Effective Weld Properties for RHS-to-RHS Moment T-Connections

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A thesis submitted in conformity with the requirements for
the degree of Master of Applied Science,
Graduate Department of Civil Engineering,
University of Toronto

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ABSTRACT

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Master of Applied Science (2012)

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An experimental program was developed to test various unreinforced RHS-to-RHS 90° T-connections subject to branch in-plane bending moment with the objective of determining the effectiveness of the welded joint. Twelve unique test specimens were designed to be weld-critical and the results from the full-scale tests revealed that the current equation for the effective elastic section modulus for in-plane bending, \( S_{ip} \), given in Table K4.1 of ANSI/AISC 360 (2010) is conservative. A modification to the current requirements that limit the effective width of the transverse weld elements is proposed, resulting in a safe and more economical weld design method for RHS-to-RHS T-, Y- and X- connections subject to branch axial load or bending moment. It is also concluded that the fillet weld directional strength enhancement factor, \((1.00 + 0.50\sin^{1.5}\Theta)\), should not be used for strength calculations of welded joints to square and rectangular hollow structural sections.
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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tr>
<td>( A_g )</td>
<td>Gross cross-sectional area of RHS member, in.(^2)</td>
</tr>
<tr>
<td>AISC</td>
<td>American Institute of Steel Construction</td>
</tr>
<tr>
<td>( A_m )</td>
<td>Area of fusion face between weld and base metal, in.(^2)</td>
</tr>
<tr>
<td>ASD</td>
<td>Allowable stress design method (AISC, 2010)</td>
</tr>
<tr>
<td>( A_{we} )</td>
<td>Effective throat area of the weld, in.(^2)</td>
</tr>
<tr>
<td>AWS</td>
<td>American Welding Society</td>
</tr>
<tr>
<td>( B )</td>
<td>Overall width of RHS chord, measured normal to the plane of the connection, in.</td>
</tr>
<tr>
<td>( B_b )</td>
<td>Overall width of RHS branch, measured normal to the plane of the connection, in.</td>
</tr>
<tr>
<td>CIDECT</td>
<td>Comité International pour le Développement et l’Etude de la Construction Tubulaire</td>
</tr>
<tr>
<td>CISC</td>
<td>Canadian Institute of Steel Construction</td>
</tr>
<tr>
<td>COV</td>
<td>Coefficient of variation</td>
</tr>
<tr>
<td>CSA</td>
<td>Canadian Standards Association</td>
</tr>
<tr>
<td>( E )</td>
<td>Effective weld throat according to AWS (2010), in.; Young’s Modulus, taken as ( 29 \times 10^3 ) ksi</td>
</tr>
<tr>
<td>( F )</td>
<td>Magnitude of Cartesian vector ( \vec{F} ), kip</td>
</tr>
<tr>
<td>( \vec{F} )</td>
<td>Cartesian vector of force, kip</td>
</tr>
<tr>
<td>( F_{EXX} )</td>
<td>Filler metal classification strength, ksi</td>
</tr>
<tr>
<td>( F_{nw} )</td>
<td>Nominal stress of weld metal, ksi</td>
</tr>
<tr>
<td>( F_u )</td>
<td>Ultimate tensile strength of RHS chord, ksi</td>
</tr>
<tr>
<td>( F_{ub} )</td>
<td>Ultimate tensile strength of RHS branch, ksi</td>
</tr>
<tr>
<td>( F_{w,Ed} )</td>
<td>Design value of the weld force per unit length according to EN 1993-1-8 (CEN, 2005), kip/in.</td>
</tr>
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Design weld resistance per unit length according to EN 1993-1-8 (CEN, 2005), kip/in.

- $F_{W,Rd}$
- $F_x$ Horizontal force component for force vector $\vec{F}$, kip
- $F_y$ Specified minimum yield stress of RHS chord, ksi; Vertical force component for force vector $\vec{F}$, kip
- $F_y^*$ = $F_y$ for T-connections
- $F_{yb}$ Specified minimum yield stress of RHS branch, ksi
- $F_z$ Force component for along the z-axis for the force vector $\vec{F}$, kip
- $H$ Overall height of RHS chord member, measured in the plane of the connection, in.
- $H_b$ Overall height of RHS branch member, measured in the plane of the connection, in.
- HSS Hollow structural section
- $I_{ip}$ Moment of inertia for in-plane bending, in.$^4$
- $I_{op}$ Moment of inertia for out-of-plane bending, in.$^4$
- $L_H$ Horizontal weld leg size measured from the root to the toe, in.
- LRFD Load and resistance factor design method (AISC, 2010)
- $L_V$ Vertical weld leg size measured from the root to the toe, in.
- LVDT Linearly varying differential transformer
- $M_{ip}$ Applied in-plane bending moment, kip-in.
- $M_{op}$ Applied out-of-plane bending moment, kip-in.
- $M_n$ Nominal flexural strength of connection (AISC, 2010), kip-in.
- $M_{n-b}$ Nominal flexural strength of the branch, kip-in.
- $M_{n-ip}$ Nominal flexural strength of weld for in-plane bending (AISC, 2010), kip-in.
- $M_{n-op}$ Nominal flexural strength of weld for out-of-plane bending (AISC, 2010), kip-in.
\( \overline{M}_0 \) Cartesian vector for moment, kip-in.

\( M_{0-x} \) Component of moment about the x-axis (out-of-plane bending), kip-in.

\( M_{0-y} \) Component of moment about the y-axis (torsion), kip-in.

\( M_{0-z} \) Component of moment about the z-axis (in-plane bending), kip-in.

\( M_{pl-b} \) Plastic moment capacity of the branch, kip-in.

\( M_u \) Actual flexural strength of welded joint (ultimate moment), kip-in.

\( P \) Applied force, kip

\( Q_f \) Chord-stress interaction parameter, equal to 1.0 for chord (connecting surface) in tension

RHS Rectangular hollow section

\( R_n \) Nominal strength of RHS member, ksi

\( R_{nw} \) Nominal strength of welded joint, ksi

SG Strain gage

\( S_b \) Elastic section modulus of branch about the axis of bending, in.\(^3\)

\( S_{ip} \) Effective elastic section modulus of weld for in-plane bending (AISC, 2010), in.\(^3\)

\( S_{op} \) Effective elastic section modulus of weld for out-of-plane bending (AISC, 2010), in.\(^3\)

TC Tensile coupon

\( T_r \) Factored tensile yield resistance of RHS member according to CSA (2001; 2009)

\( V_r \) Factored shear resistance for the weld metal according to CSA (2001; 2009)

WPS Welding procedure specifications

\( Z_b \) Plastic section modulus of branch about the axis of bending, in.\(^3\)

\( a \) Effective weld throat according to EN 1993-1-8 (CEN, 2005), in.

\( b \) Width of stiffened/ un-stiffened compression element, in.
$
\begin{align*}
 b_{eoi} & \quad \text{Effective width of the branch face welded to the chord, in.} \\
 f_{wv.d} & \quad \text{Design shear strength of the weld according to EN 1993-1-8 (CEN, 2005), ksi} \\
 \vec{i} & \quad \text{Unit vector component along the x-axis} \\
 \vec{j} & \quad \text{Unit vector component along the y-axis} \\
 \vec{k} & \quad \text{Unit vector component along the z-axis} \\
 l_e & \quad \text{Effective weld length of groove and fillet welds for RHS, in.} \\
 m_R & \quad \text{Mean of the ratio (actual element strength / nominal element strength)} \\
 r & \quad \text{Magnitude of position vector } \vec{r}, \text{ in.} \\
 \vec{r} & \quad \text{Position vector along the line of action of the applied force, in.} \\
 \vec{r}_0 & \quad \text{Position vector directed from the centroid of the connection to the line of action of the applied force, in.} \\
 r_{0-x} & \quad \text{Component of position vector } \vec{r}_0 \text{ along the x-axis, in.} \\
 r_{0-y} & \quad \text{Component of position vector } \vec{r}_0 \text{ along the y-axis, in.} \\
 r_{0-z} & \quad \text{Component of position vector } \vec{r}_0 \text{ along the z-axis, in.} \\
 t & \quad \text{Wall thickness of RHS chord member, in.} \\
 t_b & \quad \text{Wall thickness of RHS branch member, in.} \\
 t_w & \quad \text{Effective weld throat around the perimeter of the branch, in.} \\
 \vec{u} & \quad \text{Unit vector} \\
 x_A & \quad \text{X-coordinate of the tail for position vector } \vec{r}, \text{ in.} \\
 x_B & \quad \text{X-coordinate of the arrowhead for position vector } \vec{r}, \text{ in.} \\
 y_A & \quad \text{Y-coordinate of the tail for position vector } \vec{r}, \text{ in.} \\
 y_B & \quad \text{Y-coordinate of the arrowhead for position vector } \vec{r}, \text{ in.} \\
 z_A & \quad \text{Z-coordinate of the tail for position vector } \vec{r}, \text{ in.} \\
 z_B & \quad \text{Z-coordinate of the arrowhead for position vector } \vec{r}, \text{ in.}
\end{align*}
\(\alpha\)  Coefficient of separation (taken to be 0.55)

\(\beta\)  Width ratio; the ratio of overall branch width to chord width for RHS

\(\beta^+\)  Safety index

\(\beta_w\)  Correlation factor for fillet welds according to EN 1993-1-8 (CEN, 2005)

\(\varepsilon_u\)  Elongation at rupture, ultimate strain, in./in.

