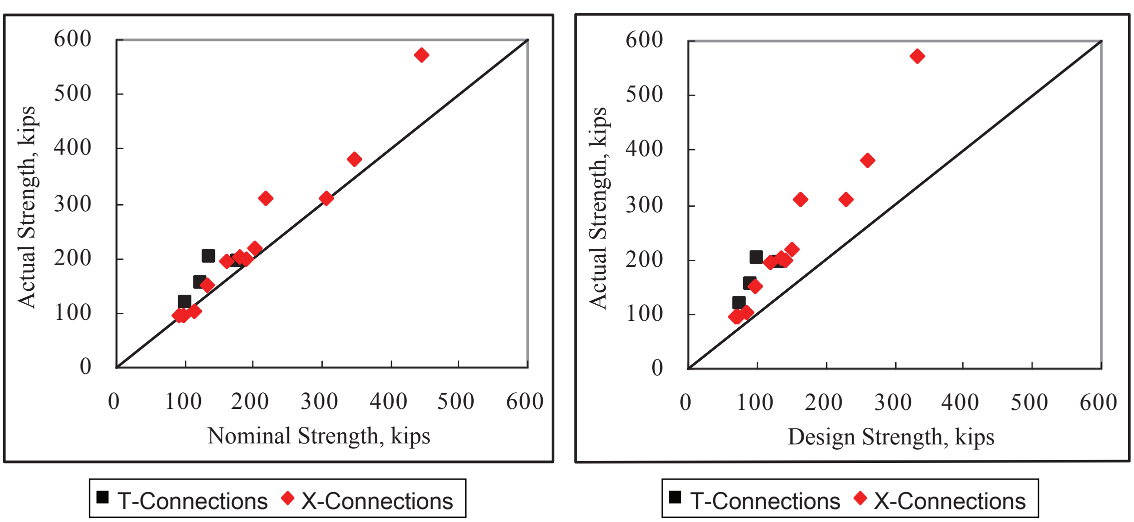
- For gapped K-connections: $m_R = 1.127$, COV = 0.145 and $\phi = 0.819$ (using Equation 16 with $\beta^+ = 4.0$).
- For T- and X-connections: $m_R = 0.998$, COV = 0.142 and $\phi = 0.730$ (using Equation 16 with $\beta^+ = 4.0$).

## CONCLUSIONS

Design guides or specifications/codes requiring the welds to develop the yield capacity of the branch members produce a conservative yet easy design process, but they may lead to extra cost due to weld overdesign. Alternate design methods that consider an effective weld length have the potential to provide a relatively smaller weld size, thus achieving a more economical weld design.

By comparing the actual strengths of fillet-welded joints in weld-critical T-, cross- (X-) and gapped K-connection specimens to their predicted nominal strengths and predicted design strengths, it has been shown that the relevant effective-length design formulas in the AISC *Specification* Section K4 (AISC, 2010)—without use of the $(1.00 + 0.50 \sin^{1.5} \theta)$ term for fillet welds—result in an appropriate weld design with an adequate safety level. The effective-length design formulas for gapped K-connections, even with the $(1.00 + 0.50 \sin^{1.5} \theta)$ term included for fillet welds, were also shown to be conservative. However, the effective-length formulas for T- and cross- (X-) connections, with the $(1.00 + 0.50 \sin^{1.5} \theta)$ term included for fillet welds, currently result in an inadequate reliability index.

A limitation of this study is that all test specimens were under predominantly axial loads in the branches. However, the weld effective length formulas for T-, Y- and cross- (X-) connections in *Specification* Table K4.1 (AISC, 2010) also address branch bending. The available test data do not provide an opportunity to evaluate the accuracy of formulas applicable to branch bending loads. Thus, the general use of the $(1.00 + 0.50 \sin^{1.5} \theta)$ strength enhancement term for fillet welds in HSS-to-HSS connections is not recommended.




