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**Weld Design for Hollow Structural Section Connections:
Application to Canadian Standards**

by

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Abstract

This article presents a comprehensive review of existing North American research on weld effective lengths for hollow structural section (HSS) connections. Data from 393 experiments and finite-element analyses is analyzed to determine the inherent reliability index (β^+) of existing/proposed AISC 360 formulae for weld effective properties in axially loaded rectangular hollow section (RHS) T-, Y- and X-connections, RHS gapped and overlapped K-connections, RHS moment-loaded T-connections, and circular hollow section T-, Y- and X-connections, when used in conjunction with CSA S16-19 Clause 13.13.4.3(a) for design of welds to the ends of HSS branches. Modifications to the formulae are proposed to achieve $\beta^+ \geq 4.0$ (the target reliability index for connectors according to Annex B.4 of CSA S16-19), and recommendations are made to facilitate a “fit-for-purpose” design approach for welds to HSS. These are proposed for the next scheduled revision of CSA W59.

Keywords

Steel structures, hollow structural sections, fillet welds, partial joint penetration groove welds, connections, joints, reliability, design.

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1. Introduction

It is well known that the flexibility of hollow structural section (HSS) connections results in the non-uniform loading of welds around a joint. Thus, historically, international design recommendations have required welds to HSS branches to develop the yield strength of the member wall to allow plastic stress redistribution in the weld and resist any arrangement of loads in the member (IIW 1989). This approach (Approach 1) is justifiable when there is low confidence in the design forces or if plastic stress redistribution is required in the connection; however, in most situations (where HSS branch members have low utilization ratios because their design is governed by connection strength), this approach is over-conservative. Moreover, it invariably results in large weld sizes.

Over the last 30 years, weld-critical tests (in which weld fracture was designed to occur) have been carried out on HSS connections and trusses (e.g. Frater and Packer 1992a,b; Packer and Cassidy 1995). These tests have led to the development of a more modern, “fit-for-purpose” approach to weld design for HSS connections (IIW 2012; ISO 2013). This approach (Approach 2) is based on designing the welds to resist the actual forces in the branch member (rather than to develop the yield strength of the branch member wall). This typically results in smaller, more economical weld sizes. With Approach 2, non-uniform loading of welds is considered by using weld effective lengths. Intuitively, weld effective lengths are a measure of the “effective” length of a weld in a connection. They hence provide a way for engineers to discount portions of the weld that do not contribute to the overall joint resistance. Both Approaches (1 and 2) are outlined in Canadian HSS design guidance (Packer and Henderson 1997), and now in CSA S16-19.

Weld effective lengths (and section moduli) have been recommended for axially loaded rectangular hollow section (RHS) T-, Y-, and X-connections, RHS gapped and overlapped K-connections, and RHS moment-loaded T-connections (Frater and Packer 1992a,b; Packer and Cassidy 1995; McFadden and Packer 2014; Tousignant and Packer 2015). These recommendations form the basis of weld design rules in AWS D1.1-15 (AWS 2015) (the American welding code) and AISC 360-16 (AISC 2016) (the American steel code) that are based on Approach 2. In fact, AISC 360-16 has an entire subchapter devoted to Approach 2 for RHS connections (Chapter K5: “Welds of plates and branches to RHS”). Recommendations have also been made for weld effective lengths in circular

53 hollow section (CHS) T-, Y- and X-connections – for connections designed in accordance with AISC 360 (AISC
54 2016) (Tousignant and Packer 2019a).

55 While methods to calculate weld effectiveness for RHS (i.e. RHS-to-RHS) connections exist in Canada
56 [in Packer and Henderson (1997) (as referenced in Part 3: “Connections and Tension Members” of the 11th edition
57 of the CISC Handbook) (CISC 2016)], many of these methods are substantially out-of-date, and their reliability
58 when used in conjunction with modern Canadian weld design provisions [e.g. Clause 13 of CSA S16-19 (CSA
59 2019)] is unclear.

60 This paper presents a comprehensive review of the experimental and finite-element (FE) research on weld
61 effective lengths for HSS connections, and provides recommendations, substantiated by reliability analyses, for a
62 modern, “fit-for-purpose” design approach (Approach 2) for welds in HSS connections for use with Canadian
63 Standards.

64 **2. Weld Effective Properties for RHS-to-RHS Connections**

65 **2.1. RHS-to-RHS Gapped K- and N-connections**

66 Tests on welds in HSS-to-HSS joints were first carried out by Frater and Packer (1992a,b) on welds in 10
67 isolated RHS-to-RHS “X-connections” (which had restraints to simulate gapped K-connections) (Fig. 1) and 15
68 gapped K-connections (in two large-scale, 12.2-m and 12.0-m span, simply supported Warren trusses) (e.g. Fig.
69 2). In these tests, combinations of fillet and partial joint penetration (PJP) groove welds in each connection were
70 tested to failure under quasi-static axial tension applied to the branch(es), either directly (as shown in Fig. 1) or by
71 a point load applied to a panel point along the top chord of the truss (as shown in Fig. 2).

72 To permit “sequential testing” of the welds in the K-connections in the truss, repair of the ruptured welds (or
73 failed connections) was undertaken after each unloading. Subsequent testing of an alternate connection was then
74 done by changing the location of the applied point load (to generate a different web member force distribution).
75 The “X-” and K-connection measured member geometries, material strengths, and ultimate tensile strengths (N_{lu}),
76 are summarized in Table 1, with the symbols used defined in the footnote.

77 Twenty-three of the 25 tests conducted by Frater and Packer (1992a,b) failed by weld rupture, as shown in
78 Fig. 3; however, in the final gapped K-connection test of each truss (T1-9 and T2-8), and in two isolated connection

tests (P3-20 and P3-25), the welds were so large such that they virtually remained intact (failure involved fracture of the RHS chord, or “connection failure”). These tests are denoted in Table 1 by a ‡ symbol next to the value of N_{lu} .

For each connection in Table 1, Frater and Packer (1992a,b) used sliced segments through a negative mould of the weld to generate 20 cross sections of the weld around each branch (five for each of the weld elements, a, b, c and d shown in Figs. 4a and b). Using these, Frater and Packer (1992a,b) measured the weld throat dimension (t_w) (shown on photographs of macro-etch specimens in Fig. 5) and the weld legs (for fillets) at discrete points along the weld perimeter. The average value of t_w for each weld element and the measured electrode tensile strength (X_u) for each connection are summarized in Table 2. Therein, t_w is taken as the height of the largest triangle that can be inscribed within the diagrammatic weld (e.g. see Fig. 5).

The weld failures observed by Frater and Packer (1992a,b) involved shear failure of the weld metal – branch-metal interface along the side and toe welds (elements a, b and c in Figs. 4a and b), which was speculated to have occurred through the heat-affected zone. Failure of the joint at the heel (element d) generally involved a combination of chord/branch member tearing plus weld rupture, but was likely a result of overloading (or “unzipping”) (AWS 2015) that occurred after the weld ruptured along the other three sides.

Based on this work, Frater and Packer (1992a,b) recommended that the welds along all four sides of the branch (elements a, b, c and d) should be considered effective when $\theta \leq 50^\circ$, and that the welds along the two sides and the toe of the branch (element a, b and c) should be considered effective when the branch-to-chord angle (θ) $\geq 60^\circ$. When $50^\circ < \theta < 60^\circ$, a linear interpolation was proposed. These recommendations were subsequently adopted in AISC 360-10 (AISC 2010) and its later edition, AISC 360-16 (AISC 2016), with reduced branch width and height dimensions, by subtracting $1.2t_f$ from each branch side, to account for rounded RHS branch corners.

In AISC 360-16 Table K5.1 (AISC 2016), the following is given for the weld effective length (l_e) in RHS-to-RHS gapped K- and N-connections under axial load:

When $\theta \leq 50^\circ$:

$$(1) \quad l_e = \frac{2(h_1 - 1.2t_f)}{\sin \theta} + 2(b_1 - 1.2t_f)$$

When $\theta \geq 60^\circ$:

$$(2) \quad l_e = \frac{2(h_1 - 1.2t_1)}{\sin \theta} + b_1 - 1.2t_1$$

When $50^\circ < \theta < 60^\circ$, linear interpolation between Eqs. (1) and (2) is used to determine l_e .

Eqs. (1) and (2) are believed to provide a safety/reliability index (β^+) ≥ 4.0 when they are used in conjunction with the provisions of AISC 360-16 Chapters J and K (AISC 2016), which do not permit the “directional strength-increase factor” ($1.00 + 0.5\sin^{1.5}\theta$) for fillet welds to the ends of RHS branches. [According to Chapter B of the AISC 360-16 Commentary (AISC 2016), $\beta^+ \geq 4.0$ is the target for connectors (welds and bolts) designed using the load and resistance factor design (LRFD) method].

For application to limit states design (LSD) in Canada, a higher target for welded joints ($\beta^+ \geq 4.5$) was cited in the informative Annex B.4 of CSA S16-14 (CSA 2014); however, this has been reduced to a target reliability index of 4.0 in Annex B.4 of CSA S16-19 (CSA 2019).

2.2. RHS-to-RHS T- Y- and X-connections

2.2.1. Under Branch Axial Load

Packer and Cassidy (1995) conducted 16 tests on fillet and PJP groove welds in isolated RHS-to-RHS T- and X-connections. Like those in the “X-” and K-connections, Packer and Cassidy’s (1995) tests were designed to be “weld-critical”.