\(\varepsilon_y\)  Strain at material yield point, in./in.

\(\eta\)  Load length parameter; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width

\(\lambda_p\)  Limiting slenderness parameter for compact element (AISC, 2010)

\(\lambda_r\)  Limiting slenderness parameter for non-compact element (AISC, 2010)

\(\phi\)  Resistance factor (associated with the load and resistance factor design method)

\(\phi_w\)  Resistance factor for welded joints according to CSA (2001; 2009) equal to 0.67

\(\gamma_{M0}\)  Partial safety factor equal to 1.00 (CEN, 2005)

\(\gamma_{M2}\)  Partial safety factor for the resistance of welds equal to 1.25 (CEN, 2005)

\(\Omega\)  Safety factor (associated with the allowable stress design method)

\(\sigma_{\parallel}\)  Normal stress parallel to the longitudinal axis of the weld (CEN, 2005), ksi

\(\sigma_{\perp}\)  Normal stress perpendicular to the effective weld throat (CEN, 2005), ksi

\(\tau_{\parallel}\)  Shear stress parallel to the longitudinal axis of the weld (CEN, 2005), ksi

\(\tau_{\perp}\)  Shear stress perpendicular to the longitudinal axis of the weld (CEN, 2005), ksi

\(\Theta\)  Included angle between the branch and chord, degrees; Angle of loading measured from the weld longitudinal axis, degrees
Chapter 1: Introduction

The design of tubular structures has been of significant interest since the 1950s, initiating international research programs to investigate stability, fire protection, wind loading, composite construction, and static and fatigue behavior of connections. Much of the research has been sponsored by the International Committee for the Development and Study of Tubular Construction (CIDECT) and the results have been reported and incorporated into national/ international codes, standards and guidelines including the American Institute of Steel Construction (AISC) Specification for Structural Steel Buildings (2010), Canadian Institute of Steel Construction (CISC) guide (Packer & Henderson, 1997) and Eurocode 3: Design of steel structures (CEN, 2005).

Figure 1-1 Applications of HSS in exposed steel structures

Hollow structural sections (HSS) are used in a wide variety of structural applications such as building construction, bridges, roller coasters, offshore oil rigs, retractable stadium roofs (Figure 1-1(a)), power transmission towers and many others. They are used quite often in building structures (such as airport terminals, Figure 1-1(b)) that have exposed architectural steel-work because of their aesthetically pleasing visuals especially, when bent along their longitudinal axis, giving an enhanced visual impact.
Aside from aesthetics, HSS have several other benefits including (STI, 2005):

- **Strength** – high strength-to-weight ratios, excellent torsional resistance due to the closed shape, excellent compression support characteristics.
- **Ease of manufacture/fabrication** – can be readily formed into various cross-sectional shapes (round, square, elliptical), bent about the longitudinal axis, welded together or to other structural steel products, punched and drilled.
- **Weight/cost effectiveness** – because of its high strength-to-weight ratio in compression, HSS can provide significant column weight reductions which may lead to reduced costs associated with transportation, fabrication and erection.
- **Fire resistance** – when filled with concrete or coated with fire retardant paint, HSS can have exceptional resistance to flames.
- **Environmentally friendly** – steel is recyclable and in some cases, reusable unlike other structural materials.

### 1.1. Vierendeel Truss Connections between RHS

Vierendeel trusses were first proposed in 1896 by a Belgian engineer, Arthur Vierendeel, whose vision was to have “open” web girders comprised of horizontal top and bottom chord members welded to vertical branch members (Figure 1-2(a)). Traditional trusses, such as Warren or Pratt trusses, are comprised of diagonal branch members that carry predominantly axial forces and the connections are nearly pin-jointed at their ultimate limit state. Due to the absence of diagonal members, the branches in Vierendeel trusses are subjected to substantial bending moments as well as axial and shear forces at the connections. Hence, such connections cannot be considered pin-jointed. Because of this, earlier versions of Vierendeel trusses were difficult to design and often required heavily reinforced connections between welded wide flange sections that were not aesthetically pleasing. With the development of HSS, the potential for these trusses has greatly improved since they can add aesthetic value as well as structural efficiency (when designed properly).

A substantial amount of testing has been performed on isolated Vierendeel connections (typically referred to as moment T-connections) between rectangular hollow sections.
(RHS) since the 1960s. Most of the specimens were tested in an inverted T position with a lateral point load applied to the end of the branch member (Figure 1-2(b)) inducing bending moment and shear force at the connection. A very limited number of tests subjecting such connections to bending, shear and axial loading has been performed.

Researchers have observed that both the flexural strength and rigidity of an unreinforced welded connection decrease as the branch-to-chord width ratio ($\beta$) decreases and as the chord wall slenderness value ($B/t$) increases (Packer & Henderson, 1997). Matched connections ($\beta = 1.0$) with a low $B/t$ value can potentially achieve almost full rigidity, however all other unreinforced connections should be classified as semi-rigid, as they are unable to develop the moment capacity of the branch member.

1.2. Design Philosophy for Welded Joints between RHS

The design criteria for fillet welds have evolved over the years as more data becomes available through experimental research. With welded connections between RHS there are currently two design methods 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 perimeter.
(ii) The welds may be designed as “fit-for-purpose”, and proportioned to resist the applied forces in the branch.

The latter design method has emerged as a result of highly non-uniform distributions of normal stress observed around the branch perimeter adjacent to the welded joint. This is due to the relative flexibility of the connecting RHS face and requires the use of effective weld properties which are a focus of this research project. Since this research project is focused on RHS-to-RHS moment T-connections, the equation for the effective elastic section modulus for in-plane bending postulated in Section K4 of the *AISC Specification* (2010) is being investigated.

### 1.3. Experimental Program Overview

An experimental program was developed at the University of Toronto to test various unreinforced RHS-to-RHS 90° T-connections subject to branch in-plane bending moment. The objective was to investigate the effectiveness of the weld in resisting the applied forces at the ultimate limit state of weld rupture along the plane of the effective throat and verify or adjust the current effective weld properties postulated in Section K4 (*AISC*, 2010) for such connections. Twelve test specimens were designed to be weld-critical under the application of branch in-plane bending moment. Key parameters such as branch-to-chord width ratios ($\beta$-ratios) ranging from 0.25 to 1.00 and chord wall slenderness values of 17, 23 and 34 were investigated. The experimental program also includes tensile coupon tests of the RHS material and as-laid weld metal.

The following chapters summarize previous research performed on weld-critical connections between RHS and the historical development of the effective length concept, a detailed description of the experimental program and the results/analysis from full-scale experiments. Conclusions and recommendations based on the analysis of the results are summarized in the final chapter.
Chapter 2: Relevant Research and Current Design Criteria

This chapter provides an overview of the information that is currently available for the design of unreinforced welded RHS-to-RHS moment T-connections. The connection behavior, strength and flexibility are briefly discussed prior to a comparative analysis of the different effective weld throats required to develop the yield strength of the connected branch member wall for various national and international codes, specifications and guidelines. A discussion about the historical development of effective weld properties including full-scale tests performed on weld-critical RHS-to-RHS gapped K-connections as well as T-, Y- and X- (Cross-) connections follows. Finally, the current effective weld properties used to calculate the strength of welded joints for connections between RHS, with a brief introduction to the controversial \((1.00 + 0.50 \sin^{1.5} \Theta)\) factor, are explained and evaluated.