(*a*) (*b*)

*Fig. 6. Correlation with test results for T- and X-connection test specimens, without inclusion of the $(1.00 + \sin^{1.5}\theta)$ term: (a) actual strength vs. predicted nominal strength; (b) actual strength vs. predicted LRFD design strength.*

## DESIGN EXAMPLES

Two design examples follow, using square and rectangular ASTM A500 HSS (ASTM, 2010). These examples illustrate the *Specification* Section K4 (AISC, 2010) weld design method for square and rectangular HSS, with the examples performed in both LRFD and ASD and presented to AISC format (Packer et al., 2010).

### Design Example for a T-Connection

**Given:**

Determine the weld size to connect the square HSS branch member to the square HSS chord member to resist the applied load as shown in Figure 7. The dead and live loads on the branch member are $P_D$ = 20 kips and $P_L$ = 20 kips.

The material properties are as follows:
HSS chord member: ASTM A500 Grade B steel, $F_y$ = 46 ksi, $F_u$ = 58 ksi
HSS branch member: ASTM A500 Grade B steel, $F_{yb}$ = 46 ksi, $F_{ub}$ = 58 ksi
Electrode E70: $F_{E70}$ = 70 ksi

The geometric properties are as follows:
HSS 10 × 10 × ½: $H = B$ = 10 in., $t$ = 0.465 in.
HSS 5 × 5 × ⅜: $H_b = B_b$ = 5 in., $t_b$ = 0.349 in.
Branch member inclination angle: $\theta$ = 90°

**Solution:**

*Calculate the required strength*

| LRFD | ASD |
|---|---|
| $P_r$ = 1.2(20 kips) + 1.6(20 kips)<br>= 56 kips | $P_r$ = 20 kips + 20 kips<br>= 40 kips |

The T-connection in the design example meets all the limits of applicability of Table K2.2A in the *Specification* (AISC, 2010). The available axial strength of the T-connection, determined by applying the formulas in Table K2.2, also exceeds the required strength.

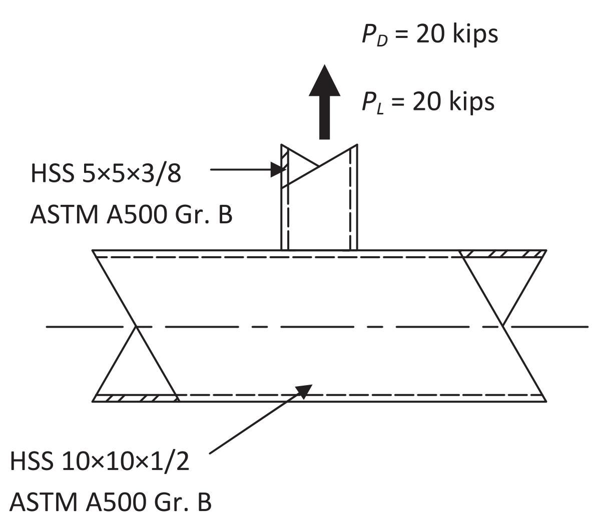



*Fig. 7. Design example for a T-connection.*

*Calculate the effective weld length*

Apply Equation 9 to determine the effective width of the branch weld transverse to the chord:

$$
\begin{aligned}
b_{eoi} &= \frac{10}{B/t}\left(\frac{F_y t}{F_{yb} t_b}\right)B_b \\
&= \frac{10}{10\text{ in.}/0.465\text{ in.}}\left(\frac{46\text{ ksi}\left(0.465\text{ in.}\right)}{46\text{ ksi}\left(0.349\text{ in.}\right)}\right)\left(5\text{ in.}\right) \\
&= 3.10\text{ in.} \leq B_b = 5\text{ in.}
\end{aligned}
$$

Apply Equation 6 to determine the effective weld length for resisting the applied load:

$$L_e = \frac{2H_b}{\sin\theta} + 2b_{eoi} = \frac{2(5\text{ in.})}{\sin 90°} + 2(3.10\text{ in.}) = 16.20\text{ in.}$$

