WELD DESIGN AND FABRICATION FOR RHS 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* to include a Section K4: *Welds of Plates and Branches to Rectangular HSS*. This paper discusses the historical development of the effective weld properties and analyses the structural reliability of the provisions contained therein. Additionally there is a discussion on recent changes in the U.S. and Canadian specifications/codes with regard to the limit states for fillet weld design and the acceptance/ rejection of the ($1.00 + 0.50 \sin^{1.5}\theta$) term. Finally, the details of an experimental research programme being performed at the University of Toronto, in collaboration with AISC to determine the weld effective length in RHS T-connections under branch in-plane bending moments, are discussed. In conclusion, it is found that the inclusion of the ($1.00 + 0.50 \sin^{1.5}\theta$) term for RHS gapped K- connections and T- and X- connections, based on the limit state of shear failure along the effective throat of the weld, may be unsafe for fillet weld design when used in conjunction with the current weld effective length rules.

1. INTRODUCTION

With welded connections between rectangular hollow sections (RHS) there are currently two design methods that can be used for weld design (Packer et al., 2010):

(i) The welds may be proportioned to develop the yield strength of the connected branch wall at all locations around the branch. This approach may be appropriate if there is low confidence in the design forces, uncertainty regarding method (ii) or if plastic stress-redistribution is required in the connection. This method will produce an upper limit on the required weld size and may be excessively conservative in some situations.

(ii) The welds may be designed as “fit-for-purpose”, and proportioned to resist the applied forces in the branch. The non-uniform loading around the weld perimeter due to the relative flexibility of the connecting RHS face requires the use of effective weld lengths. This approach may be appropriate when there is high confidence in the design forces or if the branch forces are particularly low relative to the branch member capacity. When applicable, this approach may result in smaller weld sizes providing a more economical design with increased aesthetic value.

The primary focus of this paper is method (ii), but it is interesting to compare the results of method (i) for the design of fillet welds in various steel specifications/ codes (see Table 1). Clearly there is quite a disparity.

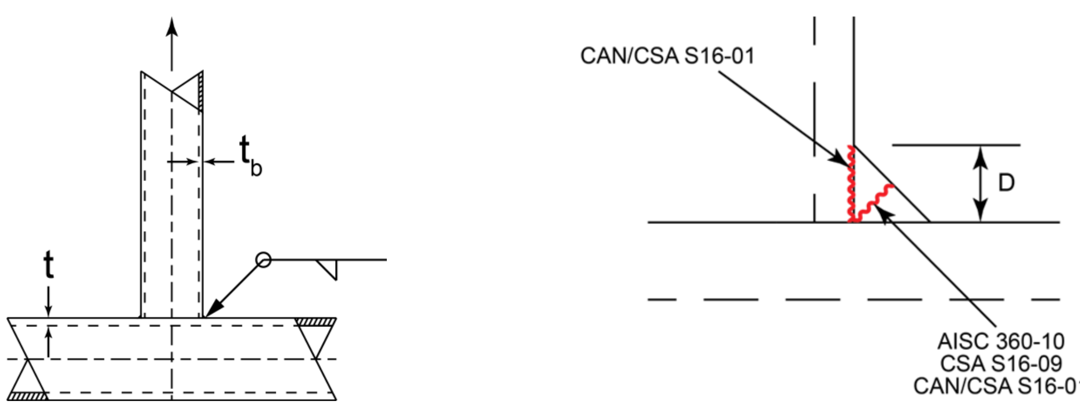
Table 1. Comparison of fillet weld effective throats to develop the yield resistance of the connected branch member wall in Figure 1(a)

Specification or Code	t_W
ANSI/AISC 360-10 Table J2.5	$1.43t_b$
AWS D1.1/D1.1M: 2010 Clause 2.25.1.3 and Fig. 3.2	$1.07t_b$
CSA S16-09 Clause 13.13.2.2	$0.95t_b$
CAN/CSA S16-01 Clause 13.13.2.2	$1.14t_b$
CEN (2005) or IIW (2012)	$1.10t_b$

Fillet welds, being the least expensive and easiest weld type, are the preferred and most common weld type for hollow section connections. The design of fillet welds in structural steel buildings in the U.S. is governed by Table J2.5 of the AISC *Specification* and is based on the limit state of shear failure along the effective throat using a matching (or under-matching) filler metal. For a simple 90° RHS T-connection under branch axial tension (see Figure 1(a)) the LRFD strength of a single weld is given by:

$$\emptyset R_n = \emptyset F_{nw} A_{we} = (0.75)(0.60 F_{EXX})\left(D/\sqrt{2}\right)(l_w)$$

The design of fillet welds in Canada is governed by CSA S16-09 Clause 13.13.2.2 and, although different coefficients are used, an identical resistance is obtained. The prior edition, CAN/CSA S16-01, included an additional check for shearing of the base metal at the edge of a fillet weld along the fusion face (see Figure 1(b)), which frequently governed and thus resulted in generally larger weld sizes. However, the current fillet weld design requirements for both AISC 360-10 and CSA S16-09 are based solely on the limit state of shear failure along the effective throat.