2.1. Connection Behavior, Strength and Flexibility


Researchers have observed that both the flexural strength and rigidity of an unreinforced welded connection decrease as the branch-to-chord width ratio \((\beta)\) decreases and as the chord wall slenderness value \((B/t)\) increases (Packer & Henderson, 1997). Matched connections \((\beta = 1.0)\) with a low \(B/t\) value achieve almost full rigidity however all other unreinforced connections should be classified as semi-rigid, as they are unable to develop the moment capacity of the branch member.
Figure 2-1 Possible failure modes for RHS-to-RHS moment T-connections (Wardenier, 1982; Packer et al., 2009)
It was also observed that the flexibility of the chord face resulted in a highly non-uniform distribution of normal stress around the branch perimeter at the connection and that the failure mode was heavily influenced by the β-ratio as well as the chord wall slenderness value. The possible failure modes for RHS-to-RHS moment T-connections are (Wardenier, 1982; Packer et al., 2009):

(a) Chord face plastification
(b) Cracking in the chord (punching shear)
(c) Local yielding in the branch(es) due to uneven load distribution
(d) Chord sidewall local yielding
(e) Chord shear failure

These are represented in Figure 2-1. Since cracking in the chord (b) occurs after the limit state of chord face plastification, it is not considered a critical limit state and hence is not a design consideration. Additionally, chord shear failure is considered a member failure, not a connection failure, and is not a design consideration.

2.2. Strength of RHS-to-RHS Moment T-Connections

A RHS-to-RHS moment T-connection, as defined by ANSI/AISC 360 (2010), is a connection comprised of one branch welded perpendicular to a continuous chord passing through the connection, with the branch loaded by bending moment. These types of connections are applicable to Vierendeel truss-type frames and are considered partially- or fully-rigid depending on the connection flexibility. The centerlines of the branch and chord members must lie in a common plane for the following design criteria to be applicable.

The strength of RHS-to-RHS moment T-connections is addressed in Section K3 of ANSI/AISC 360 (2010) however the available test data for such connections is much less extensive than that for axially-loaded RHS-to-RHS T-, Y-, X- (Cross-) and gapped K-connections and hence the governing limit states for the latter are used as a basis for the former. Thus, the design criteria for RHS-to-RHS connections subject to branch in-plane bending are based on the limit states of chord wall plastification, chord sidewall
local yielding and local yielding in the branch(es) due to uneven load distribution. The strength of axially-loaded Y-, X- (Cross-) connections and gapped or overlapped K-connections will not be considered herein.

Two basic design methods may be used in conjunction with the above-mentioned limit states to determine the strength of structural steel members and connections (AISC, 2010):

1) Allowable Stress Design (ASD): The nominal strength of a structural component is divided by a safety factor, $\Omega$, and the resulting allowable strength is then required to equal or exceed the required strength of the component determined by structural analysis for the appropriate ASD load combinations specified by the applicable building code.

2) Load and Resistance Factor Design (LRFD): The nominal strength of a structural component is multiplied by a resistance factor, $\phi$, and the resulting design strength is then required to equal or exceed the required strength of the component determined by structural analysis for the appropriate LRFD load combinations specified by the applicable building code.

While the ASD method is still acceptable for use in practice, it has generally been replaced by the modern LRFD method. In addition, the LRFD method is more comparable to design methods used in current Canadian standards and design guides. Hence, only the LRFD method is considered for this section.

Thus, the LRFD flexural strength for RHS-to-RHS T-connections, within the limits of applicability of Table K3.2A (AISC, 2010), is taken as the lowest value calculated from Table 2-1 for the applicable connection geometric configurations.

The equations in Table 2-1 are in accord with the European Standard, EN-1993-1-8 (CEN, 2005), and have been adopted in the most recent CIDECT Design Guide No. 3 (Packer et al., 2009) as well as the Canadian design guide (Packer & Henderson, 1997).
Table 2-1 Strength of RHS-to-RHS moment T-connections according to ANSI/AISC 360 (2010)

<table>
<thead>
<tr>
<th>Limit State</th>
<th>$M_n$</th>
<th>$\varnothing$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord wall plastification $\beta \leq 0.85$</td>
<td>$F_y t^2 H_b \left[ \frac{1}{2\eta} + \frac{2}{\sqrt{1 - \beta}} + \frac{\eta}{(1 - \beta)} \right] Q_f$</td>
<td>1.00</td>
</tr>
<tr>
<td>Sidewall local yielding $\beta &gt; 0.85$</td>
<td>$0.5 F_y^* t (H_b + 5t)^2$</td>
<td>1.00</td>
</tr>
<tr>
<td>Local yielding of branch $\beta &gt; 0.85$</td>
<td>$F_{yb} \left[ Z_b - \left( 1 - \frac{b_{eol}}{B_b} \right) B_b H_b t_b \right]$</td>
<td>0.95</td>
</tr>
</tbody>
</table>

where,

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

$Q_f = 1$ For chord (connecting surface) in tension

$F_y^* = F_y$ For T-connections

![Figure 2-2 Geometric dimensions for connections between RHS](image-url)
The geometric dimensions for connections between RHS are shown in Figure 2-2.

In addition to the connection available flexural strength calculations, the branch flexural strength (overall member strength) must be calculated in accordance with Section F7 (AISC, 2010). The nominal flexural strength is taken as the lowest value calculated based on the limit states of full cross-sectional yielding (plastic moment), flange local buckling and web local buckling. These depend on whether the webs are compact or non-compact and whether the flanges are compact, non-compact or slender (defined in Section B4.1 of ANSI/ AISC 360-10). The LRFD resistance factor ($\phi$) for RHS under moment loading is equal to 0.90. The lowest calculated value of connection and branch flexural strength governs.

2.3. Design Methods for Welded Joints of RHS-to-RHS T-Connections

The design criteria for fillet welds have evolved over the years as more data has become available through experimental research. While often viewed as simplistic in nature, the way in which a fillet weld transfers the load through a connection can be complex, especially in semi-rigid connections between RHS. Since welding can only be performed around the outer perimeter of the RHS walls, the fillet welds are inherently eccentrically-loaded which causes secondary bending moments at the root (Figure 2-3). Because the majority of connections between RHS are fillet-welded, it is important to examine the current design criteria in various steel codes, specifications and standards.

With welded connections between RHS there are currently two design methods 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 perimeter. 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 highly non-uniform distribution of stress 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.

![Figure 2-3 Eccentrically-loaded fillet weld under axial tension](image)

The following section includes a comparison of the current design criteria using method (i), for the design of fillet welds in connections between RHS, for various national and international steel codes, specifications and guidelines. The vast disparity in the current design criteria becomes apparent.

Since the primary focus of this research project is on method (ii), a summary of the historical treatment of weld design for RHS connections up to and including the development of the current effective weld properties is included. Finally, an introduction to the \((1.00 + 0.50 \sin^{1.5}\Theta)\) factor, or fillet weld directional strength enhancement factor, is presented and analysed using the data from weld-critical experiments.
2.3.1. Design Method (i) – Develop the Branch Yield Strength

In this section, the design methods for fillet welds to develop the yield strength of the connected branch wall at all locations around the perimeter in various national and international codes, specifications and guidelines are compared. The equations are rearranged to solve for the minimum required effective weld throat per unit length of the weld in terms of the branch wall thickness for the simple case of an axially-loaded RHS-to-RHS T-connection \( \theta = 90^\circ \), shown in Figure 2-4 (a), using the equivalent of cold-formed RHS made to ASTM A500 Grade C with matching electrodes. The objective is to demonstrate the disparity of the fillet weld design criteria.

2.3.1.1. American specifications and codes

In the United States there are two governing references for designing fillet welded joints between RHS: the *AISC Specification for Structural Steel Buildings* (2010) and the *American Welding Society (AWS) Structural Welding Code - Steel* (2010). The design criteria for both are based on the limit state of shear rupture along the plane of the effective weld throat.