*Determine the required weld size*

Section J2.4 of the *Specification* (AISC, 2010) requires the design strength, $\phi R_n$, and the allowable strength, $R_n/\Omega$, of welded joints to be the lower value of the base material strength (determined according to the limit states of tensile rupture and shear rupture) and the weld metal strength (determined according to the limit state of rupture). In other words, $R_n$ shall be the least value among the nominal strength of the branch member base metal ($R_{nbb}$), the nominal strength of the chord member base metal ($R_{nbc}$) and the nominal strength of the weld metal ($R_{nw}$).

Let $w_b = w_c = w$, where $w$ is the required weld size (leg).

- For the branch member base metal

  $R_{nbb} = F_{nBM} A_{BM}$ — Equation 15a

  $= (0.6F_{ub})(L_e w_b)$ — Table 5

  $= (0.6 \times 58\text{ ksi})(16.20\text{ in.} \times w)$

  $= 563.8w$ kips

- For the chord member base metal

  $R_{nbc} = F_{nBM} A_{BM}$ — Equation 15a

  $= (0.6F_u)(L_e w_c)$ — Table 5

  $= (0.6 \times 58\text{ ksi})(16.20\text{ in.} \times w)$

  $= 563.8w$ kips

- For the weld metal

  For all four sides of weld, $t_w = w\sqrt{2}$, as all welds are 90° fillets.

  $R_{nw} = F_{nw} A_{we}$ — Equation 15b

  $= (0.6F_{EXX})(L_e t_w)$ — Table 5

  $= (0.6 \times 70\text{ ksi})(16.20\text{ in.} \times w\sqrt{2})$

  $= 481.1w$ kips

Thus, $R_n = \min R_{nbb}, R_{nbc}, R_{nw} = \min 563.8w$ kips, $563.8w$ kips, $481.1w$ kips $= 481.1w$ kips. In other words, the weld metal controls, as is expected, even considering all base metals as loaded in shear.

| LRFD | ASD |
|---|---|
| $\phi = 0.75$<br>$\phi R_n \geq P_r = 56$ kips<br>$0.75(481.1w \text{ kips}) \geq 56$ kips<br>$w \geq 0.155$ in.<br>Hence, use 3⁄16-in. weld. | $\Omega = 2.00$<br>$R_n/\Omega \geq P_r = 40$ kips<br>$481.1w \text{ kips} / 2.00 \geq 40$ kips<br>$w \geq 0.166$ in.<br>Hence, use 3⁄16-in. weld. |

The 3⁄16-in. weld is placed all around the joint. Check that fillet welding along the longitudinal sides of the branch is still possible on the "flat" of the HSS chord connecting face.

$$\text{Minimum width of HSS "flat"} \approx B - 2(3t)$$

$$= 10.0 \text{ in.} - 2(3 \times 0.465 \text{ in.})$$

$$= 7.21 \text{ in.} > 5.0 \text{ in.} + 2(\tfrac{3}{16} \text{ in.})$$

$$= 5.38 \text{ in.} \quad \textbf{o.k.}$$

**Design Example for a Gapped K-Connection**

**Given:**

Determine the weld size to connect the square HSS branch member to the rectangular HSS chord member to resist the applied loads as shown in Figure 8. The dead and live loads on each branch member are $P_D = 20$ kips and $P_L = 20$ kips.