(a) 90° RHS T-connection under branch axial tension

(b) Detail of the fillet weld cross-section showing assumed failure planes

Figure 1. Comparison of fillet weld limit state design checks

2. HISTORICAL TREATMENT OF WELD DESIGN FOR RHS CONNECTIONS

In 1981 Subcommission XV-E of the International Institute of Welding (IIW) produced their first design recommendations for statically-loaded RHS 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 RHS structures, including those in Europe (CEN, 2005), Canada (Packer and Henderson, 1997) and the U.S. (AISC, 2010).

Research at the University of Toronto (Frater and Packer, 1992a, 1992b) on fillet-welded RHS branches in large-scale Warren trusses with gapped K-connections 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 RHS 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 formulae for the effective length of branch member welds in planar, gapped, RHS K- and N-connections, subject to predominantly static axial load, were taken in Packer and Henderson (1992) as:

When θ ≤ 50°: $$L_e = \frac{2H_b}{\sin\theta} + 2B_b \quad (1a)$$

When θ ≥ 60°: $$L_e = \frac{2H_b}{\sin\theta} + B_b \quad (1b)$$

In a further study by Packer and Cassidy (1995), by means of 16 large-scale connection tests which were designed to be weld-critical, new weld effective length formulae 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 formulae for the effective length of branch member welds in planar T-, Y- and Cross- (or X-) RHS connections, subjected to predominantly static axial load, were revised in Packer and Henderson (1997) to:

When θ ≤ 50°: $$L_e = \frac{2H_b}{\sin\theta} + B_b \quad (2a)$$

When θ ≥ 60°: $$L_e = \frac{2H_b}{\sin\theta} \quad (2b)$$

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

The latest (third) edition of the IIW recommendations (2012) requires that the design resistance of hollow section connections be based on failure modes that do not include weld failure, with the latter being avoided by satisfying either of the following criteria:

(i) Welds are to be proportioned to be “fit for purpose” and to resist forces in the connected members, taking account of connection deformation/rotation capacity and considering weld effective lengths, or

(ii) Welds are to be proportioned to achieve the capacity of the connected member walls.

This IIW (2012) document thus specifically acknowledges the effective length concept for weld design.

3. 2010 AISC SPECIFICATION, SECTION K4 WELD DESIGN PROCEDURES

In Section K4 of the AISC *Specification* (AISC, 2010) a detailed design method considering effective weld properties for various RHS connection types is given.

- **For T-, Y- and Cross- (or X-) connections under branch axial load or bending**
  Effective weld properties are given by:

$$L_e = \frac{2H_b}{\sin\theta} + 2b_{eoi} \tag{3}$$

$$S_{ip} = \frac{t_w}{3}\left(\frac{H_b}{\sin\theta}\right)^2 + t_w b_{eoi}\left(\frac{H_b}{\sin\theta}\right) \tag{4}$$

$$S_{op} = t_w\left(\frac{H_b}{\sin\theta}\right)B_b + \frac{t_w}{3}\left(B_b^2\right) - \frac{t_w/3\left(B_b - b_{eoi}\right)^3}{B_b} \tag{5}$$

$$b_{eoi} = \frac{10}{B/t}\left(\frac{F_y t}{F_{yb} t_b}\right)B_b \le B_b \tag{6}$$

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

In contrast to Equations 2a and 2b, the weld effective length in Equation 3 was – for consistency – made equivalent to the branch wall effective lengths used in Section K2.3 of the AISC *Specification* for the limit state of local yielding of the branch(es) due to uneven load distribution, which in turn is based on IIW (1989). The effective width of the weld transverse to the chord, b_{eoi}, is illustrated in Figure 2(b). This term, b_{eoi}, was empirically derived on the basis of laboratory tests in the 1970s and 1980s (Davies and Packer, 1982). The effective elastic section modulus of welds for in-plane bending and out-of-plane bending, S_{ip} and S_{op} respectively (Equations 4 and 5), apply in the presence of the bending moments, M_{ip} and M_{op} as shown in Figure 2(b).