2.3.1.1.1. ANSI/AISC 360 (2010)

The design strength of a fillet welded joint (Equation 2.2) is taken as the product of the nominal weld stress \( F_{nw} \) and effective weld area \( A_{we} \) with a resistance factor applied,

\[
\begin{align*}
\phi R_{nw} & = 0.75 \cdot (F_{nw} \cdot A_{we}) \\
\phi R_{nw} & = 0.75 \cdot (0.60 \cdot F_{EXX} \cdot A_{we})
\end{align*}
\]

where the effective area of a fillet weld is defined as the effective length \( l_e \) multiplied by the shortest distance from the root to the face of the diagrammatic weld (effective throat, \( t_w \)). By setting the design strength of a fillet welded joint equal to that of the yield strength for the connected branch member for cold-formed RHS made to ASTM A500 Grade C with matching electrodes, the minimum required effective throat (per unit length of fillet weld) can be calculate in terms of the connected branch wall thickness (per unit length of the branch perimeter):
\[ \phi R_{nw} = \phi R_n \]
\[ 0.75 \cdot (0.60 \cdot 70 \text{ ksi} \cdot t_w) = 0.90 \cdot (50 \text{ ksi} \cdot t_b) \]
\[ \therefore t_w = 1.43 \cdot t_b \]

2.3.1.1.2. **ANSI/AWS D1.1/D1.1M: 2010**

The *Structural Welding Code* (AWS, 2010) requires welds that are capable of developing the lesser of the branch member yield strength or local strength of the chord member to prevent “unzipping” or progressive failure of the weld and thus ensure ductile behavior of the joint. For fillet welded connections between cold-formed RHS made to ASTM A500 Grade C with matching electrodes, AWS gives a prequalified effective throat size \((E)\) in terms of the connected branch wall thickness (Equation 2.4).

\[ E = t_w = 1.07 \cdot t_b \]

It is interesting to see the difference in the required effective throat size for a fillet weld to develop the branch member yield strength, between ANSI/AISC 360 (2010) and ANSI/AWS D1.1 (2010). The following sections will further demonstrate the disparity amongst other national and international codes, specifications and guidelines.

2.3.1.2. **Canadian standards**

This section compares the limit state checks for calculating the factored resistance of fillet welds using the current design standard, CSA S16 (2009), and its previous version, CAN/CSA S16 (2001). The effective weld throat required to develop the factored yield resistance of the branch is calculated herein.

2.3.1.2.1. **CAN/CSA S16 (2001)**

The factored resistance of fillet welds according to the 2001 Canadian design standard (CSA, 2001) was taken as the lesser of two limit states: (i) shear rupture along the fusion face with the base metal and (ii) shear rupture along the plane of the effective weld throat. Figure 2-4 (b) shows the difference between both limit states.
For limit state (i), by setting the factored shear resistance for the base metal equal to the
factored tensile yield resistance of the connected branch member for cold-formed RHS
made to ASTM A500 Grade C with matching electrodes, the minimum required effective
throat (per unit length of fillet weld) can be calculated in terms of the connected branch
wall thickness (per unit length of the branch perimeter):

\[ V_r = T_r \]

\[ 0.67 \cdot \phi_w A_m F_{ub} = \phi A_g F_{yb} \]

\[ 0.67 \cdot 0.67 \cdot (\sqrt{2} \cdot t_w) \cdot 62 \text{ ksi} = 0.90 \cdot t_b \cdot 50 \text{ ksi} \]

\[ \therefore t_w = 1.14 \cdot t_b \]

Limit state (ii) permits (or rather, does not specifically exclude) a directional strength
enhancement factor to the factored shear resistance of the weld based on the angle (in
degrees) of the axis of the weld segment with respect to the line of action of the applied
force. For an axially loaded 90° T-connection this angle (\( \theta \)) is equal to 90° which
increases the overall factored shear resistance of the weld by a factor of 1.5. By setting
the factored shear resistance for the weld metal equal to the factored tensile yield
resistance of the connected branch member for cold-formed RHS made to ASTM A500
Grade C with matching electrodes, the minimum required effective throat (per unit
length of fillet weld) can be calculated in terms of the connected branch wall thickness
(per unit length of the branch perimeter):

\[ V_r = T_r \]

\[ 0.67 \cdot \phi_w A_w F_{EXX} (1.00 + 0.50 \sin^{1.5} \theta) = \phi A_g F_{yb} \]

\[ 0.67 \cdot 0.67 \cdot t_w \cdot 70 \text{ ksi} \cdot 1.5 = 0.90 \cdot t_b \cdot 50 \text{ ksi} \]

\[ \therefore t_w = 0.95 \cdot t_b \]

Therefore, the limit state of shear rupture along the fusion face with the base metal
governs over shear rupture along the plane of the effective throat.

2.3.1.2.2. CSA S16 (2009)

The factored resistance of fillet welds according to the 2009 Canadian design standard
(CSA, 2009) is based solely on the limit state of shear rupture along the plane of the
effective throat. A comparison of the assumed failure planes for the 2001 and 2009 design standards is shown in Figure 2-4 (b).

By setting the factored shear resistance for the weld metal equal to the factored tensile yield resistance of the connected branch member for cold-formed RHS made to ASTM A500 Grade C with matching electrodes, the minimum required effective throat (per unit length of fillet weld) equals the same as in Equation 2.6. The check for shear rupture along the fusion face with the base metal is not required, even though it governs in the case presented in Section 2.3.1.2.1.

![Figure 2-4 Comparison of fillet weld limit state design checks between CAN/CSA S16 (2001) and CSA S16 (2009)](image)

2.3.1.3. European Standard

This section compares the two design methods used for determining the resistance of fillet welds according to the European standard, EN-1993-1-8 (CEN, 2005): the directional method and the simplified method. The design weld throats required to develop the yield resistance of the branch for both methods are calculated herein.
2.3.1.3.1. **Design resistance of fillet welds according to the directional method**

The directional method of EN 1993-1-8 (CEN, 2005) is based on the Von Mises yield criterion and divides the forces transmitted by a unit length of weld into components of stress parallel and transverse to the longitudinal axis of the weld and normal and transverse to the plane of its design throat.

The design throat thickness and hence area ($A_{we}$) are assumed to be measured from the root of the fillet weld and are calculated using Equation 2.7:

$$A_{we} = \sum a \cdot l_e$$  \hspace{1cm} 2.7

where the effective length of a fillet weld ($l_e$) is taken as the length over which the fillet is full-size and the effective throat thickness ($a$) is taken as the height of the largest triangle (with equal or unequal legs) that can be inscribed within the fusion faces and the weld surface, measured perpendicular to the outer side of this triangle, shown in Figure 2-5 for equal and unequal leg sizes (a) as well as joints with consistent deep root penetration (b).

![Figure 2-5 Effective throat thickness (a) of a fillet weld according to EN 1993-1-8 (CEN, 2005)](image)

The normal stresses perpendicular to the throat ($\sigma_\perp$) and parallel to the longitudinal axis of the weld ($\sigma_\parallel$) as well as the shear stresses (in the plane of the throat) perpendicular to the axis of the weld ($\tau_\perp$) and parallel to the axis of the weld ($\tau_\parallel$), shown in Figure 2-6 (a) are assumed to be uniformly distributed over the design throat area.
For the case of an axially-loaded RHS-to-RHS T-connection, the resultant planar normal stress and shear stress (per unit length of weld) acting on the effective throat under an applied load, $P$, are as shown in Figure 2-6(b).

The design resistance of the fillet weld is sufficient if Equations 2.8 and 2.9 are satisfied:

$$\sqrt{\sigma_{\perp}^2 + 3 \cdot \left(\tau_{\parallel}^2 + \tau_{\perp}^2\right)} \leq \frac{F_{ub}}{(\beta_w \cdot \gamma_{M2})}$$ \hspace{1cm} 2.8

$$\sigma_{\perp} \leq 0.9 \cdot \frac{F_{ub}}{\gamma_{M2}}$$ \hspace{1cm} 2.9

with $F_{ub}$ being the nominal tensile strength of the weaker section joined (assumed to be the branch in this case), the correlation factor for fillet welds ($\beta_w$) taken from Table 2-2 and the partial safety factor for the resistance of welds ($\gamma_{M2}$) taken equal to 1.25 (CEN, 2005).