Four T-connections and 12 X-connections were tested under quasi-static tension applied to the branch(es), in test set-ups like that shown in Fig. 6a (for the X-connections) and Fig. 6b (for the T-connections). As shown in Fig. 6b, for the T-connections, a special-purpose testing jig was used to apply a shear force to the chord to react the tension in the branch. Each of the specimens failed in a sudden manner, by fracture through the weld and/or the base metal (which still constituted a failure of the welded joint). The measured member geometries, material strengths, and ultimate tensile strengths for these tests are summarized in Table 3.

Like Frater and Packer (1992a,b), Packer and Cassidy (1995) determined t_w by cutting through and measuring a negative mould of each weld element normal to the weld axis, at four or five locations. The average values of t_w and X_u for these connections are summarized in Table 4.

By using strain gauges adjacent to the weld to measure the relative load around the branch, Packer and Cassidy (1995) found that more of the weld perimeter was effective for lower values of θ and lower branch-to-chord width

ratios ($\beta = b_1/b_0$). Based on this, Packer and Cassidy (1995) recommended that the welds to the sides of the branch (elements a and b in Fig. 4b) should be considered “fully effective”, and that the effective length of the transverse welds along the toe and heel of the joint (elements c and d) be estimated using an already-existing formula for the “effective width” (b_e) of an RHS member transverse element (Packer and Henderson 1992):

$$(3) \quad b_e = \left(\frac{10t_0}{b_0} \right) \left(\frac{F_{y0}t_0}{F_{y1}t_1} \right) b_1 \leq b_1$$

Despite having tested only RHS-to-RHS T- and X-connections, the observed nonuniform-loading trends were rationally extended to Y-connections. Later on, for simplicity, it was recommended that the weld along the sides and heel of the branch (elements a, b and d, in Fig. 4b) should be considered effective when $\theta \leq 50^\circ$, and that only the side welds (elements a and b) should be considered effective when $\theta \geq 60^\circ$. Again, a linear interpolation was recommended when $50^\circ < \theta < 60^\circ$; however, in AISC 360-10 (AISC 2010) [and subsequently in AISC 360-16 (AISC 2016)], “de-simplified” versions of these recommendations were adopted which reintroduced the effective width term for the transverse welds, given by Eq. (3). In AISC 360-10 (AISC 2010), the following equation for l_e in RHS-to-RHS T-, Y- and X-connections under axial load was given (AISC 360-10 Table K4.1) (AISC 2010):

$$(4) \quad l_e = \frac{2h_1}{\sin \theta} + 2b_e$$

where b_e is given by Eq. (3).

Additionally, when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ was not permitted to exceed $2t_0$. This “notwithstanding clause” was added to AISC 360-10 (AISC 2010) as a precaution to prevent “unzipping”.

The reliability index (i.e. β^+) of Eq. (4) in AISC 360-10 (AISC 2010) was later investigated by McFadden and Packer (2014), who found that the notwithstanding clause (“when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ shall not exceed $2t_0$ ”) was overly restrictive. It was therefore proposed that the clause be modified to “when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ shall not exceed $b_1/4$ ”, which generally permits a longer effective length for the transverse welds. In AISC 360-16 (AISC 2016), Eq. (4) for l_e in RHS-to-RHS T-, Y- and X-connections was carried forward to Table K5.1 subject to this new notwithstanding clause.

2.2.2. Under Branch Bending

McFadden and Packer (2014) conducted 12 tests on fillet and PJP groove welds in RHS-to-RHS moment connections (i.e. T-connections subjected to branch in-plane bending). The connections were tested by applying a quasi-static, lateral point load to the branch member, which was welded, all around, to a simply supported chord (see Fig. 7). Ten tests failed by weld rupture (as planned), and two failed by punching shear (i.e. rupture of the chord face) on the tension side of the connection (which occurred only after extensive plastification of the chord face). The measured member geometries, material strengths, and ultimate moment strengths (M_{lu}) of the connections tested by McFadden and Packer (2014) are summarized in Table 5, where F_u = tensile strength of the governing base metal (lower of the branch and the chord) for connections that contained PJP groove welds.

Unlike the two previous test programs (Frater and Packer 1992a,b; Packer and Cassidy 1995), weld sizes in the moment T-connections were measured after testing, using scanned macroetch specimens similar to those shown in Fig. 5. The macroetch specimens were prepared by cutting the joints normal to the longitudinal weld axis at numerous locations around the branch. Software with built-in measuring tools was then used to determine t_w for each weld element – average values of which (along with X_u for each connection) are summarized in Table 6.

McFadden and Packer (2014) found that the load on the weld around the branch was largest at the branch corners and, as expected [based on the previous test program by Packer and Cassidy (1995)], that the magnitude of the load along the transverse elements (c and d) decreased relative to that at the corners as β increased.

Prior to this research, AISC 360-10 (AISC 2010) contained speculative equations for the weld effective elastic section moduli for in-plane and out-of-plane bending (S_{ip} and S_{op} , respectively) in RHS-to-RHS moment T-, Y- and X-connections (i.e. while being based on informed knowledge of general HSS connection behaviour, the equations had not yet been substantiated by tests). The S_{ip} and S_{op} equations therein were derived simply, from Eq. (4), by calculating the weld effective section moduli that were implied (McFadden et al. 2013). The purpose of the research conducted by McFadden and Packer (2014) was hence to verify (and/or modify) the AISC 360-10 (AISC 2010) equations for S_{ip} and S_{op} given below to provide $\beta^+ \geq 4.0$.

In AISC 360-10 (AISC 2010), the following equations for S_{ip} and S_{op} in RHS-to-RHS T-, Y- and X-connections under branch bending were given (AISC 360-10 Table K4.1) (AISC 2010):

$$(5) \quad S_{ip} = \frac{t_w}{3} \left(\frac{h_1}{\sin \theta} \right)^2 + t_w b_e \left(\frac{h_1}{\sin \theta} \right)$$

$$(6) \quad S_{op} = t_w \left(\frac{h_1}{\sin \theta} \right) b_1 + \frac{t_w}{3} (b_1^2) - \frac{(t_w/3)(b_1 - b_e)^3}{b_1}$$

where b_e is given by Eq. (3).

As for the axially loaded T-, Y-, and X-connections, when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ was not permitted to exceed $2t_0$ to prevent “unzipping”.

Based on a reliability analysis of Eq. (5) pertaining to AISC 360-10 (AISC 2010), McFadden and Packer (2014) found that the notwithstanding clause (“when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ shall not exceed $2t_0$ ”) was again overly restrictive, and recommended that it be modified to “when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ shall not exceed $b_1/4$ ”. Eqs. (5) and (6) for S_{ip} and S_{op} in RHS-to-RHS T-, Y- and X-connections were hence carried forward to Table K5.1 of AISC 360-16 (AISC 2016) subject to this new notwithstanding clause.

The work by McFadden and Packer (2014) was recently extended using FE models of fillet welds in RHS-to-RHS moment T-connections under branch in-plane bending. Yaghoubshahi et al. (2019) used an equivalent-strain failure criterion to simulate weld fracture in the fillets, which was calibrated using a subset of the tests by McFadden and Packer (2014) (and validated by comparing the FE M_{Iw} , branch load-deflection responses, and spot-strain measurements to those of the experiments). A parametric study of 61 weld-critical moment T-connections [with $\theta = 90^\circ$; $15 \leq b_0/t_0 \leq 35$; $0.25 \leq \beta \leq 0.85$; and $0.20 \leq \tau (= t_1/t_0) \leq 1.00$] was performed, the results of which broadly supported McFadden and Packer’s (2014) recommendations. However, Yaghoubshahi et al. (2019) showed that, in some cases, bearing (between the branch and chord on the compression side of the connection) could help to increase the weld ultimate strength in moment T-connections and, in such cases, Eq. (5) (AISC 2016) is exceptionally conservative. A new S_{ip} equation for these cases, with a reduced range of applicability ($\beta \leq 0.85$), was given.

2.3. RHS-to-RHS Overlapped K- and N-Connections

Tousignant and Packer (2015) conducted nine tests on fillet and PJP welds in RHS-to-RHS overlapped K-connections in a large-scale, 10-m, simply supported Warren truss. Like the truss tests by Frater and Packer

(1992a,b), Tousignant and Packer (2015) sequentially tested welds around predetermined branch (or web) members in the truss. The test welds, in this case, were those to the overlapping branch members (member i in Fig. 8), which were loaded in tension during the tests. All nine of the connections tested by Tousignant and Packer (2015) failed by weld rupture (see Fig. 9); however, one of the joints (K-60-0.50 in Table 7) had an imperfect root detail along a single PJP groove weld element (i.e. element a'), which arose due to the backing bar bending inwards during fit-up.

Two tests on overlapped K-connections were also conducted by Frater and Packer (1992a,b) (within the 12.0-m span truss); however, due to having very large welds, both failed by a combination of weld fracture and RHS web yielding (followed by web wall fracture). The measured member geometries, material strengths, and ultimate tensile strengths of these 11 tests [nine by Tousignant and Packer (2015), denoted by "K" in the test number, and two by Frater and Packer (1992a,b) denoted by "T2" in the test number] are summarized in Table 7. New symbols used therein are defined in the footnote (also see Fig. 8).

Like the previous test programs by Frater and Packer (1992a,b) and Packer and Cassidy (1995), Tousignant and Packer (2015) used negative moulds of each weld element to determine t_w . The average values of t_w and X_u are summarized in Table 8.