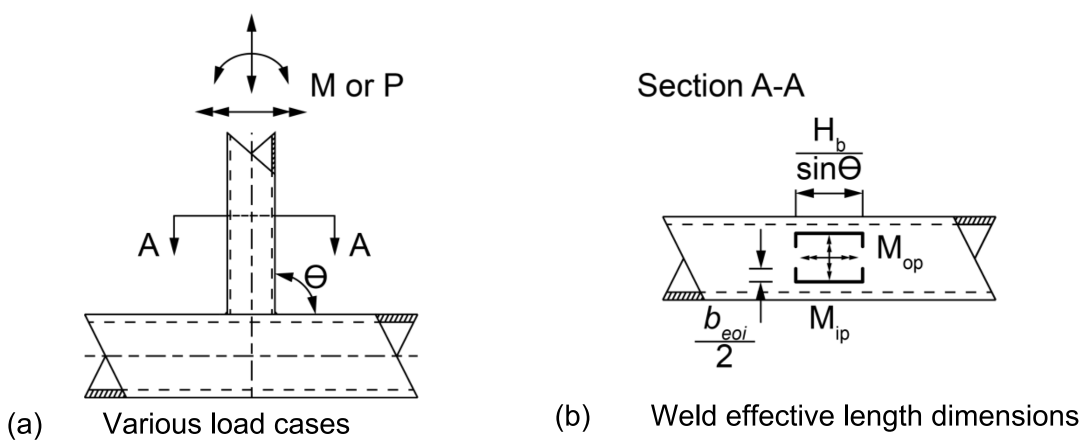


Figure 2. Weld effective length terminology for T-, Y- and Cross- (or X-) connections under branch axial load or bending

While being based on informed knowledge of general RHS connection behaviour, Equations 4 and 5 have not been substantiated by tests, and therefore are purely speculative.

- **For Gapped K- and N-Connections under Branch Axial Load**
  Effective weld lengths are given by:

When θ ≤ 50°: $$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b) \tag{7a}$$

When θ ≥ 60°: $$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b) \tag{7b}$$

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

Equations 7a and 7b are similar to Equations 1a and 1b but the former incorporate a reduction to allow for a typical RHS corner radius. The simplified nature of these effective length formulae (Equations 7a and 7b) was preferred, for gapped K- and N-connections, to the more complex ones that would result if the branch effective widths of the RHS walls in the AISC *Specification* Section K2.3 were adopted. Weld effective length provisions for overlapped RHS K- and N-connections were also provided in the AISC *Specification* Section K4 (AISC, 2010), based on branch effective widths of the RHS walls in Section K2.3, however in this case no research data on weld-critical overlapped RHS K- and N-connections was available.

The available strength of branch welds is determined, allowing for non-uniformity of load transfer along the line of weld, as follows by AISC (2010):