---

Figure 2-6 Forces and stresses acting on the design throat area of a fillet weld according to EN 1993-1-8 (CEN, 2005)
Table 2-2 Correlation factor ($\beta_w$) for fillet welds according to EN-1993-1-8 (CEN, 2005)

<table>
<thead>
<tr>
<th>Standard and steel grade</th>
<th>Correlation factor $\beta_w$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EN 10025</strong> (hot rolled structural steel products)</td>
<td><strong>EN 10210</strong> (hot finished structural hollow sections)</td>
</tr>
<tr>
<td>S 235</td>
<td>S 235 H</td>
</tr>
<tr>
<td>S 235 W</td>
<td></td>
</tr>
<tr>
<td>S 275</td>
<td>S 275 H</td>
</tr>
<tr>
<td>S 275 N/NL</td>
<td>S 275 NH/NLH</td>
</tr>
<tr>
<td>S 275 M/ML</td>
<td></td>
</tr>
<tr>
<td>S 355</td>
<td>S 355 H</td>
</tr>
<tr>
<td>S 355 N/NL</td>
<td>S 355 NH/NLH</td>
</tr>
<tr>
<td>S 355 M/ML</td>
<td></td>
</tr>
<tr>
<td>S 355 W</td>
<td></td>
</tr>
<tr>
<td>S 420 N/NL</td>
<td>-</td>
</tr>
<tr>
<td>S 420 M/ML</td>
<td></td>
</tr>
<tr>
<td>S 460 N/NL</td>
<td>S 460 NH/NLH</td>
</tr>
<tr>
<td>S 460 M/ML</td>
<td></td>
</tr>
<tr>
<td>S 460 Q/QL/QL1</td>
<td></td>
</tr>
</tbody>
</table>

For the requirement of Equation 2.8, setting the design resistance of a fillet welded joint equal to that of the yield resistance of the connected branch member for cold-formed RHS made to EN 10219 – S355H with matching electrodes, the minimum required effective throat (per unit length of fillet weld) in terms of the connected branch wall thickness (per unit length of the branch perimeter) gives:

$$\frac{F_{ub}}{(\beta_w \cdot \gamma_{M2})} \geq \sqrt{4 \cdot \frac{P}{\left(\sqrt{2} \cdot a \cdot \gamma_{MO}\right)}}$$

$$\frac{F_{ub}}{(\beta_w \cdot \gamma_{M2})} \geq \frac{2}{\sqrt{2}} \cdot \frac{P}{(a \cdot \gamma_{MO})}$$

$$\frac{F_{ub}}{(\beta_w \cdot \gamma_{M2})} \geq \sqrt{2} \cdot \frac{F_{yb} \cdot t_b}{(a \cdot \gamma_{MO})}$$

$$a \geq \sqrt{2} \cdot \beta_w \cdot \frac{F_{yb} \cdot \gamma_{M2}}{F_{ub} \cdot \gamma_{MO} \cdot t_b}$$

2.10
Chapter 2: Relevant Research and Current Design Criteria

\[ a \geq \sqrt{2} \cdot 0.9 \cdot \frac{355 \text{ MPa} \cdot 1.25}{510 \text{ MPa} \cdot 1.00} \cdot t_b \]
\[ \therefore a = t_w \geq 1.11 \cdot t_b \]

Similarly, for the requirement of Equation 2.9:

\[ \frac{0.9 \cdot F_{ub}}{\gamma_{M2}} \geq \frac{P}{\left(\sqrt{2} \cdot a \cdot \gamma_{MO}\right)} \]
\[ \frac{0.9 \cdot F_{ub}}{\gamma_{M2}} \geq \frac{F_{yb} \cdot t_b}{\left(\sqrt{2} \cdot a \cdot \gamma_{MO}\right)} \]
\[ a \geq \left(\frac{1}{0.9 \cdot \sqrt{2}}\right) \cdot \frac{F_{yb} \cdot \gamma_{M2}}{F_{ub} \cdot \gamma_{MO}} \cdot t_b \]
\[ a \geq \left(\frac{1}{0.9 \cdot \sqrt{2}}\right) \cdot \frac{355 \text{ MPa} \cdot 1.25}{510 \text{ MPa} \cdot 1.00} \cdot t_b \]
\[ \therefore a = t_w \geq 0.68 \cdot t_b \]

Therefore the governing minimum effective throat size using the directional method of EN-1993-1-8 (CEN, 2005) is 1.11 times the wall thickness of the connected branch member.

For a more direct comparison between the national/ international codes, specifications and guidelines, Equations 2.10 and 2.11 are re-calculated for cold-formed RHS made to ASTM A500 Grade C. Thus, the minimum required effective throat (per unit length of fillet weld) in terms of the connected branch wall thickness (per unit length of the branch perimeter) gives:

\[ a \geq \sqrt{2} \cdot \beta_w \cdot \frac{F_{yb} \cdot \gamma_{M2}}{F_{ub} \cdot \gamma_{MO}} \cdot t_b \]
\[ a \geq \sqrt{2} \cdot 0.9 \cdot \frac{50 \text{ ksi} \cdot 1.25}{62 \text{ ksi} \cdot 1.00} \cdot t_b \]
\[ \therefore a = t_w \geq 1.28 \cdot t_b \]

and when Equation 2.11 is re-calculated for cold-formed RHS made to ASTM A500 Grade C:
\[ a \geq \left( \frac{1}{0.9 \cdot \sqrt{2}} \right) \cdot \frac{F_{yb}}{F_{ub}} \cdot \frac{\gamma_{M2}}{\gamma_{MO}} \cdot t_b \]

\[ a \geq \left( \frac{1}{0.9 \cdot \sqrt{2}} \right) \cdot \frac{50 \text{ ksi}}{62 \text{ ksi}} \cdot 1.25 \cdot 1.00 \cdot t_b \]

\[ \therefore a = t_w \geq 0.79 \cdot t_b \quad 2.13 \]

Therefore the governing minimum effective throat size using the directional method of EN-1993-1-8 (CEN, 2005) for cold-formed RHS made to ASTM A500 Grade C would be 1.28 times the wall thickness of the connected branch member.

### 2.3.1.3.2. Design resistance of fillet welds according to the simplified method

The simplified method of EN 1993-1-8 (CEN, 2005) is an alternative to the directional method for the design of fillet welded connections between RHS. The strength of a fillet weld may be deemed sufficient if the design value of the resultant force per unit length transmitted by the weld \((F_{w,Ed})\) is less than or equal to the design resistance per unit length of the weld \((F_{w,Rd})\), as follows:

\[ F_{w,Ed} \leq F_{w,Rd} \quad 2.14 \]

Unlike the directional method, it is independent of the orientation of the applied force to the plane of the design weld throat. The design resistance per unit length of the fillet weld (Equation 2.15) is based on the effective throat thickness and the design shear strength \((f_{vw,d})\).

\[ F_{w,Rd} = f_{vw,d} \cdot a \quad \text{where,} \quad f_{vw,d} = \frac{F_{ub}/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} \quad 2.15 \]

The values for \(F_{ub}\) and \(\beta_w\) are the same as those specified in Section 2.3.1.3.1.

For the requirement of Equation 2.14, setting the design resistance of a fillet welded joint equal to that of the yield resistance for the connected branch member for cold-formed RHS made to EN 10219 – S355H with matching electrodes, the minimum required effective throat (per unit length of fillet weld) in terms of the connected branch wall thickness (per unit length of the branch perimeter) gives:
\[
\frac{F_{ub} \cdot a}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} \geq \frac{F_{yb} \cdot t_b}{\gamma_{MO}}
\]

\[
a = \sqrt{3} \cdot \beta_w \cdot \frac{F_{yb} \cdot \gamma_{M2}}{F_{ub} \cdot \gamma_{MO}} \cdot t_b
\]

\[
a = \sqrt{3} \cdot 0.90 \cdot \frac{355 \text{ MPa}}{510 \text{ MPa}} \cdot \frac{1.25}{1.00} \cdot t_b
\]

\[\therefore a = t_w = 1.36 \cdot t_b\]

Therefore the minimum effective throat size for the simplified method of EN-1993-1-8 (CEN, 2005) is 1.36 times the wall thickness of the connected branch member.

For a more direct comparison between the national/ international codes, specifications and guidelines, Equation 2.16 will be re-calculated for cold-formed RHS made to ASTM A500 Grade C. Thus, the minimum required effective throat (per unit length of fillet weld) in terms of the connected branch wall thickness (per unit length of the branch perimeter) gives:

\[
a = \sqrt{3} \cdot \beta_w \cdot \frac{F_{yb} \cdot \gamma_{M2}}{F_{ub} \cdot \gamma_{MO}} \cdot t_b
\]

\[
a = \sqrt{3} \cdot 0.90 \cdot \frac{50 \text{ ksi}}{62 \text{ ksi}} \cdot \frac{1.25}{1.00} \cdot t_b
\]

\[\therefore a = t_w = 1.57 \cdot t_b\]

Therefore the governing minimum effective throat size using the simplified method of EN-1993-1-8 (CEN, 2005) for cold-formed RHS made to ASTM A500 Grade C is 1.57 times the wall thickness of the connected branch member.