The material properties are as follows:
HSS chord member: ASTM A500 Grade B steel, $F_y = 46$ ksi, $F_u = 58$ ksi
HSS branch member: ASTM A500 Grade B steel, $F_{yb} = 46$ ksi, $F_{ub} = 58$ ksi
Electrode E70: $F_{E70} = 70$ ksi

The geometric properties are as follows:
HSS 12 × 8 × ½: $H = 12$ in., $B = 8$ in., $t = 0.465$ in.
HSS 4 × 4 × 3⁄8: $H_b = B_b = 4$ in., $t_b = 0.349$ in.
Branch member inclination angle: $\theta = 60°$

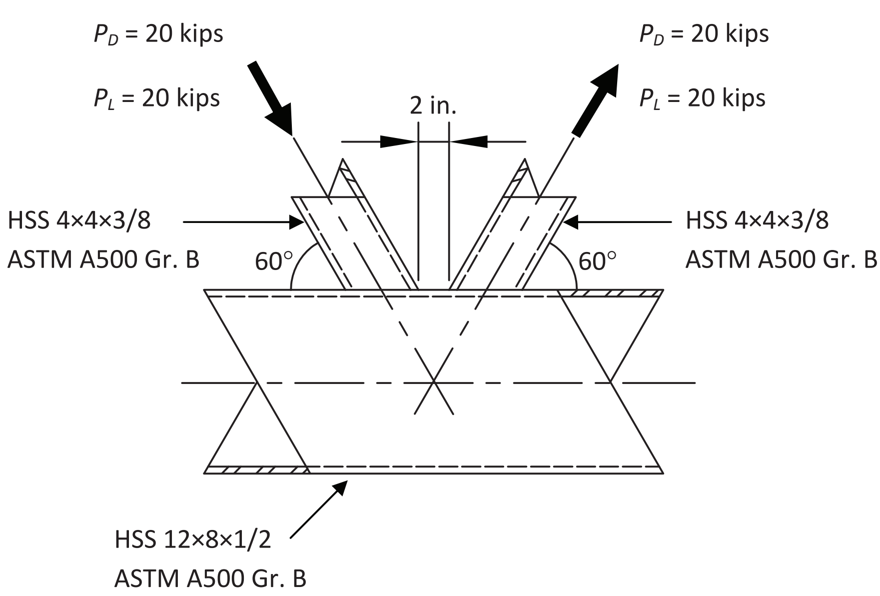



*Fig. 8. Design example for a gapped K-connection.*

**Solution:**

*Calculate the required strength*

| LRFD | ASD |
|---|---|
| $P_r = 1.2(20 \text{ kips}) + 1.6(20 \text{ kips})$<br>$= 56 \text{ kips}$ | $P_r = 20 \text{ kips} + 20 \text{ kips}$<br>$= 40 \text{ kips}$ |

The gapped K-connection in the design example meets all the limits of applicability of Table K2.2A in the *Specification* (AISC, 2010). The available axial strength of the gapped K-connection, determined by applying the formulas in Table K2.2, also exceeds the required strength.

*Calculate the effective weld lengths*

Apply Equation 14b to determine the effective weld lengths for one branch member. For the two longitudinal side welds:

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} = \frac{2\left[4 \text{ in.} - (1.2)(0.349 \text{ in.})\right]}{\sin 60°} = 8.27 \text{ in.}$$

For the transverse toe weld:

$$L_e = (B_b - 1.2t_b) = (4 \text{ in.} - 1.2 \times 0.349 \text{ in.}) = 3.58 \text{ in.}$$

Note that this effective length assumes that the weld at the heels of both branches is totally ineffective, at this branch angle.

*Determine the required weld size*

Section J2.4 of the *Specification* (AISC, 2010) requires the design strength, $\phi R_n$, and the allowable strength, $R_n/\Omega$, of welded joints to be the lower value of the base material strength (determined according to the limit states of tensile rupture and shear rupture) and the weld metal strength (determined according to the limit state of rupture). In other words, $R_n$ shall be the least value among the nominal strength of the branch member base metal ($R_{nbb}$), the nominal strength of the chord member base metal ($R_{nbc}$) and the nominal strength of the weld metal ($R_{nw}$).

Assume that a common effective weld size is placed all around each branch. Try ¼-in. weld ($= w_b = w_c$) for weld 1 and weld 2 as shown in Figure 4.