$$R_n \text{ or } P_n = F_{nw} t_w L_e \tag{8}$$

$$M_{n-ip} = F_{nw} S_{ip} \tag{9}$$

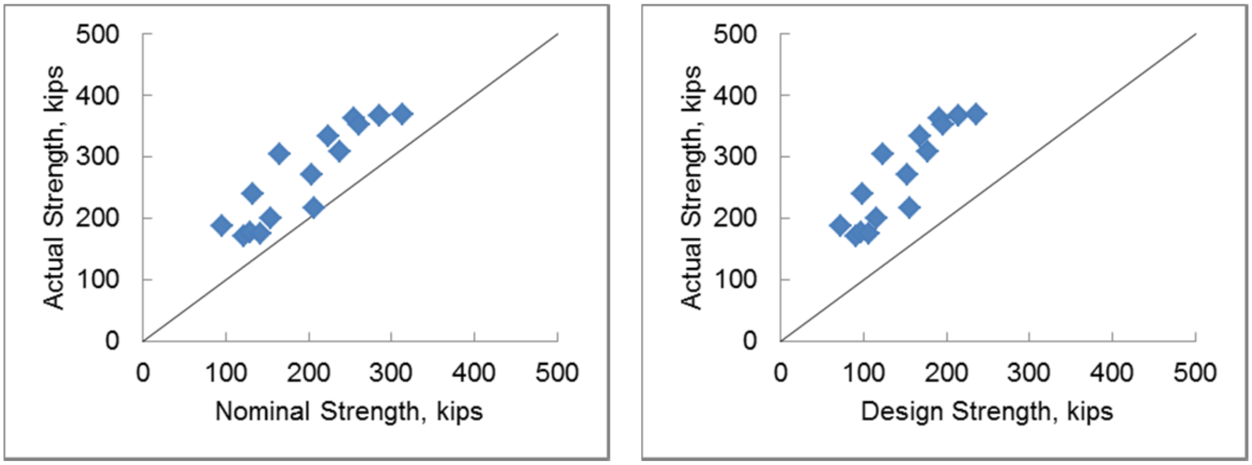
$$M_{n-op} = F_{nw} S_{op} \tag{10}$$

where,

$$F_{nw} = 0.60 F_{EXX} \tag{11}$$

4. EVALUATION OF AISC 2010 SPECIFICATION WITH EXPERIMENTS ON RHS WELDS UNDER PREDOMINANTLY AXIAL LOADS

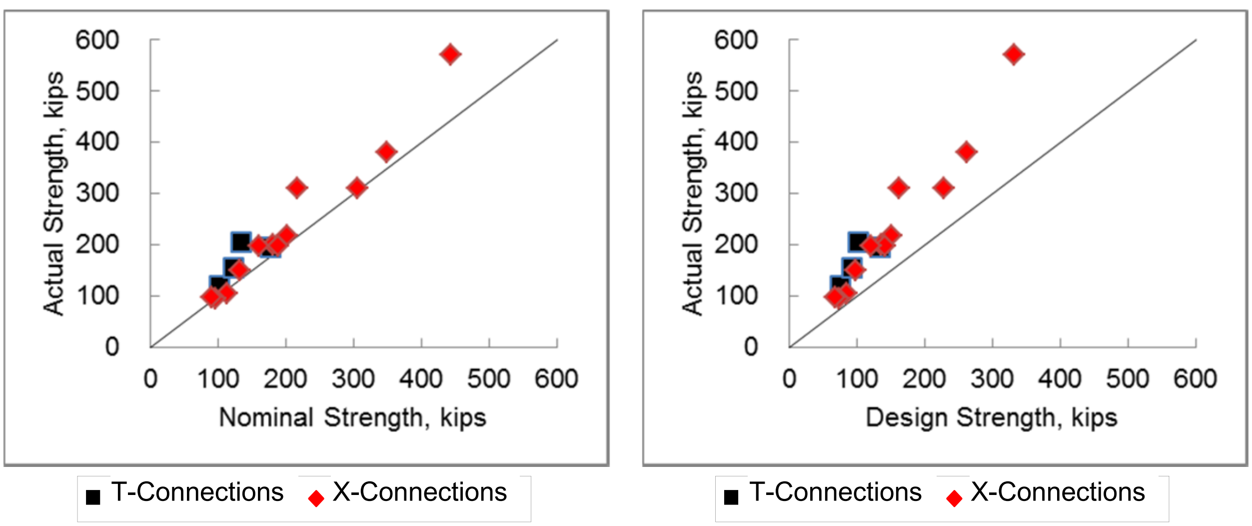
Two large-scale, 39.4-ft (12.0-m) and 40.0-ft (12.2-m) span, simply supported, fillet-welded, RHS Warren trusses, comprised of 60° gapped and overlapped K-connections, were tested by Frater and Packer (1992a, 1992b). Quasi-static loading was performed in a carefully controlled manner to produce sequential failure of the tension-loaded, fillet-welded connections (rather than connection failures). In addition, a series of weld-critical tests have been performed by Packer and Cassidy (1995) on four T-connections and 12 X-connections, with the branches loaded in quasi-static, axial tension. The effective leg sizes of the welds, measured along the branch member and chord member respectively, plus the throat sizes, were recorded. Measured geometric and mechanical properties of these trusses and welds and the failure loads of all welded connections are subsequently used herein to evaluate nominal weld strengths and predicted weld design strengths according to the AISC *Specification* with weld failure as the only limit state.