2.3.1.4. **CIDECT Design Guide No. 3**

The design of fillet welded joints according to the CIDECT Design Guide No. 3 for RHS connections under predominantly static loading (Packer et al., 2009) is based on the directional method of EN-1993-1-8 (CEN, 2005) and prequalified sizes are given that develop the yield resistance of the branch member wall. The minimum throat thickness required to achieve this, assuming RHS made to EN 10219 – S355H with matching
electrodes, is rounded to 1.10 times the wall thickness of the connected branch member.

2.3.1.5. Summary of design method (i)

The effective throats required to develop the yield strength of the connected branch member wall of axially-loaded 90° T-connections between RHS made to ASTM A500 Grade C with matching electrodes for various national and international codes, specifications and guidelines are summarized in Table 2-3. Clearly there is quite a disparity and no definitive agreement exists for how to properly design a fillet weld to develop the yield strength of the connected branch member wall.

Table 2-3 Comparison of fillet weld effective throats required to develop the yield strength of the connected branch member wall, for an axially-loaded 90° T-connection between RHS made to ASTM A500 Grade C with matching electrodes

<table>
<thead>
<tr>
<th>Specification or Code</th>
<th>tw</th>
<th>tb</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANSI/AISC 360-10 Table J2.5</td>
<td>1.43</td>
<td>t_b</td>
</tr>
<tr>
<td>AWS D1.1/D1.1M: 2010 Clause 2.25.1.3 and Fig. 3.2</td>
<td>1.07</td>
<td>t_b</td>
</tr>
<tr>
<td>CAN/CSA S16-01 Clause 13.13.2.2</td>
<td>1.14</td>
<td>t_b</td>
</tr>
<tr>
<td>CSA S16-09 Clause 13.13.2.2</td>
<td>0.95</td>
<td>t_b</td>
</tr>
<tr>
<td>CEN (2005): Directional method</td>
<td>1.28</td>
<td>t_b</td>
</tr>
<tr>
<td>CEN (2005): Simplified method</td>
<td>1.57</td>
<td>t_b</td>
</tr>
</tbody>
</table>

2.3.2. Design Method (ii) - Effective Weld Properties

As mentioned, a common weld design method for connections between RHS is to proportion the weld to achieve the yield strength of the connecting branch member wall at all locations around the perimeter. Modern design methods based on data from full-scale tests of weld-critical connections between RHS performed at the University of Toronto (Frater & Packer, 1992a, 1992b; Packer & Cassidy, 1995) have led to the development and use of effective weld properties. These properties take into account the non-uniform distribution of normal stress around the weld perimeter due to the flexibility of the connecting RHS chord face and exclude portions of the weld that are ineffective in resisting the applied loads.
2.3.2.1. Historical development

In 1981 Subcommission XV-E of the International Institute of Welding (IIW) produced their first design recommendations for predominantly 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 & Henderson, 1997) and the United States (AISC, 2010).

Research at the University of Toronto (Frater & Packer, 1992a, 1992b) on fillet-welded RHS branches in full-scale Warren trusses with gapped K-connections (shown in Figure 2-7) revealed that fillet welds in that context can be proportioned on the basis of the applied loads in the branch members, 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 angle of inclination of the branch relative to the chord ($\theta$) is 50° or less (Equation 2.18), but that the weld along the heel should be considered as completely ineffective when the angle is 60° or more (Equation 2.19). A linear interpolation was recommended when $\theta$ is between 50° and 60°. Based on this research, the formulas 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 $\theta \leq 50^\circ$:

$$l_e = \frac{2H_b}{\sin \theta} + 2B_b$$  \hspace{1cm} \text{(2.18)}

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

$$l_e = \frac{2H_b}{\sin \theta} + B_b$$  \hspace{1cm} \text{(2.19)}

In a further study by Packer and Cassidy (1995), by means of 16 full-scale connection tests (Figure 2-8) which were designed to be weld-critical, new weld effective length formulas for T-, Y- and X- (Cross-) connections were developed. It was found that more of the weld perimeter was effective for lower branch member inclination angles for such connections. Thus, the formulas 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 $\theta \leq 50^\circ$:

$$l_e = \frac{2H_b}{\sin \theta} + B_b$$  \hspace{1cm} \text{(2.20)}

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

$$l_e = \frac{2H_b}{\sin \theta}$$  \hspace{1cm} \text{(2.21)}

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

The latest (3rd edition) of the IIW recommendations (IIW, 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:

- Welds are to be proportioned as “fit for purpose” and to resist forces in the connected members, taking account of connection deformation/rotation capacity and considering weld effective lengths, or
- Welds are to be proportioned to achieve the capacity of the connected member walls.

This document (IIW, 2012) thus specifically acknowledges the effective length concept for designing welds between RHS.
Specific design criteria for RHS connections (statically and cyclically loaded) are given in Clause 2: Design of welded connections, Part D (AWS, 2010) and used with the applicable requirements of Part A. Those provisions may be used in conjunction with governing steel design codes, such as ANSI/AISC 360 (2010), to determine the strength of structural steel members or connections.

In Section K4 (AISC, 2010) a detailed design method considering effective weld properties for predominantly statically loaded RHS-to-RHS connections is given. The available strength of such connections incorporates the non-uniform load transfer around the perimeter of the weld due to differences in the relative flexibilities of the chord loaded normal to its surface and membrane stresses carried by the branch parallel to its surface (as mentioned earlier). The nominal strengths of connections subject to branch axial load or bending are based on the limit state of shear rupture along the plane of the effective weld throat and are calculated as follows:

\[
R_n \text{ or } P_n = F_{nw} \ t_w \ l_e \tag{2.22}
\]

\[
M_{n-ip} = F_{nw} S_{ip} \tag{2.23}
\]

\[
M_{n-op} = F_{nw} S_{op} \tag{2.24}
\]
where the LRFD resistance factor, $\emptyset$, is equal to 0.75 and 0.80 for fillet welds and partial-joint-penetration (PJP) flare-bevel-groove welds, respectively.

The nominal stress of the weld metal, $F_{nw}$, for fillet welds and PJP groove welds, specified in Table J2.5 (AISC, 2010), is taken as 0.60 times the minimum tensile strength of the weld metal, $F_{Exx}$, for fillet welds subject to shear and PJP groove welds subject to tension normal to the weld axis. The use of a directional strength enhancement factor for fillet welds in RHS-to-RHS connections is currently not allowed (AISC, 2010; Packer et al., 2010).

The effective weld lengths associated with Equation 2.22 for gapped K- and N-connections under branch axial load, given in Table K4.1 (AISC, 2010), are summarized as follows:

For $\theta \leq 50^\circ$:
$$l_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + 2(B_b - 1.2t_b)$$  \hspace{1cm} 2.25

For $\theta \geq 60^\circ$:
$$l_e = \frac{2(H_b - 1.2t_b)}{\sin \theta} + (B_b - 1.2t_b)$$  \hspace{1cm} 2.26

When $50^\circ < \theta < 60^\circ$ a linear interpolation shall be used to determine $l_e$. In contrast to Equations 2.18 and 2.19, a reduction to the individual weld element lengths (equal to $1.2t_b$) has been implemented to account for the typical RHS corner radius. The equations have been simplified compared to the more complex ones that would result if the branch effective widths, specified in Section K2.3 (AISC, 2010), were used. Effective weld length provisions for RHS-to-RHS overlapped K- and N-connections are also provided which use the aforementioned branch effective widths in Section K2.3 (AISC, 2010); however there is no experimental data on weld-critical connections of that type to support the postulated equations. Finally, no provisions are given for K-connections under branch bending since the branch ends are assumed to be pin-jointed at the ultimate limit state, connected to a continuous chord, transferring only axial load.

The effective weld properties associated with Equations 2.22, 2.23 and 2.24 for T-, Y- and cross-connections under branch axial load or bending are specified in Table K4.1 (AISC, 2010), and summarized as follows:
For axial load:

\[ l_e = \frac{2H_b}{\sin \theta} + 2b_{eoi} \]  \hspace{1cm} 2.27

For in-plane bending:

\[ S_{ip} = \frac{t_w}{3} \left( \frac{H_b}{\sin \theta} \right)^2 + t_w b_{eoi} \left( \frac{H_b}{\sin \theta} \right) \]  \hspace{1cm} 2.28

For out-of-plane bending:

\[ S_{op} = t_w \left( \frac{H_b}{\sin \theta} \right) B_b + \frac{t_w}{3} \left( \frac{B_b^2}{B_b} \right) - \frac{\left( t_w / 3 \right) (B_b - b_{eoi})^3}{B_b} \]  \hspace{1cm} 2.29

where \( b_{eoi} \) is calculated using Equation 2.1 and for connections with \( \beta > 0.85 \) or \( \theta > 50^\circ \), \( b_{eoi} / 2 \) shall not exceed \( 2t \).