At the time of this research, AISC 360-10 (AISC 2010) contained speculative equations for l_e in RHS-to-RHS overlapped K-connections (like those in RHS-to-RHS moment T-connections). The objective of the research was therefore to verify (and/or modify) these equations to provide $\beta^+ \geq 4.0$.

In AISC 360-10 (AISC 2010), the following equations for l_e in RHS-to-RHS overlapped K-connections were given (AISC 360-10 Table K4.1) (AISC 2010) for the overlapping branch (member i):

When $25\% \leq \text{overlap } (O_v) \leq 50\%$:

$$(7) \quad l_{e,i} = \frac{2O_v}{50} \left[\left(1 - \frac{O_v}{100} \right) \left(\frac{h_i}{\sin \theta_i} \right) + \frac{O_v}{100} \left(\frac{h_i}{\sin(\theta_i + \theta_j)} \right) \right] + b_{eoi} + b_{eov}$$

When $50\% < O_v < 80\%$:

$$(8) \quad l_{e,i} = 2 \left[\left(1 - \frac{O_v}{100} \right) \left(\frac{h_i}{\sin \theta_i} \right) + \frac{O_v}{100} \left(\frac{h_i}{\sin(\theta_i + \theta_j)} \right) \right] + b_{eoi} + b_{eov}$$

When $80\% \leq O_v \leq 100\%$:

$$(9) \quad l_{e,i} = 2 \left[\left(1 - \frac{O_v}{100} \right) \left(\frac{h_i}{\sin \theta_i} \right) + \frac{O_v}{100} \left(\frac{h_i}{\sin(\theta_i + \theta_j)} \right) \right] + b_i + b_{eov}$$

where $l_{e,i}$ = weld effective length for the overlapping branch (member i); b_{eoi} = effective length of the transverse weld at the heel of the overlapping branch; and b_{eov} = effective length of the transverse weld at the toe of the overlapping branch. The term b_{eoi} in Eqs. (7) and (8), and the term b_{eov} in Eqs. (7)-(9), were given as follows:

$$(10a) \quad b_{eoi} = \left(\frac{10t_0}{b_0} \right) \left(\frac{F_{y0}t_0}{F_{yi}t_i} \right) b_i \leq b_i$$

$$(10b) \quad b_{eov} = \left(\frac{10t_j}{b_j} \right) \left(\frac{F_{yj}t_j}{F_{yi}t_i} \right) b_i \leq b_i$$

A notwithstanding clause, “when $b_i/b_0 > 0.85$ or $\theta_i > 50^\circ$, $b_{eoi}/2$ shall not exceed $2t_0$ and when $b_i/b_j > 0.85$ or $(180^\circ - \theta_i - \theta_j) > 50^\circ$, $b_{eov}/2$ shall not exceed $2t_j$ ”, was also given (AISC 2010).

For welds to the overlapped branch (member j) in RHS-to-RHS overlapped K-connections (AISC 360-10 Table K4.1) (AISC 2010):

$$(11a) \quad l_{e,j} = \frac{2h_j}{\sin \theta_j} + 2b_{eoj}$$

$$(11b) \quad b_{eoj} = \left(\frac{10t_0}{b_0} \right) \left(\frac{F_{y0}t_0}{F_{yj}t_j} \right) b_j \leq b_j$$

where $l_{e,j}$ = weld effective length for the overlapped branch (member j)-and b_{eoj} = effective length of the transverse weld(s) between the overlapped branch and the chord (two locations).

However, when $b_j/b_0 > 0.85$ or $\theta_j > 50^\circ$ (AISC 360-10 Table K4-1) (AISC 2010):

$$(12) \quad l_{e,j} = \frac{2(h_j - 1.2t_j)}{\sin \theta_j}$$

Tousignant and Packer (2015) used ultimate tensile loads and strain gauge readings (see Fig. 9) to investigate Eqs. (7)-(9) pertaining to AISC 360-10 (AISC 2010). In general, it was verified that when $O_v \geq 50\%$, the sides to the overlapping branch (elements a and b in Fig. 4c) are “fully effective”, while the welds to the toe and heel (elements c and d) are only partially so; however, the notwithstanding clause [“when $b_i/b_0 > 0.85$ or $\theta_i > 50^\circ$, $b_{eoi}/2$

251 shall not exceed $2t_0$ and when $b_i/b_j > 0.85$ or $(180^\circ - \theta_i - \theta_j) > 50^\circ$, $b_{eov}/2$ shall not exceed $2t_j$] was found to be
252 over-conservative. It was therefore recommended that the clause be modified to: “when $b_i/b_0 > 0.85$ or $\theta_i > 50^\circ$,
253 $b_{eoi}/2$ shall not exceed $b_i/4$ and when $b_i/b_j > 0.85$ or $(180^\circ - \theta_i - \theta_j) > 50^\circ$, $b_{eov}/2$ shall not exceed $b_i/4$ ”, for
254 consistency with rules for RHS-to-RHS T-, Y- and X-connections. This was shown to still provide $\beta^+ \geq 4.0$ and
255 subsequently adopted in AISC 360-16 (AISC 2016).

256 As a limitation of this research, weld-critical tests have not been conducted on welds to overlapped branches
257 in RHS-to-RHS overlapped K-connections which, when under tension, would be “pinned down” by the adjacent
258 overlapping branch (making rupture in welds to overlapped branches difficult to investigate experimentally). This
259 is further complicated by the option of welding (or not welding) the “hidden toe” of the overlapped branch member
260 to the chord (Wardenier et al. 2016). Eqs. (11a), (11b), and (12) are hence rationally based on Eq. (4) (for l_e in Y-
261 connections) and are speculative.

262 3. Weld Effective Properties for CHS-to-CHS Connections

263 While much research has been performed on weld-critical RHS-to-RHS connections, leading to “weld
264 effective length rules” for their design in AISC 360-16 (AISC 2016), until recently little research had been
265 conducted on weld-critical CHS-to-CHS connections.

266 3.1. CHS-to-CHS T- Y- and X-connections

267 Tousignant and Packer (2017a) conducted 12 experimental tests on fillet welds in CHS-to-CHS X-
268 connections, fabricated from large-size hollow sections, and loaded in a similar manner to tests shown in Fig. 6a.
269 The measured member geometries, material strengths, and ultimate tensile strengths of the 12 connections are
270 given in Table 9.

271 Due to their complexity, convenient dimensions of the all-around fillet weld in each connection were
272 measured externally, and computer modelling software was used to replicate the weld profile to accurately
273 determine t_w (at numerous locations along the weld length). The average values of t_w and X_u for each connection
274 are summarized in Table 10.

275 By using strain gauges around the branch members, close to the welds, Tousignant and Packer (2017a)
276 found that the load on the weld peaked at the saddle point(s) and decreased as a function of the distance to the

crown of the connection. Additionally, the non-uniform loading was shown to be more pronounced for connections with higher β ($=d_1/d_0$, where d_1 = branch diameter and d_0 = chord diameter) and d_0/t_0 ratios, resulting in lower weld strengths.

The 12 tests by Tousignant and Packer (2017a) were extended by Tousignant and Packer (2018), who conducted a parametric study of 256 FE weld-critical CHS-to-CHS X-connections using a weld failure criterion similar to that used by Yaghoubshahi et al. (2019). The FE models included connections with $60^\circ \leq \theta \leq 90^\circ$, $10 \leq d_0/t_0 \leq 50$, $0.10 \leq \beta \leq 0.50$, and $0.20 \leq \tau \leq 1.00$.

Based on this work, weld effective lengths in CHS-to-CHS X-connections (over the parameter range covered) were found to increase as β and d_0/t_0 (predominantly) decreased, with the weld length becoming 100% effective for $\beta(d_0/t_0) \leq 8$. This “weld-effective-length trend” was deemed likely to be prevalent in CHS-to-CHS connections of similar geometries under branch axial loading (i.e. T- and Y-connections) and for different weld types (Tousignant and Packer 2019a). An equation for l_e in CHS-to-CHS T-, Y- and X-connections was hence proposed, which utilizes a simplified “weld length factor” (K_a) (AWS 2015) to calculate the total weld length (l_w) from the branch circumference (πd_1):

$$(13a) \quad l_e = \frac{4}{\sqrt{2\beta(D/t)}} l_w \leq l_w$$

$$(13b) \quad l_w = \pi d_1 K_a$$

$$(13c) \quad K_a = \frac{1 + 1/\sin \theta}{2}$$

This work was extended by Tousignant and Packer (2019a) to illustrate how the so-called “directional strength-enhancement factor”, given by the term $(1.00+0.5\sin^{1.5}\theta)$ in Eq. J2.5 of AISC 360-16 (AISC 2016) and Clause 13.13.2.2 of CSA S16-19 (CSA 2019), could be applied to fillet welds in CHS-to-CHS joints, and it was shown that the “ $\sin\theta$ factor” provided $\beta^+ \geq 4.0$ in conjunction with the LRFD method of AISC 360-16 (AISC 2016). However, unlike AISC 360-22 (AISC 2022), which will permit the $(1.00+0.5\sin^{1.5}\theta)$ factor for fillet welds to the ends of CHS branches, CSA S16-19 (CSA 2019) does not currently allow it for any single-sided fillet welds connected to an element in tension. This is a topic of recent (Packer et al. 2016; Tousignant and Packer 2016, 2017b, 2019b) and ongoing research.