(a) Actual strength vs. Predicted nominal strength (R_n)

(b) Actual strength vs. Predicted LRFD strength ($0.75R_n$)

Figure 3. Correlation with test results for gapped K-connections without the inclusion of the ($1.00 + 0.50 \sin^{1.5}\theta$) term



(a) Actual strength vs. Predicted nominal strength (R_n)

(b) Actual strength vs. Predicted LRFD strength ($0.75R_n$)

Figure 4. Correlation with test results for T- and X-connections without inclusion of the ($1.00 + 0.5 \sin^{1.5}\theta$) term

Table J2.5, Section J4 (AISC, 2010) and Equations 3, 6, 7 and 8 were used to calculate the nominal strengths (excluding the resistance factor) of the 31 welded connections tested by Frater and Packer (1992a, 1992b) and Packer and Cassidy

(1995). The predicted strength of each welded connection, without a fillet weld directional strength increase of [$1.00 + 0.50 \sin^{1.5}\theta$] (discussed in the following section), was determined by the summation of the individual weld element strengths along the four walls around the branch footprint and is given as a predicted nominal strength, R_n.

In order to assess whether adequate, or excessive, safety margins are inherent in the correlations shown in Figures 3a and 4a, one can check to ensure that a minimum safety index of $\beta+ = 4.0$ (as currently adopted by AISC per Chapter B of the Specification Commentary) is achieved, using a simplified reliability analysis in which the resistance factor Φ is given by Equation 12 (Fisher et al., 1978); (Ravindra and Galambos, 1978).

$$\Phi = m_R \exp(-\alpha\beta^{+}COV) \tag{12}$$

where m_R = mean of the ratio: (actual element strength)/(nominal element strength = R_n); COV = associated coefficient of variation of this ratio; and α = coefficient of separation taken to be 0.55 (Ravindra and Galambos, 1978). Equation 12 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 12 produced $\Phi = 0.959$ for welded connections in gapped K-connections and $\Phi = 0.855$ for T- and X- (Cross-) connections. As both of these exceed $\Phi = 0.75$ the effective weld length concepts advocated in Section K4 of the AISC *Specification* can, on the basis of the available experimental evidence, be deemed adequately conservative.

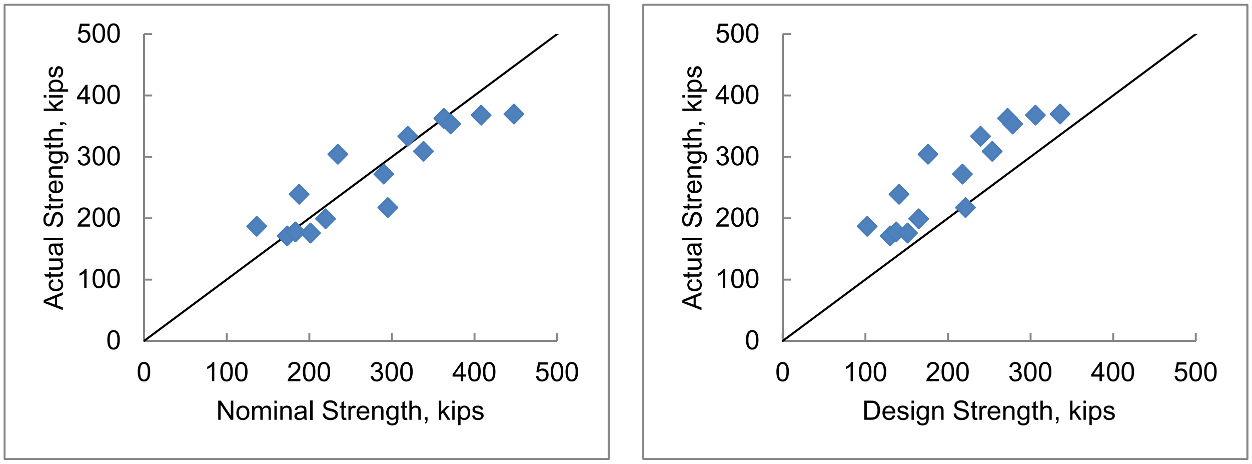
5. INTRODUCTION OF THE ($1.00 + 0.50 \sin^{1.5}\theta$) TERM

A debate about the application of an enhancement factor to the nominal strength of the weld metal (of $1.00 + 0.50 \sin^{1.5}\theta$) for fillet welds loaded at an angle of θ degrees to the weld longitudinal axis in hollow section connections has recently emerged. In the U.S., the AISC does not permit the fillet weld directional strength increase whereas in Canada, the CSA and CISC do not explicitly disallow it, so designers use it. Adopting this enhancement factor leads to a greater calculated resistance for a fillet weld group in a RHS connection and hence much smaller weld sizes (as demonstrated in Table 1).