In contrast to Equations 2.20 and 2.21, the weld effective length in Equation 2.27 was – for consistency – made equivalent to the branch wall effective lengths used in Section K2.3 (AISC, 2010) 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 2-9(b). This term was empirically derived on the basis of laboratory tests in the 1970s and 1980s (Davies & 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 2.28 and 2.29), apply in the presence of the bending moments, \( M_{ip} \) and \( M_{op} \) (shown in Figure 2-9). Equation 2.28 is derived from:

\[ I_{ip} = 2 \times \frac{t_w}{12} \left( \frac{H_b}{\sin \theta} \right)^3 + 4t_w \left( \frac{b_{eoi}}{2} \right) \left( \frac{H_b}{2 \sin \theta} \right)^2 \]

\[ = \frac{t_w}{6} \left( \frac{H_b}{\sin \theta} \right)^3 + \frac{t_w b_{eoi}}{2} \left( \frac{H_b}{\sin \theta} \right)^2 \]  \hspace{1cm} 2.30

and substituted into:

\[ S_{ip} = \frac{I_{ip}}{(H_b / 2 \sin \theta)} \]  \hspace{1cm} 2.31
In a similar manner, Equation 2.29 is derived from:

\[ I_{op} = 2 \times \left[ t_w \left( \frac{H_b}{\sin \theta} \right) \left( \frac{B_b}{2} \right)^2 + \frac{t_w}{12} (B_b)^3 - \frac{t_w}{12} (B_b - b_{eol})^3 \right] \]

\[ = \frac{t_w}{2} \left( \frac{H_b}{\sin \theta} \right) (B_b)^2 + \frac{t_w}{6} (B_b)^3 - \frac{t_w}{6} (B_b - b_{eol})^3 \]

and substituted into:

\[ S_{op} = \frac{I_{op}}{\left( B_b / 2 \right)} \]

While being based on informed knowledge of general RHS connection behavior, Equations 2.28 and 2.29 have not been substantiated by tests, and therefore are purely speculative.

2.3.2.3. Evaluation of ANSI/AISC 360-10 with prior weld-critical experiments of connections between RHS

A series of weld-critical tests were performed on 14 RHS-to-rigid plate X-connections and nine “simulated gapped K-connections” with the branches inclined at various angles and loaded in quasi-static, axial tension (Frater, 1986). Horizontal and vertical weld leg
sizes as well as the effective weld throats were measured and recorded at numerous locations around the branch perimeter. The geometric and mechanical properties have been used to predict the nominal and LRFD design strengths of the welded joints according to the ANSI/AISC 360 (2010) at the limit state of shear rupture along the plane of the effective weld throat.

The predicted strength of each welded joint, without a fillet weld directional strength enhancement factor of \((1.00 + 0.50 \sin^{1.5}\theta)\), was determined by the summation of the individual weld element strengths along the four walls around the branch perimeter and is given as a predicted nominal strength, \(R_n\). The entire perimeter of the weld was considered effective for the RHS-to-rigid plate X-connections whereas a reduced weld length, specified by Equation 2.26, was used for the “simulated gapped K-connections”.

In order to assess whether adequate, or excessive, safety margins are inherent in the correlations shown in Figure 2-10(a) and Figure 2-11(a), one can check to ensure that a minimum safety index of \(\beta^+ = 4.0\) (as currently adopted by ANSI/AISC 360 (2010) per Chapter B of the Specification Commentary) is achieved, using a simplified reliability analysis in which the resistance factor (\(\varphi\)) is given by Equation 2.34 (Fisher et al., 1978; Ravindra & Galambos, 1978):

\[
\varphi = m_R \cdot \exp(-\alpha \cdot \beta^+ \cdot COV)
\]

where \(m_R\) = mean of the ratio: (actual element strength)/(nominal element strength); \(COV\) = associated coefficient of variation; and \(\alpha\) = coefficient of separation taken to be 0.55 (Ravindra & Galambos, 1978). Equation 2.34 neglects variations in material properties, geometric parameters and fabrication defects, 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. A resistance factor of 0.75, specified in Section K4 (AISC, 2010), is applied to Equations 2.22, 2.23 and 2.24 to calculate the LRFD design strength of fillet welded joints.

Application of Equation 2.34 produced \(\varphi = 0.73\) for welded joints in RHS-to-rigid plate X-connections which is slightly less than \(\varphi = 0.75\) and indicates that the desired safety
margin may not be achieved. The “simulated gapped K-connections” produced $\phi = 0.65$ which is much less than 0.75. Thus, the effective weld length concepts advocated in Section K4 (AISC, 2010) cannot, on the basis of the available experimental evidence, be deemed adequately conservative for “simulated gapped K-connections”.

Two large-scale, 40.0-ft. 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 joints (rather than connection failures). In addition, a series of weld-critical tests were performed by Packer and Cassidy (1995) on four T-connections and 12 X-connections, with the branches loaded in quasi-static, axial tension.

The geometric and mechanical properties have been used to predict the nominal and LRFD design strengths of the welded joints according to ANSI/AISC 360 (2010) at the limit state of shear rupture along the plane of the effective weld throat. They have been compared to the actual strengths recorded during full-scale testing and their correlations are plotted in Figure 2-12 and Figure 2-13 for gapped K-connections and T- and X-(Cross-) connections, respectively.

Application of Equation 2.34 produced $\phi = 0.96$ for welded connections in real truss gapped K-connections and $\phi = 0.86$ for T- and X- (Cross-) connections. As both of these exceed $\phi = 0.75$ the effective weld length concepts advocated in Section K4 (AISC, 2010) can, on the basis of the available experimental evidence, be deemed adequately conservative.
Chapter 2: Relevant Research and Current Design Criteria

Effective Weld Properties for RHS-to-RHS Moment T-Connections

Figure 2-10 Correlation with test results for RHS-to-rigid plate X-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 ($0.75 \cdot R_n$)

Figure 2-11 Correlation with test results for RHS-to-RHS “simulated 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 ($0.75 \cdot R_n$)
Figure 2-12 Correlation with test results for RHS-to-RHS 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 ($0.75 \cdot R_n$)

Figure 2-13 Correlation with test results for RHS-to-RHS T- and X-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 ($0.75 \cdot R_n$)
2.3.2.4. Introduction of the \((1.00 + 0.50 \sin^{1.5}\theta)\) factor

The application of a directional strength enhancement factor (of \(1.00 + 0.50 \sin^{1.5}\theta\)) to the nominal strength for fillet welds loaded at an angle of \(\theta\) degrees to the weld longitudinal axis has again been recently reviewed for lapped plate shear connections with multi-orientation welds (Callele et al., 2009). In the US, ANSI/AISC 360 (2010) does not at present permit the use of the fillet weld directional strength enhancement factor for connections between hollow sections, whereas in Canada, the CSA and CISC do not explicitly disallow it, so some designers use it. Adopting this enhancement factor leads to a much greater calculated resistance for a fillet weld group in a RHS connection, resulting in much smaller required weld sizes.

The correlation plots in Figure 2-10, Figure 2-11, Figure 2-12 and Figure 2-13 have been recomputed for the limit state of shear rupture along the plane of the effective weld throat with the inclusion of the \((1.00 + 0.5 \sin^{1.5}\theta)\) factor shown in Figure 2-14, Figure 2-15, Figure 2-16 and Figure 2-17 (respectively). 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 RHS-to-rigid plate X-connections:
  
  \[
  m_R = 0.831 \quad \text{COV} = 0.255 \quad \emptyset = 0.47
  \]

- For "simulated gapped K-connections":
  
  \[
  m_R = 0.768 \quad \text{COV} = 0.261 \quad \emptyset = 0.43
  \]

- For gapped K-connections in full-scale truss tests:
  
  \[
  m_R = 0.999 \quad \text{COV} = 0.180 \quad \emptyset = 0.67
  \]

- For T- and X- (Cross-) connections:
  
  \[
  m_R = 0.819 \quad \text{COV} = 0.164 \quad \emptyset = 0.58
  \]