4. Application to Canadian Standards

As discussed in Section 1, methods to calculate weld effectiveness for RHS-to-RHS connections in Canada are given in Packer and Henderson (1997), and referenced in Part 3: “Connections and Tension Members” of the 11th edition of the CISC Handbook (CISC 2016); however, many of these methods are substantially out-of-date, and their reliability when they are used in conjunction with modern Canadian standards is unclear. The forthcoming section of this article reviews the resistance of welds according to CSA S16-19 (CSA 2019) and examines the inherent reliability of the foregoing AISC 360-16 (AISC 2016) weld effective property equations for RHS-to-RHS connections [and the equations recommended by Tousignant and Packer (2019a) for CHS-to-CHS connections for AISC 360-22] when they are applied with Canadian standards.

4.1. Weld Resistance according to CSA S16-19

According to CSA S16 (CSA 2019) Clause 13.13.3.2, the factored tensile resistance (T_r) of PJP groove welds made with matching electrodes and loaded normal to their axis is taken as:

$$(14) \quad T_r = \phi_w A_n F_u \leq \phi A_1 F_{y1}$$

where, in conjunction with weld effective lengths, A_n = fusion face effective area (calculated using the penetration depth, multiplied by the effective length) and ϕ_w = resistance factor for welds (0.67). PJP welds in the tests herein are all longitudinal welds at an RHS corner, in matched-width RHS connections, and the penetration depth in all cases was the branch thickness, t_l .

For PJP groove welds in compression, made with matching electrodes, the compressive resistance is taken as that of the effective area of the base metal in the joint (CSA S16-19 Clause 13.13.4.1), which could still conservatively be treated as being equal to T_r [Eq. (14)].

For single-sided fillet welds, such as those to HSS under branch axial tension or bending, made with matching electrodes, the factored shear resistance (V_r), for direct shear and tension- or compression-induced shear, is given by Clause 13.13.2.2 (CSA 2019):

$$(15) \quad V_r = 0.67 \phi_w A_w X_u$$

where A_w = weld effective throat area ($=l_e \times t_w$).

Unlike AISC 360-16 (AISC 2016) Chapter K, which specifically covers the resistance of welds to the ends of RHS branches using weld effective properties, it is necessary to use engineering judgment to infer the resistance of welds to HSS branches according to CSA S16-19 (CSA 2019). In doing so, CSA S16-19 Clause 13.13 could be extended to imply the following:

For PJP groove welds to the ends of HSS branches, the weld resistance is:

$$(16a) \quad P_r = \phi_w A_n F_u \leq \phi A_1 F_{y1} \quad \text{for branches under axial loading}$$

$$(16b) \quad M_{r-ip} = \phi_w S_{n-ip} F_u \leq \phi S_{l-ip} F_{y1} \quad \text{for branches under in-plane bending}$$

$$(16c) \quad M_{r-op} = \phi_w S_{n-op} F_u \leq \phi S_{l-op} F_{y1} \quad \text{for branches under out-of-plane bending}$$

where P_r = factored axial resistance of weld; M_{r-ip} = factored in-plane bending resistance of weld; M_{r-op} = factored out-of-plane bending resistance of weld; S_{n-ip} = fusion face effective elastic section modulus for in-plane bending (calculated using the corresponding equation for S_{ip} , but with t_l instead of t_w); S_{n-op} = fusion face effective elastic section modulus for out-of-plane bending (calculated using the corresponding equation for S_{op} , but with t_l instead of t_w); S_{l-ip} = elastic section modulus of the branch for in-plane bending; and S_{l-op} = elastic section modulus of the branch for out-of-plane bending.

For fillet welds made with matching electrodes to the ends of HSS branches, the weld resistance is given by:

$$(17a) \quad P_r = 0.67 \phi_w t_w l_e X_u \quad \text{for branches under axial loading}$$

$$(17b) \quad M_{r-ip} = 0.67 \phi_w S_{ip} X_u \quad \text{for branches under in-plane bending}$$

$$(17c) \quad M_{r-op} = 0.67 \phi_w S_{op} X_u \quad \text{for branches under out-of-plane bending}$$

For the different RHS-to-RHS and CHS-to-CHS connection types covered in Sections 2 and 3 of this paper, the effective section properties (i.e. lengths or moduli) can be calculated according to the equations given therein and substituted into the corresponding equation above for the branch loading sense. The mechanical and geometric properties can then also be substituted to determine the welded joint resistance.

4.2. Reliability Analysis

To evaluate the weld effective property equations in Sections 2 and 3, when used in conjunction with CSA S16-19 (CSA 2019), a standard reliability analysis (Eq. 18) can be performed to calculate the inherent reliability

index (i.e. β^+) and compare it to the current target of 4.0 for connectors per Annex B of CSA S16-19 (CSA 2019) using the method of Ravindra and Galambos (1978) and Fisher et al. (1978) whereby:

$$(18) \quad \phi = \phi_{\beta^+} \rho_R \exp(-\alpha_R \beta^+ V_R)$$

where α_R = coefficient of separation, taken as 0.55 (Ravindra and Galambos 1978); ρ_R = bias coefficient for resistance; V_R = associated coefficient of variation (COV) of ρ_R ; and ϕ_{β^+} = adjustment factor for ϕ_w (needed when β^+ is not equal to the reliability index used for the evaluation of the load factors, which is normally 3.0) (Fisher et al. 1978), which can be closely approximated using Eq. (19) (Franchuk et al. 2002):

$$(19) \quad \phi_{\beta^+} = 0.0062(\beta^+)^2 - 0.131\beta^+ + 1.338$$

The bias coefficient for resistance, ρ_R , and its associated COV, V_R , are given by:

$$(20) \quad \rho_R = \rho_M \rho_G \rho_P$$

$$(21) \quad V_R = \sqrt{V_M^2 + V_G^2 + V_P^2}$$

where ρ_M = mean ratio of actual-to-nominal ultimate tensile strength for the weld metal (X_u); ρ_G = mean ratio of actual-to-nominal values for the weld throat area; ρ_P = mean ratio of actual (experimental or FE)-to-predicted joint strength. V_M , V_G , and V_P are the associated COVs of ρ_M , ρ_G , and ρ_P , respectively.

For consistency with the most-recent weld effective length recommendations for AISC 360-16 (AISC 2016) (Tousignant and Packer 2019a), values of $\rho_M = 1.12$, $V_M = 0.12$, $\rho_G = 1.03$ and $V_G = 0.10$ are used herein (see Table 11). The rationale for selecting these values is discussed in Tousignant and Packer (2019a).

The values of ρ_P , which relate the actual (experimental or FE) weld strength to the nominal (unfactored) weld strength ($P_n = P_r/\phi_w$ or $M_{n-ip} = M_{r-ip}/\phi_w$), are calculated as the average over all tests with reliable/known root details on each connection type (i.e. omitting K-0.60-0.60 for the overlapped K-connections) using N_{lu} (or M_{lu}) divided by P_n (or M_{n-ip}), with P_n (or M_{n-ip}) calculated using the actual measured geometric and material properties, as follows:

- For RHS-to-RHS connections:

$$(22) \quad P_n = \sum P_{n,k}$$

$$(23) \quad M_{n-ip} = \sum M_{n-ip,k}$$

where $P_{n,k}$ = nominal (unfactored) axial strength of weld element k (with $k = a, b, c$, and d – and a' and b' for overlapped K-connections, see Fig. 4) and $M_{n-ip,k}$ = nominal (unfactored) in-plane bending strength of weld element k , with $P_{n,k}$ and $M_{n-ip,k}$ given by:

$$(24) \quad P_{n,k} = A_{n,k} F_u \leq A_{l,k} F_{y1} \quad \text{for PJP weld elements to branches under axial loading}$$

$$(25) \quad M_{n-ip,k} = S_{n-ip,k} F_u \leq \phi S_{l-ip,k} F_{y1} \quad \text{for PJP weld elements to branches under in-plane bending}$$

$$(26) \quad P_{n,k} = 0.67 t_w l_{e,k} X_u \quad \text{for fillet weld elements to branches under axial loading}$$

$$(27) \quad M_{n-ip,k} = 0.67 S_{ip,k} X_u \quad \text{for fillet weld elements to branches under in-plane bending}$$

where $l_{e,k}$ = effective length of weld element k ; $S_{ip,k}$ = effective elastic section modulus of weld element k for in-plane bending; $A_{n,k}$ = fusion face effective area of weld element k (calculated as $t_l \times l_{e,k}$ for the tests herein); $A_{l,k}$ = cross-sectional area of branch wall adjacent to weld element k ; and $S_{n-ip,k}$ = fusion face effective elastic section modulus of weld element k for in-plane bending (calculated using the equation for $S_{ip,k}$, but with t_l instead of t_w).

For example, the nominal axial strength of a transverse fillet weld element in an RHS-to-RHS T-connection is [e.g. element c , giving $k = c$ in Eq. (26)]:

$$(28a) \quad P_{n,c} = 0.67 t_w l_{e,c} X_u \quad \text{with}$$

$$(28b) \quad l_{e,c} = b_e$$

and the nominal in-plane bending strength of the same weld element ($M_{n-ip,c}$) is:

$$(29a) \quad M_{n-ip,c} = 0.67 S_{ip,c} X_u \quad \text{with}$$

$$(29b) \quad S_{ip,c} = \frac{t_w b_e}{2} \left(\frac{h_1}{\sin \theta} \right)$$

The above equations for $l_{e,c}$ and $S_{ip,c}$ are inferred from Eqs. (4) and (5), respectively and, in both cases, the notwithstanding clause “when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ shall not exceed $b_l/4$ ” would apply to calculating b_e .