The correlation plots in Figures 3 and 4 have been recomputed with weld metal failure as the only limit state and the inclusion of the ($1.00 + 0.5 \sin^{1.5}\theta$) in Figures 5 and 6. If the ($1.00 + 0.5 \sin^{1.5}\theta$) term is taken into consideration in the analysis of the data presented in this paper, the statistical outcomes change to:

- For gapped K-connections: $m_R = 0.999$, COV = 0.180 and $\Phi = 0.673$ (using Equation 12 with $\beta^{+} = 4.0$)
- For T- and X- (Cross-) connections: $m_R = 0.819$, COV = 0.164 and $\Phi = 0.571$ (using Equation 12 with $\beta^{+} = 4.0$).

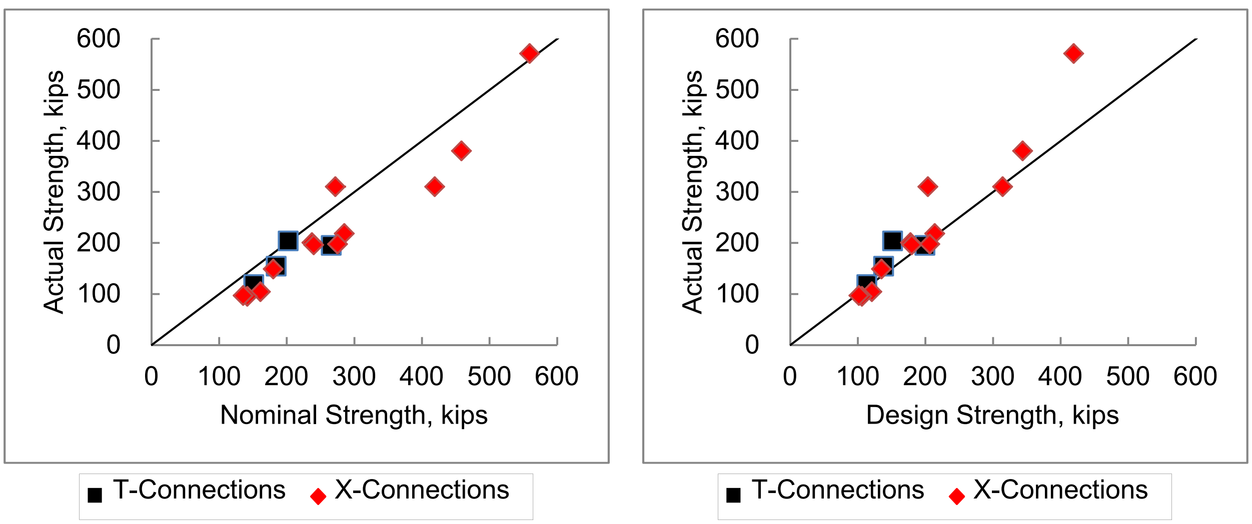
As both of these Φ-factors are below 0.75, the effective length formulae, with the (1.00 + 0.50sin1.5θ) term included, may be unsafe to use for fillet weld design.



(a) Actual strength vs. Predicted nominal strength (R_n)

(b) Actual strength vs. Predicted LRFD design strength ($0.75R_n$)

Figure 5. Correlation with test results for gapped K-connections with inclusion of the ($1.00 + 0.5 \sin^{1.5}\theta$) term



(a) Actual strength vs. Predicted nominal strength (R_n)

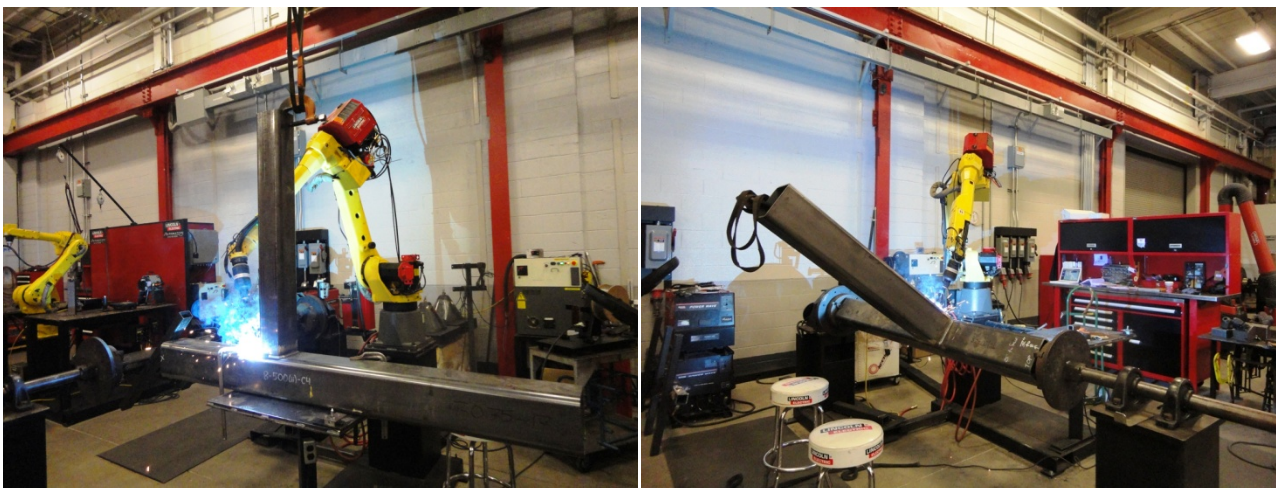
(b) Actual strength vs. Predicted LRFD design strength ($0.75R_n$)