- For weld 1 and weld 2 branch member base metal
  $R_{nbb} = F_{nBM} A_{BM}$ — Equation 15a
  $= (0.6F_{ub})(L_e w_b)$ — Table 5
  $= (0.6 \times 58 \text{ ksi})(8.27 \text{ in.} \times 0.25 \text{ in.})$
  $= 71.9 \text{ kips}$

- For weld 1 and weld 2 chord member base metal
  $R_{nbc} = F_{nBM} A_{BM}$ — Equation 15a
  $= (0.6F_u)(L_e w_c)$ — Table 5
  $= (0.6 \times 58 \text{ ksi})(8.27 \text{ in.} \times 0.25 \text{ in.})$
  $= 71.9 \text{ kips}$

- For weld 1 and weld 2 weld metal
  For weld 1 and weld 2 (see Figure 4), the angle between the branch face and the chord face is 90°, so $t_w = w\sqrt{2}$ $= 0.177$ in.
  $R_{nw} = F_{nw} A_{we}$ — Equation 15b
  $= (0.6F_{EXX})(L_e t_w)$ — Table 5
  $= (0.6 \times 70 \text{ ksi})(8.27 \text{ in.} \times 0.177 \text{ in.})$
  $= 61.5 \text{ kips}$

Thus, $R_n$ = min $R_{nbb}$, $R_{nbc}$, $R_{nw}$ = min 71.9 kips, 71.9 kips, 61.5 kips = 61.5 kips for (weld 1 + weld 2). In other words, the weld metal controls, as is expected, even considering all base metals as loaded in shear.

Now consider weld 3 (see Figure 4). To maintain a weld effective throat of $t_w$ = 0.177 in., $w_b = w_c$ = 0.354 in. for weld 3.

- For weld 3 branch member base metal

$$
\begin{aligned}
R_{nbb} &= F_{nBM} A_{BM} && \text{Equation 15a}\\
&= (0.6F_{ub})(L_e w_b) && \text{Table 5}\\
&= (0.6 \times 58 \text{ ksi})(3.58 \text{ in.} \times 0.354 \text{ in.})\\
&= 44.1 \text{ kips}
\end{aligned}
$$

- For weld 3 chord member base metal

$$
\begin{aligned}
R_{nbc} &= F_{nBM} A_{BM} && \text{Equation 15a}\\
&= (0.6F_{u})(L_e w_c) && \text{Table 5}\\
&= (0.6 \times 58 \text{ ksi})(3.58 \text{ in.} \times 0.354 \text{ in.})\\
&= 44.1 \text{ kips}
\end{aligned}
$$

- For weld 3 weld metal

$$
\begin{aligned}
R_{nw} &= F_{nw} A_{we} && \text{Equation 15b}\\
&= (0.6F_{EXX})(L_e t_w) && \text{Table 5}\\
&= (0.6 \times 70 \text{ ksi})(3.58 \text{ in.} \times 0.177 \text{ in.})\\
&= 26.6 \text{ kips}
\end{aligned}
$$

Thus, $R_n$ = min $R_{nbb}$, $R_{nbc}$, $R_{nw}$ = min 44.1 kips, 44.1 kips, 26.6 kips = 26.6 kips for weld 3. In other words, the weld metal controls, as is expected, even considering all base metals as loaded in shear.