- For CHS-to-CHS connections: the nominal strength of the weld was calculated using Eq. (17a) with Eq. (13a) for l_e .

It should be noted that the above weld element-by-element approach for RHS-to-RHS connections is only necessary for the interpretation of experimental results, whereas in design the objective is to specify a required

effective throat. Furthermore, Eqs. (26) and (27), which pertain to fillet welds, assume that matching electrodes are used.

4.3. Reliability Analysis Results

The reliability analysis results are summarized as follows (also see Table 11, Fig. 10 and Fig. 11):

1. For the 25 RHS-to-RHS gapped K-connections (all with $\theta \geq 60^\circ$) (Frater and Packer 1992a,b), Eq. (2) gives $\rho_P = 1.16$ with a COV, V_P of 0.24 and a reliability index, $\beta^+ = 3.92$ (see Table 11). This is within 2.25% of the target ($\beta^+ = 4.0$). Fig. 10a shows the actual-to-predicted (A/P) strength graph (i.e. N_{lu} vs. P_n) for the 25 tests. In Fig. 10a (as well as Figs. 10c and d), labelled data points with upward-pointing arrows denote connection failures rather than weld failures (and illustrate that the actual weld strength in the joint would be higher).
2. For the 16 RHS-to-RHS T-, Y-, and X-connections tested under branch under axial load (Packer and Cassidy 1995), Eq. (4) (subject to the notwithstanding clause “when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ shall not exceed $b_l/4$ ”) gives $\rho_P = 0.99$ (Fig. 10b), $V_P = 0.14$, and $\beta^+ = 3.91$ (see Table 11) (within 2.25% of the target).
3. For the 73 RHS-to-RHS T-connections tested under branch in-plane bending (McFadden and Packer 2014; Yaghoubshahi et al. 2019), Eq. (5) (subject to the notwithstanding clause “when $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ shall not exceed $b_l/4$ ”) gives $\rho_P = 2.25$ (Fig. 10c), $V_P = 0.35$, and $\beta^+ = 5.44 \geq 4.0$.
4. For the 10 RHS-to-RHS overlapped K-connections (Tousignant and Packer 2015; Frater and Packer 1992a,b), Eqs. (7)-(9) (subject to the notwithstanding clause “when $b_i/b_0 > 0.85$ or $\theta_i > 50^\circ$, $b_{eoi}/2$ shall not exceed $b_i/4$ and when $b_i/b_j > 0.85$ or $(180^\circ - \theta_i - \theta_j) > 50^\circ$, $b_{eov}/2$ shall not exceed $b_i/4$ ”) give $\rho_P = 1.18$ (Fig. 10d), $V_P = 0.22$, and $\beta^+ = 4.15 \geq 4.0$.
5. For the 268 CHS-to-CHS X-connections (Tousignant and Packer 2017a, 2018), Eq. (13a) gives $\rho_P = 1.42$, $V_P = 0.08$ (Fig. 11), and $\beta^+ = 6.39 \geq 4.0$.

5. Conclusions and Recommendations

Based on the preceding analyses, it can be concluded that the AISC 360-16 (AISC 2016) weld effective property equations (and the proposed design approach) for RHS-to-RHS moment T-, Y- and X-connections under branch in-plane bending and RHS-to-RHS overlapped K-connections meet the target reliability index of 4.0 per Annex B.4 of CSA S16-19 (CSA 2019). Similarly, the AISC 360-16 (AISC 2016) weld effective property

equations (and the proposed design approach) for RHS-to-RHS T-, Y-, X-, and gapped K-connections fall within 2.25% of this target. As $\beta^+ = 4.0$ (or 4.5) is based on calibration with past practice, and recent precedent exists in North America for adopting rules with marginally lower β^+ values [e.g. $\beta^+ = 3.7$ for fillet welds in CHS-to-plate connections (see Tousignant and Packer 2019a)], it is recommended that these rules, for welds in RHS-to-RHS connections, be adopted “as-are” for use with Canadian Standards.

It can also be concluded that Eq. (13a) for l_e in CHS-to-CHS T-, Y- and X-connections produces a very high reliability index ($\beta^+ = 6.39 \geq 4.0$ in Table 11) when used in conjunction with CSA S16-19, which prohibits the use of the $(1.00+0.5\sin^{1.5}\theta)$ factor for fillet welds (CSA 2019). If the $(1.00+0.5\sin^{1.5}\theta)$ factor is permitted to be used to calculate P_n [which can be done by multiplying P_n in Eq. (17a) by the appropriate “directional strength-increase factor for CHS-to-CHS connections” (K_{CHS}) in Table 12 (Tousignant and Packer 2019a)], then the reliability analysis parameters in the rightmost column of Table 11 become $\rho_P = 0.96$, $V_P = 0.06$, and $\beta^+ = 4.23 \geq 4.0$. This acceptable result (i.e. $\beta^+ \geq 4.0$) is made possible by the recent change to the target β^+ for connectors (from 4.5 to 4.0) in Annex B.4 of CSA S16-19 (CSA 2019) (see Section 2.1 of this paper). Accordingly, it is recommended to permit the “ $\sin\theta$ factor” for design of single-sided fillet welds to the ends of CHS branches in Clause 13.13.2.2 of CSA S16-24 (CSA 2024). Doing so would also bring CSA S16-24 into line with AISC 360-22 with respect to the design resistance of fillet welds.

A composite actual-to-predicted (A/P) comparison of the above recommendations (i.e. weld effective property equations for use in conjunction with proposed CSA S16-24 rules for the strength of fillet and PJP welds to HSS branches) is shown in Fig. 12, wherein the recommended weld effective property equations (summarized in Tables 13 and 14) have been used with Eqs. (16a,b) and (17a,b) to compute P_n or M_{n-ip} . (Note that in the tables, P = applied axial load and M = applied bending moment).

Since the recommendations of Table 13 are based on connection parameters in accord with AISC 360, they are suitable for RHS-to-RHS connections that meet the “Limits of Applicability of Table K2.2” (i.e. Table K2.2A) of AISC 360-10 (AISC 2010). (Less-explicit tables are given in AISC 360-16 (AISC 2016)). The recommendations of Table 14 are suitable for CHS-to-CHS X-connections with: $60^\circ \leq \theta \leq 90^\circ$, $10 \leq d_o/t_o \leq 50$, $0.10 \leq \beta \leq 0.50$, and $0.20 \leq \tau \leq 1.00$, where t_w is constant around the joint (Tousignant and Packer 2019a). These “limits of applicability”

453 for Table 14 reflect the range of parameters over which experimental testing or FE analysis has been performed
454 (see Section 3.1).

455 The authors consider that these tables (i.e. Tables 13 and 14) would be appropriate for introduction into the
456 next scheduled revision of CSA W59 (CSA 2023).

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461 **List of Symbols**

A_0	chord cross-sectional area
A_l	branch cross-sectional area
$A_{l,k}$	cross-sectional area of branch wall adjacent to weld element k
A_i	overlapping branch cross-sectional area
A_j	overlapped branch cross-sectional area
A_n	fusion face effective area, for PJP welds
$A_{n,k}$	fusion face effective area of weld element k
A_w	weld effective throat area ($= l_e \times t_w$)
F_u	tensile strength of the governing base metal (lower of the branch and the chord)
F_{y0}	chord yield strength
F_{yl}	branch yield strength
F_{yi}	overlapping branch yield strength
F_{yj}	overlapped branch yield strength
K_{CHS}	directional strength-increase factor for CHS-to-CHS connections
K_a	weld length factor
M	applied bending moment
M_{lu}	ultimate moment strength
M_{n-ip}	nominal in-plane bending strength of weld
$M_{n-ip,k}$	nominal in-plane bending strength of weld element k
M_{r-ip}	factored in-plane bending resistance of weld
M_{r-op}	factored out-of-plane bending resistance of weld
N_{lu}	ultimate tensile strength

O_v	overlap ($= q/p \times 100\%$)
P	applied axial load
P_n	nominal axial strength of weld
$P_{n,k}$	nominal axial strength of weld element k
P_r	factored axial resistance of weld
S_{I-ip}	elastic section modulus of the branch for in-plane bending
S_{I-op}	elastic section modulus of the branch for out-of-plane bending
S_{ip}	weld effective elastic section modulus for in-plane bending
$S_{ip,k}$	effective elastic section modulus of weld element k for in-plane bending
S_{n-ip}	fusion face effective elastic section for in-plane bending modulus
$S_{n-ip,k}$	fusion face effective elastic section modulus of weld element k for in-plane bending
S_{n-op}	fusion face effective elastic section modulus for out-of-plane bending
S_{op}	weld effective elastic section modulus for out-of-plane bending
T_r	factored tensile resistance of PJP weld
V_G	coefficient of variation of ρ_G
V_M	coefficient of variation of ρ_M
V_P	coefficient of variation of ρ_P
V_R	coefficient of variation of ρ_R
V_r	factored shear resistance of fillet weld
X_u	electrode tensile strength
b_0	chord width, transverse to the plane of the connection
b_l	branch width, transverse to the plane of the connection
b_e	effective width of an RHS member; effective length of a transverse weld element
b_{eoi}	effective length of the transverse weld at the heel of the overlapping branch
b_{eov}	effective length of the transverse weld at the toe of the overlapping branch
b_{eoj}	effective length of the transverse weld(s) between the overlapping branch and the chord
b_i	overlapping branch width
b_j	overlapped branch width
d_0	chord diameter
d_l	branch diameter
h_0	chord height, in the plane of the connection
h_l	branch height, in the plane of the connection
h_i	overlapping branch height
i	subscript denoting the overlapping branch in an overlapped K-connection
j	subscript denoting the overlapped branch in an overlapped K-connection