Figure 6. Correlation with test results for T- and X-connections with inclusion of the ($1.00 + 0.5 \sin^{1.5}\theta$) term

6. CURRENT RESEARCH ON RHS MOMENT CONNECTIONS

A further experimental study to determine the weld effective length in RHS T-connections subject to branch in-plane bending moments is being carried out at the University of Toronto. The test specimens have been designed such that they are weld-critical under the application of branch in-plane bending moments (weld failure to precede connection failure). The bending moment at the connection is induced by the application of a lateral point load to the end of the branch in a quasi-static manner until weld failure. Key parameters such as branch-to-chord width ratios (β-ratios) of 0.25, 0.50, 0.75 and 1.00 with chord wall slenderness values of 17, 23 and 34 are being investigated. In order to determine the effectiveness of the weld in resisting the applied forces, the nonuniform distribution of normal strain and stress in the branch near the connection will be measured using strain gauges oriented along the longitudinal axis of the branch at numerous locations around the footprint. This will give a representative strain and stress distribution around the adjacent weld and hence the effectiveness of the weld can be determined. Based on the results of the experimental programme, the values postulated in Table K4.1 of the 2010 AISC *Specification* (AISC, 2010) will be verified or adjusted.

Fabrication of the specimens was performed at Lincoln Electric Co.'s Automation Division in Cleveland, Ohio. An experienced robotic welding technologist controlled a Fanuc Robot Arc-Mate 120iC 10L, adapted to perform the gas metal arc welding process with spray metal transfer (GMAW-P), to weld the connections. For the experimental programme, robotic welding offers several advantages: improved weld quality, excellent weld/base-metal fusion and root penetration, continuous electrodes, consistent travel speeds and the capability of welding in all positions.



(a) Stepped box connections welded in the horizontal position

(b) Matched box connections welded in the flat position using coordinated motion

Figure 7. Automated welding of specimens at Lincoln Electric Co.

The welding process parameters used were as follows: 0.035" diameter AWS ER70S-6 MIG wire, 23 Volts, 375 ipm wire feed speed, 90% Ar - 10% CO_2 shielding gas mixture at 30 to 50 CFH, ¼" to ½" contact tube to work distance and varying travel speeds depending on the weld type and size. Stepped connections ($\beta \leq 0.85$) were clamped to a level table and welded in the horizontal position as shown in Figure 7a. The matched connections ($\beta > 0.85$) were mounted to rotating chucks and welded in the flat position using coordinated motion, shown in Figure 7b, with fillet welds along the transverse branch walls and PJP flare-bevel-groove welds along the longitudinal branch walls.