Thus, for (weld 1 + weld 2 + weld 3), $R_n$ = 61.5 kips + 26.6 kips = 88.1 kips. Note that when $\theta \geq 60°$, weld 4 at the branch heel is not effective in resisting the applied load according to Equation 14b.

| LRFD | ASD |
|---|---|
| $\phi = 0.75$<br>$\phi R_n = 0.75(88.1 \text{ kips}) = 66.1$ kips<br>$P_r = 56 \text{ kips} < \phi R_n$ **o.k.**<br>Hence, use ¼-in. weld. | $\Omega = 2.00$<br>$R_n/\Omega = 88.1 \text{ kips}/2.00 = 44.1$ kips<br>$P_r = 40 \text{ kips} < R_n/\Omega$ **o.k.**<br>Hence, use ¼-in. weld. |

The fillet weld is placed all around the joint. Calculations can sometimes lead to slightly different answers for LRFD and ASD, depending on the live-to-dead load ratio. LRFD and ASD procedures are calibrated to yield the same result for a live-to-dead load ratio of 3:1. Check that the fillet welding along the longitudinal sides of the branch is still possible on the "flat" of the HSS chord connecting face.

$$
\begin{aligned}
\text{Minimum width of HSS "flat"} &\approx B - 2(3t)\\
&= 8.0 \text{ in.} - 2(3 \times 0.465 \text{ in.})\\
&= 5.21 \text{ in.} > 4.0 \text{ in.} + 2(\tfrac{1}{4} \text{ in.})\\
&= 4.5 \text{ in.} \quad \textbf{o.k.}
\end{aligned}
$$

## SYMBOLS

$A_{BM}$ = effective area of the base metal, in.$^2$

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

$B$ = overall width of rectangular HSS chord member, measured 90° to the plane of the connection, in.

$B_b$ = overall width of rectangular HSS branch member, measured 90° to the plane of the connection, in.

$F_{EXX}$ = filler metal classification strength, ksi

$F_{nBM}$ = nominal stress of the base metal, ksi

$F_{nw}$ = nominal stress of the weld metal, ksi

$F_u$ = tensile strength of the HSS chord member material, ksi

$F_{ub}$ = tensile strength of the HSS branch member material, ksi

$F_{uw}$ = tensile strength of the weld metal, ksi

$F_y$ = yield strength of the HSS chord member material, ksi

$F_{yb}$ = yield strength of the HSS branch member material, ksi

$H$ = overall height of rectangular HSS chord member, measured in the plane of the connection, in.

$H_b$ = overall height of rectangular HSS branch member, measured in the plane of the connection, in.

$I_{ip}$ = weld effective moment of inertia for in-plane bending, in.$^4$

$I_{op}$ = weld effective moment of inertia for out-of-plane bending, in.$^4$

$L_e$ = effective length of groove and fillet welds to rectangular HSS, for weld strength calculations, in.

$M_{ip}$ = in-plane bending moment, kip-in.

$M_{op}$ = out-of-plane bending moment, kip-in.

$P_r$ = required strength, kips

$R_n$ = nominal strength of the welded joint (the least value of $R_{nbb}$, $R_{nbc}$ and $R_{nw}$), kips

$R_{nbb}$ = nominal strength of the branch member base metal, kips

$R_{nbc}$ = nominal strength of the chord member base metal, kips

$R_{nw}$ = nominal strength of the weld metal, kips

$S_{ip}$ = weld effective elastic section modulus for in-plane bending, in.$^3$

$S_{op}$ = weld effective elastic section modulus for out-of-plane bending, in.$^3$

$b_{eoi}$ = effective width of the transverse branch face welded to the chord, in.

$m_{\mathrm{R}}$ = mean of ratio: (actual element strength)/(nominal element strength) = professional factor

$t$ = wall thickness of HSS chord member, in.

$t_b$ = wall thickness of HSS branch member, in.

$t_w$ = effective weld throat thickness, in.

$w_b$ = effective leg size of weld, measured along the branch member, in.

$w_c$ = effective leg size of weld, measured along the chord member, in.

$\alpha$ = separation factor = 0.55

$\beta$ = width ratio = the ratio of overall branch width to chord width for HSS connection

$\beta^+$ = safety (reliability) index for LRFD and LSD

$\theta$ = acute angle between the branch and chord, degrees; angle of loading measured from a weld longitudinal axis for fillet weld strength calculation, degrees

$\phi$ = resistance factor

$\Omega$ = safety factor

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