k	subscript denoting a weld element ($k = a, b, c, d, a'$ or b') (see Fig. 4)
h_j	overlapped branch height
l_e	weld effective length
$l_{e,i}$	weld effective length for the overlapping branch
$l_{e,j}$	weld effective length for the overlapped branch
$l_{e,k}$	effective length of weld element k
l_w	total weld length
p	length of the projected contact area of the overlapping branch onto the face of the chord, in the absence of the overlapped branch
q	length of overlap, measured at the face of the chord, between one web member toe and the position of the other projected web member toe
t_0	chord thickness
t_l	branch thickness
t_i	overlapping branch thickness
t_j	overlapped branch thickness
t_w	weld throat dimension
α_R	coefficient of separation, taken as 0.55
β	branch-to-chord width ratio ($= b_l/b_0$); branch-to-chord diameter ratio ($= d_l/b_0$)
β^+	safety/reliability index
ϕ_w	resistance factor for welds ($= 0.67$)
ϕ_β^+	adjustment factor for ϕ_w
θ	included angle between the branch and chord
θ_i	included angle between the overlapping branch and chord
θ_j	included angle between the overlapped branch and chord
ρ_G	mean ratio of actual-to-nominal values for the weld throat area
ρ_M	mean ratio of actual-to-nominal ultimate tensile strength for the weld metal
ρ_P	mean ratio of actual-to-predicted weld strength
ρ_R	bias coefficient for resistance
τ	branch-to-chord thickness ratio ($= t_l/t_0$)

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Fig. 12. Actual-to-predicted (A/P) correlation for all available tests using the recommended weld effective properties (Tables 13 and 14) in conjunction with Eqs. (16a,b) and (17a,b) and permitting the fillet weld directional strength-increase factor for CHS (*two-column figure*)

Tables

Table 1. Measured geometric and mechanical properties of RHS-to-RHS gapped K- and N-connection test specimens

Test number	HSS chord			HSS branch			θ (°)	N_{lu} (kN)
	$h_0 \times b_0 \times t_0$ (mm $\times$ mm $\times$ mm)	A_0 (mm ²)	F_{y0} (MPa)	$h_l \times b_l \times t_l$ (mm $\times$ mm $\times$ mm)	A_l (mm ²)	F_{yl} (MPa)		
T1-1	305.3 $\times$ 204.4 $\times$ 12.0	11335	405	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	761
T1-2	305.3 $\times$ 204.4 $\times$ 12.0	11335	405	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	790
T1-3	305.0 $\times$ 204.4 $\times$ 11.9	11279	457	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	782
T1-4	305.0 $\times$ 204.4 $\times$ 11.9	11279	457	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	885
T1-5	305.3 $\times$ 204.4 $\times$ 12.0	11335	405	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	967
T1-6	305.3 $\times$ 204.4 $\times$ 12.0	11335	405	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	1209
T1-7	305.0 $\times$ 204.4 $\times$ 11.9	11279	457	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	1613
T1-8	305.0 $\times$ 204.4 $\times$ 11.9	11279	457	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	1572
T1-9	305.3 $\times$ 204.4 $\times$ 12.0	11335	405	127.7 $\times$ 127.7 $\times$ 11.8	5347	481	60	1637 [‡]
T2-1	204.0 $\times$ 204.0 $\times$ 12.0	8925	394	127.9 $\times$ 127.9 $\times$ 12.0	5377	416	60	831
T2-2	204.0 $\times$ 204.0 $\times$ 12.0	8925	394	127.9 $\times$ 127.9 $\times$ 12.0	5377	416	60	1063
T2-3	203.8 $\times$ 203.8 $\times$ 12.1	8922	374	127.9 $\times$ 127.9 $\times$ 12.0	5377	416	60	1353
T2-5	203.8 $\times$ 203.8 $\times$ 12.1	8922	374	127.9 $\times$ 127.9 $\times$ 12.0	5377	416	60	1483
T2-7	203.8 $\times$ 203.8 $\times$ 12.1	8922	374	127.9 $\times$ 127.9 $\times$ 12.0	5377	416	60	1374
T2-8	203.8 $\times$ 203.8 $\times$ 12.1	8922	374	127.9 $\times$ 127.9 $\times$ 12.0	5377	416	60	1644 [‡]
P3-16	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	90	801
P3-17	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	90	1181
P3-18	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	90	1023
P3-19	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	90	602
P3-20	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	90	1388 [‡]
P3-21	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	60	1140
P3-22	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	60	1328
P3-24	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	60	861
P3-25	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	60	1521 [‡]
P3-26	203.5 $\times$ 203.5 $\times$ 12.5	9113	352	127.6 $\times$ 127.6 $\times$ 9.54	4296	440	30	2010

Symbols: h_0 = chord height; b_0 = chord width; t_0 = chord thickness; A_0 = chord cross-sectional area; F_{y0} = chord yield strength; h_l = branch height; b_l = branch width; t_l = branch thickness; A_l = branch cross-sectional area; F_{yl} = branch yield strength; θ = included angle between the branch and chord.

Test numbers: T1 = RHS-to-RHS gapped K-connections in a 12.2-m span truss; T2 = RHS-to-RHS gapped K-connections in a 12.0-m span truss; P3 = isolated RHS-to-RHS X-connections with simulated gapped K-connection restraints.

[‡] connection (not weld) failure.

603 **Table 2.** Average measured weld properties for RHS-to-RHS gapped K- and N-connection test specimens

Test number	t_w (mm)				X_u (MPa)
	Side weld (a)	Side weld (b)	Toe weld (c)	Heel weld (d)	
T1-1	3.9	3.9	2.7	6.1	678
T1-2	4.2	4.1	2.8	5.7	678
T1-3	4.4	4.8	3.0	7.0	678
T1-4	5.0	5.1	3.2	7.9	678
T1-5	6.5	6.6	4.8	9.4	678
T1-6	6.8	6.1	4.7	9.9	678
T1-7	8.2	8.1	5.7	11.3	678
T1-8	8.4	8.3	5.8	11.4	678
T1-9	8.7	8.8	7.3	13.2	678
T2-1	3.7	3.9	2.8	6.2	540
T2-2	5.0	5.5	3.8	7.8	540
T2-3	6.3	6.4	5.2	9.8	540
T2-5	9.2	8.9	6.2	12.5	540
T2-7	9.1	9.4	7.3	13.4	540
T2-8	12.3	12.9	8.9	17.3	540
P3-16	4.6	4.6	4.6	4.6	739
P3-17	8.5	8.5	8.5	8.5	739
P3-18	5.8	5.8	5.8	5.8	687
P3-19	3.2	3.2	3.2	3.2	687
P3-20	11.9	11.9	11.9	11.9	739
P3-21	4.3	4.3	3.8	7.0	739
P3-22	7.2	7.2	7.4	11.1	739
P3-24	4.3	4.3	3.4	6.9	687
P3-25	11.7	11.7	10.6	15.8	739
P3-26	8.0	8.0	-	12.2	739

604 Test numbers: see footnote to Table 1.

605

606 **Table 3.** Measured geometric and mechanical properties of axially loaded RHS-to-RHS T- and X-connection test
607 specimens

Test number	HSS chord			HSS branch			θ (°)	N_{lu} (kN)
	$h_o \times b_o \times t_o$ (mm $\times$ mm $\times$ mm)	A_o (mm ²)	F_{y0} (MPa)	$h_l \times b_l \times t_l$ (mm $\times$ mm $\times$ mm)	A_l (mm ²)	F_{yl} (MPa)		
T1	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	126.9 $\times$ 126.9 $\times$ 12.2	5,231	545	90	527
T2	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	126.9 $\times$ 126.9 $\times$ 12.2	5,231	545	90	687
T3	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	203.0 $\times$ 203.0 $\times$ 12.1	8,684	445	90	907
T4	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	203.0 $\times$ 203.0 $\times$ 12.1	8,684	445	90	868
X1	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	126.9 $\times$ 126.9 $\times$ 12.2	5,231	545	30	1,380
X2	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	126.9 $\times$ 126.9 $\times$ 12.2	5,231	545	40	893
X3	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	126.9 $\times$ 126.9 $\times$ 12.2	5,231	545	50	663
X4	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	126.9 $\times$ 126.9 $\times$ 12.2	5,231	545	60	464
X5	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	126.9 $\times$ 126.9 $\times$ 12.2	5,231	545	70	424
X6	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	126.9 $\times$ 126.9 $\times$ 12.2	5,231	545	80	431
X7	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	203.0 $\times$ 203.0 $\times$ 12.1	8,684	445	30	2,541
X8	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	203.0 $\times$ 203.0 $\times$ 12.1	8,684	445	40	1,692
X9	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	203.0 $\times$ 203.0 $\times$ 12.1	8,684	445	50	1,380
X10	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	203.0 $\times$ 203.0 $\times$ 12.1	8,684	445	60	972
X11	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	203.0 $\times$ 203.0 $\times$ 12.1	8,684	445	70	880
X12	253.8 $\times$ 253.8 $\times$ 12.1	11,097	410	203.0 $\times$ 203.0 $\times$ 12.1	8,684	445	80	873

608 Test numbers: T = RHS-to-RHS T-connections; X = RHS-to-RHS X-connections.

Table 4. Average measured weld properties for axially loaded RHS-to-RHS T- and X-connection test specimens

Test number	t_w (mm)				X_u (MPa)
	Side weld (a)	Side weld (b)	Toe weld (c)	Heel weld (d)	
T1	4.0	3.6	3.6	3.9	574
T2	4.7	4.3	5.2	4.6	574
T3	3.7	3.3	3.2	3.4	574
T4	4.6	4.7	4.1	4.2	574
X1	3.9	4.4	5.7	9.6	574
X2	4.5	4.7	5.4	6.2	574
X3	3.7	4.2	4.4	4.2	574
X4	3.4	3.7	3.5	5.7	574
X5	3.5	3.2	2.7	4.9	574
X6	3.3	3.3	3.4	3.8	574
X7	5.1	5.4	5.7	10.8	574
X8	5.2	5.7	6.0	5.9	574
X9	5.5	5.9	5.1	5.2	574
X10	4.8	4.4	3.3	5.8	574
X11	4.1	5.0	4.2	5.4	574
X12	4.3	4.1	3.3	3.9	574

Test numbers: see footnote to Table 3.