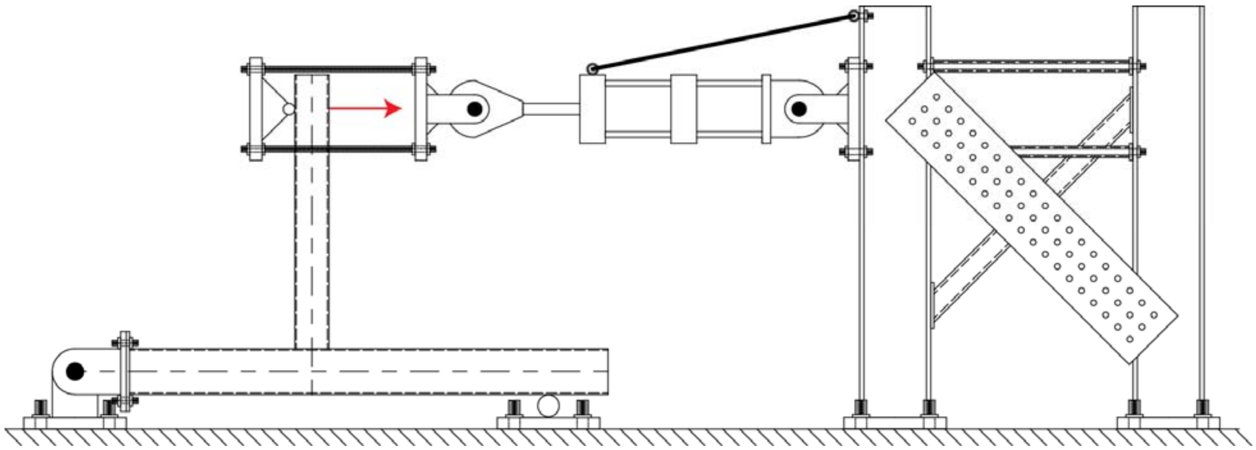


Figure 8. Elevation view of test setup assembly at the University of Toronto

With fabrication completed, the test specimens are at the University of Toronto Structural Testing Facilities undergoing instrumentation followed by full-scale testing. The test setup assembly, shown in Figure 8, consists of a simple-support for the test specimen (left) with an out-of-plane support frame (not shown) and a 77kip-capacity MTS Actuator (middle) mounted horizontally to a rigid steel frame (right).

7. CONCLUSIONS

Design guides or specifications/codes requiring the welds to develop the yield capacity of the branch members produce an upper limit on the required weld size and may be excessively conservative in some situations. While this is considered to be a simplified design method for fillet welds, it is shown that there is quite a disparity for the required effective throat size to develop the branch wall yield capacity. Additionally, the current fillet weld design requirements for both AISC 360-10 and CSA S16-09 are based solely on the limit state of weld metal shear failure along the effective throat whereas previous versions (CSA S16-01) included an additional check for shearing of the base metal at the edge of a fillet weld along the fusion face, which frequently govered and resulted in generally larger weld sizes.

Alternate design methods that consider weld effective lengths have the potential to provide a relatively smaller weld size, thus achieving a more economical design with increased aesthetic value. By comparing the actual strengths of fillet-welded joints in weld-critical T-, X- (Cross-) and gapped K- connection specimens to their predicted nominal strengths and design strengths, it has been shown that the relevant effective length design formulae 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. Conversely, it is shown that the inclusion of the ($1.00 + 0.50 \sin^{1.5}\theta$) term for such connections based solely on the limit state of weld failure along the effective throat of a fillet weld may be unsafe for design as it results 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 formulae for T-, Y- and X- (Cross-) connections in the AISC *Specification* Table K4.1 (AISC, 2010) also address branch bending. The available test data does not provide an opportunity to evaluate the accuracy of formulae applicable to branch bending loads and therefore the equations postulated are purely speculative. The objective of the research being performed at present at the University of Toronto is to verify or adjust these equations.

ACKNOWLEDGEMENTS

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NOTATION

A_{we}	effective (throat) area of the weld
B	overall width of RHS chord member, measured 90 degrees to the plane of the connection
B_b	overall width of RHS branch member, measured 90° to the plane of the connection
D	weld leg size
F_{EXX}	filler metal classification strength
F_{nw}	nominal stress of the weld metal
F_y	yield strength of the hollow section chord member material
F_{yb}	yield strength of the hollow section branch member material
H_b	overall height of RHS branch member, measured in the plane of the connection
L_e	effective length of groove and fillet welds to RHS for weld strength calculations
M_{ip}	in-plane bending moment
M_{op}	out-of-plane bending moment
$M_{n\text{-}ip}$	nominal weld resistance of in-plane bending moment
$M_{n\text{-}op}$	nominal weld resistance of out-of-plane bending moment
P_n	nominal strength of the welded joint
R_n	nominal strength of the welded joint
S_{ip}	weld effective elastic section modulus for in-plane bending
S_{op}	weld effective elastic section modulus for out-of-plane bending
b_{eoi}	effective width of the transverse branch face welded to the chord
l_w	weld length
m_R	mean of ratio: (actual element strength)/(nominal element strength) = professional factor
t	design wall thickness of hollow section chord member
t_b	design wall thickness of hollow section branch member
t_w	effective weld throat thickness
α	separation factor = 0.55
β	width ratio = the ratio of overall branch width to chord width for RHS connection
β^+	safety (reliability) index for LRFD and Limit States Design
θ	acute angle between the branch and chord (degrees); angle of loading measured from a weld longitudinal axis for fillet weld strength calculation (degrees)

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