Table 5. Measured geometric and mechanical properties of RHS-to-RHS moment-loaded T-connection test specimens

Test number	HSS chord			HSS branch			$F_u^\dagger$ (MPa)	θ (°)	M_{lu} (kN-m)
	$h_0 \times b_0 \times t_0$ (mm × mm × mm)	A_0 (mm ²)	F_{y0} (MPa)	$h_l \times b_l \times t_l$ (mm × mm × mm)	A_l (mm ²)	F_{yl} (MPa)			
T-0.25-34	203.7 × 203.7 × 5.89	4555	382	51.1 × 51.1 × 5.77	981	409	-	90	5.57
T-0.25-23	202.9 × 202.9 × 8.74	6516	393	51.1 × 51.1 × 5.77	981	409	-	90	5.92
T-0.25-17	204.5 × 204.5 × 11.6	8452	412	51.1 × 51.1 × 5.77	981	409	-	90	6.54
T-0.50-34	203.7 × 203.7 × 5.89	4555	382	102.1 × 102.1 × 5.72	2,155	428	-	90	7.61 [‡]
T-0.50-23	202.9 × 202.9 × 8.74	6516	393	102.1 × 102.1 × 5.72	2,155	428	-	90	20.6
T-0.50-17	204.5 × 204.5 × 11.6	8452	412	102.1 × 102.1 × 11.6	3,942	440	-	90	40.1 [‡]
T-0.75-34	203.7 × 203.7 × 5.89	4555	382	152.7 × 152.7 × 5.74	3,310	331	-	90	19.5
T-0.75-23	202.9 × 202.9 × 8.74	6516	393	152.4 × 152.4 × 8.69	6,239	349	-	90	36.7
T-0.75-17	204.5 × 204.5 × 11.6	8452	412	152.7 × 152.7 × 11.7	4,555	371	-	90	70.8
T-1.00-34	203.7 × 203.7 × 5.89	4555	382	203.7 × 203.7 × 5.89	4,555	382	493	90	53.8
T-1.00-23	202.9 × 202.9 × 8.74	6516	393	202.9 × 202.9 × 8.74	6,516	393	509	90	87.7
T-1.00-17	204.5 × 204.5 × 11.6	8452	412	204.5 × 204.5 × 11.6	8,452	412	508	90	127

[†] branch member ultimate tensile strength is equal to the chord member ultimate tensile strength, for the values shown.

[‡] connection (not weld) failure.

621 **Table 6.** Average measured weld properties for RHS-to-RHS moment-loaded T-connection test specimens

Test number	t_w (mm)				X_u (MPa)
	Side weld (a)	Side weld (b)	Toe weld (c)	Heel weld (d)	
T-0.25-34	2.6	2.5	2.3	2.2	607
T-0.25-23	2.4	3.3	2.4	1.3	607
T-0.25-17	2.3	2.6	2.3	2.4	607
T-0.50-23	3.9	3.4	4.3	4.6	607
T-0.75-34	2.8	2.2	1.7	3.4	607
T-0.75-23	3.5	3.1	3.5	3.0	607
T-0.75-17	6.0	5.1	4.0	7.0	607
T-1.00-34	3.3 [†]	2.0 [†]	3.2	3.0	607
T-1.00-23	4.6 [†]	5.9 [†]	5.2	5.3	607
T-1.00-17	6.1 [†]	7.9 [†]	5.7	6.4	607

622 [†]PJP groove weld.

624

625 **Table 7.** Measured geometric and mechanical properties of RHS-to-RHS overlapped K-connection test specimens

Test number	HSS chord			HSS branch(es) [†]			$F_u^{\dagger}$ (MPa)	$\theta_i = \theta_j^{\dagger\dagger}$ (°)	$O_v^{\dagger\dagger\dagger}$ (%)	N_{lu} (kN)
	$h_0 \times b_0 \times t_0$ (mm × mm × mm)	A_0 (mm ²)	F_{y0} (MPa)	$h_i \times b_i \times t_i$ (mm × mm × mm)	A_i (mm ²)	F_{yi} (MPa)				
K-90-0.50a	254.4 × 254.4 × 9.24	8804	387	127.0 × 127.0 × 7.78	3625	412	477	60	90	1232
K-90-0.50b	254.4 × 254.4 × 9.24	8804	387	127.0 × 127.0 × 7.78	3625	412	477	60	90	1277
K-60-0.50	254.4 × 254.4 × 9.24	8804	387	127.0 × 127.0 × 7.78	3625	412	477	60	60	596 ^{**}
K-30-0.50a	254.4 × 254.4 × 9.24	8804	387	127.0 × 127.0 × 7.78	3625	412	477	60	30	765
K-30-0.50b	254.4 × 254.4 × 9.24	8804	387	127.0 × 127.0 × 7.78	3625	412	477	60	30	738
K-90-0.71	178.7 × 178.7 × 12.5	7776	380	127.0 × 127.0 × 7.78	3625	412	477	60	90	1139
K-60-0.71a	178.7 × 178.7 × 12.5	7776	380	127.0 × 127.0 × 7.78	3625	412	477	60	60	974
K-60-0.71b	178.7 × 178.7 × 12.5	7776	380	127.0 × 127.0 × 7.78	3625	412	477	60	60	863
K-30-0.71	178.7 × 178.7 × 12.5	7776	380	127.0 × 127.0 × 7.78	3625	412	477	60	30	1054
T2-4	204.0 × 204.0 × 12.0	8925	394	127.9 × 127.9 × 12.0	5377	416	522	60	50	1687 [‡]
T2-6	204.0 × 204.0 × 12.0	8925	394	127.9 × 127.9 × 12.0	5377	416	522	60	50	1668 [‡]

626 [†] ultimate tensile strength is the same for both branches.

627 ^{††}Symbols (also see Fig. 8): h_i = overlapping branch height (= h_j = overlapped branch height); b_i = overlapping branch width
628 (= b_j = overlapped branch width); t_i = overlapping branch thickness (= t_j = overlapped branch thickness); A_i = overlapping
629 branch cross-sectional area (= A_j = overlapped branch cross-sectional area); F_{yi} = overlapping branch yield strength (= F_{yj} =
630 overlapped branch yield strength); θ_i = included angle between the overlapping branch and the chord (= θ_j = included angle
631 between the overlapped branch and the chord).

632 ^{†††}Overlap (O_v) = $(q/p) \times 100\%$, where q = length of overlap, measured at the face of the chord, between one web member
633 toe and the position of the other projected web member toe and p = length of the projected contact area of the overlapping
634 branch onto the face of the chord, in the absence of the overlapped branch (see Fig. 8).

635 [‡] connection (not weld) failure.

636 ^{**} imperfect root detail.

638

639

640

Table 8. Average measured weld properties for RHS-to-RHS overlapped K-connection test specimens

Test number	t_w (mm)					X_u (MPa)	
	Side weld (a)	Side weld (a')	Side weld (b)	Side weld (b')	Toe weld (c)		Heel weld (d)
K-90-0.50a	3.5	3.2 [†]	3.1	3.6 [†]	3.8	4.3	619
K-90-0.50b	4.6	3.5 [†]	3.8	3.7 [†]	3.8	4.3	619
K-60-0.50	2.7	3.8 [†]	2.4	3.6 [†]	3.9	4.2	619
K-30-0.50a	3.4	4.6 [†]	3.0	4.0 [†]	4.3	3.6	619
K-30-0.50b	3.3	4.3 [†]	3.0	4.3 [†]	4.0	3.5	619
K-90-0.71	3.2	3.9 [†]	3.2	3.8 [†]	3.6	3.8	619
K-60-0.71a	4.0	3.5 [†]	3.9	3.1 [†]	3.8	3.9	619
K-60-0.71b	3.4	3.6 [†]	3.2	3.8 [†]	3.8	4.0	619
K-30-0.71	4.6	4.9 [†]	3.4	4.8 [†]	4.3	3.8	619
T2-4	4.5	4.5 [†]	4.4	4.4 [†]	7.2	6.7	540
T2-6	7.1	7.1 [†]	6.5	6.5 [†]	9.1	10.6	540

[†]PJP groove weld.**Table 9.** Measured geometric and mechanical properties of CHS-to-CHS X-connection test specimens

Test number	HSS chord			HSS branch			θ (°)	N_{lu} (kN)
	$d_0 \times t_0$ (mm × mm × mm)	A_0 (mm ²)	F_{y0} (MPa)	$d_1 \times t_1$ (mm × mm × mm)	A_1 (mm ²)	F_{y1} (MPa)		
102-273-90a	273.5 × 11.69	9,614	460	102.0 × 7.34	2,161	373	90	672
102-273-90b	273.5 × 11.69	9,614	460	102.0 × 7.34	2,161	373	90	678
102-406-90a	406.5 × 12.34	15,283	355	102.0 × 7.34	2,161	373	90	608
102-406-90b	406.5 × 12.34	15,283	355	102.0 × 7.34	2,161	373	90	540
127-273-90a	273.5 × 11.69	9,614	460	127.4 × 11.55	4,207	431	90	653
127-273-90b	273.5 × 11.69	9,614	460	127.4 × 11.55	4,207	431	90	653
127-406-90a	406.5 × 12.34	15,283	355	127.4 × 11.55	4,207	431	90	557
127-406-90b	406.5 × 12.34	15,283	355	127.4 × 11.55	4,207	431	90	557
102-406-60a	410.0 × 12.21	15,260	373	102.0 × 7.34	2,161	373	60	721
102-406-60b	410.0 × 12.21	15,260	373	102.0 × 7.34	2,161	373	60	721
127-406-60a	410.0 × 12.21	15,260	373	127.4 × 11.55	4,207	431	60	761
127-406-60b	410.0 × 12.21	15,260	373	127.4 × 11.55	4,207	431	60	850

654 **Table 10.** Average measured weld properties for CHS-to-CHS X-connection test specimens

Test number	t_w (mm)	X_u (MPa)
102-273-90a	4.1	577
102-273-90b	4.4	577
102-406-90a	3.6	577
102-406-90b	3.1	577
127-273-90a	3.6	577
127-273-90b	4.0	577
127-406-90a	3.2	577
127-406-90b	3.5	577
102-406-60a	3.6	577
102-406-60b	3.8	577
127-406-60a	4.0	577
127-406-60b	3.4	577

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656 **Table 11.** Reliability analysis parameters and results for AISC 360-16 (AISC 2016) weld effective property
657 equations used in conjunction with CSA S16-19 (CSA 2019)

	RHS-to-RHS connections				CHS-to-CHS connections
	gapped K	T and X	moment T	overlapped K	X
Sample size	25	16	73	10	268
ϕ	0.67	0.67	0.67	0.67	0.67
ρ_M	1.12	1.12	1.12	1.12	1.12
V_M	0.12	0.12	0.12	0.12	0.12
ρ_G	1.03	1.03	1.03	1.03	1.03
V_G	0.10	0.10	0.10	0.10	0.10
ρ_P	1.16	0.99	2.25	1.18	1.42
V_P	0.24	0.14	0.35	0.22	0.08
ρ_R	1.34	1.14	2.60	1.36	1.64
V_R	0.28	0.21	0.38	0.27	0.17
ϕ_{β^+}	0.92	0.92	0.81	0.90	0.75
β^+	3.92	3.91	5.44	4.15	6.39

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659 **Table 12.** Values of the directional strength-increase factor (K_{CHS}) for fillet welds in round-to-round HSS T-, Y-
660 and X-connections under branch axial load

$\beta = d_1/d_0$	θ (°)			
	90	80	70	60
0	1.500	1.494	1.477	1.447
0.1	1.500	1.494	1.476	1.446
0.2	1.498	1.492	1.475	1.445
0.3	1.496	1.490	1.473	1.443
0.4	1.492	1.487	1.470	1.440
0.5	1.487	1.482	1.465	1.436

661 Note 1: The above values of K_{CHS} assume a constant weld throat dimension (t_w).

662 Note 2: For connections having values of β and θ not shown, but $\beta \leq 0.5$ and $60^\circ \leq \theta \leq 90^\circ$, linear interpolation may be
663 used.

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666 **Table 13.** Recommended weld effective properties for RHS-to-RHS connections

Connection Type	Weld Properties
<p>Gapped K-connections under Branch Axial Load</p> <p>Section A-A: Effective weld for $\theta \geq 60^\circ$</p>	<p>Weld Effective Lengths</p> <p>When $\theta \leq 50^\circ$:</p> $l_e = \frac{2(h_1 - 1.2t_1)}{\sin \theta} + 2(b_1 - 1.2t_1)$ <p>When $\theta \geq 60^\circ$:</p> $l_e = \frac{2(h_1 - 1.2t_1)}{\sin \theta} + b_1 - 1.2t_1$ <p>When $50^\circ > \theta > 60^\circ$, linear interpolation shall be used to determine l_e.</p>
<p>T-, Y- and X-connections under Branch Axial Load or Bending</p> <p>Section A-A: Effective weld</p>	<p>Weld Effective Properties</p> $l_e = \frac{2h_1}{\sin \theta} + 2b_e$ $S_{ip} = \frac{t_w}{3} \left(\frac{h_1}{\sin \theta} \right)^2 + t_w b_e \left(\frac{h_1}{\sin \theta} \right)$ $S_{op} = t_w \left(\frac{h_1}{\sin \theta} \right) b_1 + \frac{t_w}{3} (b_1^2) - \frac{(t_w/3)(b_1 - b_e)^3}{b_1}$ $b_e = \left(\frac{10t_0}{b_0} \right) \left(\frac{F_{y0}t_0}{F_{yi}t_i} \right) b_i \leq b_i$ <p>When $\beta > 0.85$ or $\theta > 50^\circ$, $b_e/2$ shall not exceed $b_1/4$.</p>

667 **Table 13 (cont'd).** Recommended weld effective properties for RHS-to-RHS connections

Connection Type	Weld Properties
<p data-bbox="199 237 760 264">Overlapped K-connections under Branch Axial Load</p> <p data-bbox="264 1119 695 1171">Section A-A: Effective weld to member i when $50\% \leq O_v \leq 100\%$</p> <p data-bbox="264 1518 695 1570">Section B-B: Effective weld to member j when $b_j/b_0 \leq 0.85$ and $\theta_j \leq 50^\circ$</p> <p data-bbox="264 1875 695 1927">Section B-B: Effective weld to member j when $b_j/b_0 > 0.85$ or $\theta_j > 50^\circ$</p>	<p data-bbox="865 237 1442 289">Overlapping Branch Member Weld Effective Lengths (member i)</p> <p data-bbox="824 327 1084 354">When $25\% \leq O_v \leq 50\%$:</p> $l_{e,i} = \frac{2O_v}{50} \left[\left(1 - \frac{O_v}{100} \right) \left(\frac{h_i}{\sin \theta_i} \right) + \frac{O_v}{100} \left(\frac{h_i}{\sin(\theta_i + \theta_j)} \right) \right] + b_{eoi} + b_{eov}$ <p data-bbox="824 579 1084 606">When $50\% < O_v < 80\%$:</p> $l_{e,i} = 2 \left[\left(1 - \frac{O_v}{100} \right) \left(\frac{h_i}{\sin \theta_i} \right) + \frac{O_v}{100} \left(\frac{h_i}{\sin(\theta_i + \theta_j)} \right) \right] + b_{eoi} + b_{eov}$ <p data-bbox="824 831 1101 858">When $80\% \leq O_v \leq 100\%$:</p> $l_{e,i} = 2 \left[\left(1 - \frac{O_v}{100} \right) \left(\frac{h_i}{\sin \theta_i} \right) + \frac{O_v}{100} \left(\frac{h_i}{\sin(\theta_i + \theta_j)} \right) \right] + b_i + b_{eov}$ $b_{eoi} = \left(\frac{10t_0}{b_0} \right) \left(\frac{F_{y0}t_0}{F_{yi}t_i} \right) b_i \leq b_i$ $b_{eov} = \left(\frac{10t_j}{b_j} \right) \left(\frac{F_{yj}t_j}{F_{yi}t_i} \right) b_i \leq b_i$ <p data-bbox="824 1339 1474 1434">When $b_i/b_0 > 0.85$ or $\theta_i > 50^\circ$, $b_{eoi}/2$ shall not exceed $b_i/4$ and when $b_i/b_j > 0.85$ or $(180^\circ - \theta_i - \theta_j) > 50^\circ$, $b_{eov}/2$ shall not exceed $b_i/4$.</p> <p data-bbox="865 1465 1442 1518">Overlapped Branch Member Weld Effective Lengths (member j)</p> $l_{e,j} = \frac{2h_j}{\sin \theta_j} + 2b_{eoj}$ $b_{eoj} = \left(\frac{10t_0}{b_0} \right) \left(\frac{F_{y0}t_0}{F_{yj}t_j} \right) b_j \leq b_j$ <p data-bbox="824 1812 1149 1839">When $b_j/b_0 > 0.85$ or $\theta_j > 50^\circ$:</p> $l_{e,j} = 2(h_j - 1.2t_j) / \sin \theta_j$

668 **Table 14.** Recommended weld effective properties for CHS-to-CHS connections

Connection Type	Weld Properties
<p>T-, Y- and X-connections under Branch Axial Load</p> <p>Section A-A: Effective weld</p>	<p>Weld Effective Length</p> $l_e = \frac{4}{\sqrt{2\beta(d_0/t_0)}} l_w \leq l_w$ $l_w = \pi d_1 K_a$ $K_a = \frac{1 + 1/\sin \theta}{2}$ <p>For fillet welds, P_r in Eq. (17a) may be multiplied by the directional strength-increase factor K_{CHS} given in Table 12.</p>

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