Every \(\emptyset\)-factor calculated using Equation 2.34 for the various types of connections listed above is below 0.75. Thus, use of the \((1.00 + 0.50 \sin^{1.5}\theta)\) factor is inadvisable for connections between RHS because it may result in unsafe fillet weld designs, especially
in combination with the effective weld properties specified in Section K4 (AISC, 2010).
One reason for the inapplicability of the fillet weld strength enhancement factor, even for fillet welds to a rigid base plate, may be the fact that all fillet welds to RHS branches are inherently eccentrically-loaded, due to welding on only one side of the load-bearing element (wall).
Effective Weld Properties for RHS-to-RHS Moment T-Connections

Figure 2-14 Correlation with test results for RHS-to-rigid plate X-connections including 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 ($\phi R_n$)

Figure 2-15 Correlation with test results for RHS-to-RHS "simulated gapped K-connections" including 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 ($\phi R_n$)
Figure 2-16 Correlation with test results for RHS-to-RHS gapped K-connections including the \((1.00 + 0.50 \sin^{1.5} \theta)\) term

Figure 2-17 Correlation with test results for RHS-to-RHS T- and X-connections including the \((1.00 + 0.50 \sin^{1.5} \theta)\) term
Chapter 3: Experimental Program

3.1. Scope

An experimental program was developed at the University of Toronto to test various unreinforced RHS-to-RHS 90° T-connections subject to branch in-plane bending moment. The objective of this study was to investigate the effectiveness of the weld in resisting the forces at the ultimate limit state of weld rupture and verify or adjust the current effective weld properties postulated in Section K4 (AISC, 2010) for such connections. Twelve test specimens were designed to be weld-critical, where weld rupture precedes connection failure, under the application of branch in-plane bending moment. Key parameters such as branch-to-chord width ratios ($\beta$-ratios) ranging from 0.25 to 1.00 and chord wall slenderness values of 17, 23 and 34 were investigated. In order to determine the effectiveness of the weld in resisting the applied forces, the non-uniform distribution of normal strain in the branch near the connection was measured using strain gages oriented along the longitudinal axis of the branch at numerous locations around the perimeter. All connections were fabricated with a continuous effective weld throat by industrial robots modified to perform the gas-metal arc welding (GMAW) process at Lincoln Electric’s Automation Division in Cleveland, Ohio. To induce bending moment at the connection, a lateral point load was applied to the branch member in a quasi-static manner by a 77-kip capacity MTS portable actuator mounted to a rigid steel support frame. The experimental program includes tensile coupon tests of the RHS material and as-laid weld metal, as well as 12 full-scale experiments on unreinforced, RHS-to-RHS moment T-connections.

3.2. Design Procedure for Weld-Critical Connections

Twelve test specimens were designed to be weld-critical under the application of branch in-plane bending moments. Cold-formed RHS made to ASTM A500 Grade C were used for all branch and chord members in the experimental program. Their geometric configurations were selected based on available materials and key parameters that
influence connection strength and behavior: $\beta$-ratio and chord wall slenderness value. The outside dimensions of the chord remained constant (8 x 8-in.) for all test specimens to facilitate ease of setup and takedown in the testing rig. Nominal wall thicknesses of $\frac{1}{4}$, $\frac{3}{8}$ and $\frac{1}{2}$ inches were selected and correspond to chord wall slenderness values of 34, 23 and 17 (respectively) when considering the design wall thickness (AISC, 2010). The outside dimensions of the branch were 2 x 2, 4 x 4, 6 x 6 and 8 x 8 inches giving $\beta$-ratios of 0.25, 0.50, 0.75 and 1.00 (respectively). The combination of $\beta$-ratios and chord wall slenderness values gave a vast range of potential failure modes (described in Chapter 2). Experimental designation, chord and branch dimensions, as well as the associated key parameters of the individual test specimens are represented in Table 3-1.

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Chord Designation</th>
<th>Branch Designation</th>
<th>$\beta$ - Ratio</th>
<th>Chord Wall Slenderness Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>HSS 8 x 8 x $\frac{1}{4}$&quot;</td>
<td>HSS 2 x 2 x $\frac{1}{4}$&quot;</td>
<td>0.25</td>
<td>34</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>HSS 8 x 8 x $\frac{3}{8}$&quot;</td>
<td>HSS 2 x 2 x $\frac{1}{4}$&quot;</td>
<td>0.25</td>
<td>23</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>HSS 8 x 8 x $\frac{1}{2}$&quot;</td>
<td>HSS 2 x 2 x $\frac{1}{4}$&quot;</td>
<td>0.25</td>
<td>17</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>HSS 8 x 8 x $\frac{1}{4}$&quot;</td>
<td>HSS 4 x 4 x $\frac{1}{4}$&quot;</td>
<td>0.50</td>
<td>34</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>HSS 8 x 8 x $\frac{3}{8}$&quot;</td>
<td>HSS 4 x 4 x $\frac{1}{4}$&quot;</td>
<td>0.50</td>
<td>23</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>HSS 8 x 8 x $\frac{1}{2}$&quot;</td>
<td>HSS 4 x 4 x $\frac{1}{2}$&quot;</td>
<td>0.50</td>
<td>17</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>HSS 8 x 8 x $\frac{1}{4}$&quot;</td>
<td>HSS 6 x 6 x $\frac{1}{4}$&quot;</td>
<td>0.75</td>
<td>34</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>HSS 8 x 8 x $\frac{3}{8}$&quot;</td>
<td>HSS 6 x 6 x $\frac{3}{8}$&quot;</td>
<td>0.75</td>
<td>23</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>HSS 8 x 8 x $\frac{1}{2}$&quot;</td>
<td>HSS 6 x 6 x $\frac{1}{2}$&quot;</td>
<td>0.75</td>
<td>17</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>HSS 8 x 8 x $\frac{1}{4}$&quot;</td>
<td>HSS 8 x 8 x $\frac{1}{4}$&quot;</td>
<td>1.00</td>
<td>34</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>HSS 8 x 8 x $\frac{3}{8}$&quot;</td>
<td>HSS 8 x 8 x $\frac{3}{8}$&quot;</td>
<td>1.00</td>
<td>23</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>HSS 8 x 8 x $\frac{1}{2}$&quot;</td>
<td>HSS 8 x 8 x $\frac{1}{2}$&quot;</td>
<td>1.00</td>
<td>17</td>
</tr>
</tbody>
</table>

Test specimens with $0.25 \leq \beta \leq 0.85$ are classified as stepped connections while those with $0.85 < \beta \leq 1.00$ are classified as matched connections and their general configurations are depicted in Figure 3-1 (a) and (b) respectively. Stepped connections have a continuous fillet weld around the branch footprint whereas matched connections have a fillet weld along the branch transverse walls and a PJP groove weld along the
branch longitudinal walls. A transitional zone between the two types of welds exists at the branch corners.

![Side elevation of a stepped connection](image1) ![Side elevation of a matched connection](image2)

(a) Side elevation of a stepped connection (0.25 ≤ β ≤ 0.85)  
(b) Side elevation of a matched connection (0.85 < β ≤ 1.00)

**Figure 3-1 RHS-to-RHS T-connection classification**

The next stage of design was to calculate the flexural strength of each connection using the equations postulated in Table 2-1 and the corresponding branch flexural strength (AISC, 2010). As mentioned, the more recently introduced LRFD method was used for all calculations instead of the ASD method to conform to modern design practices. A summary of the LRFD connection and branch flexural strengths, as well as governing limit states for each test specimen, is shown in Table 3-2. Sample calculations accompanied by detailed summary tables using the nominal RHS material properties are provided in Appendix A.1 and A.2, respectively.

A lower bound approach was used to design the welds to ensure that weld rupture preceded connection failure, whereby the predicted nominal flexural strength of the weld ($M_{n_{-ip}}$) was less than or equal to the predicted LRFD flexural strength of the test specimen ($\phi M_n$), calculated in Table 3-2. The corresponding weld sizes were selected based on standard sizes specified in the ANSI/AWS D1.1 *Structural Welding Code* (2010) satisfying the minimum requirements in Tables J2.3 and J2.4 of ANSI/AISC 360 (2010) for PJP groove welds and fillet welds, respectively. Matching electrodes with a nominal tensile strength of 70-ksi were used for the calculations.
### Table 3-2 LRFD flexural strength of the connections

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>LRFD Connection Flexural Strength $\phi M_n$ (kip-ft)</th>
<th>LRFD Branch Flexural Strength $\phi M_{n-b}$ (kip-ft)</th>
<th>Maximum LRFD Flexural Strength $\phi M_n$ (kip-ft)</th>
<th>Governing Limit State</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>2.09</td>
<td>3.62</td>
<td>2.09</td>
<td>Chord wall plastification</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>4.71</td>
<td>3.62</td>
<td>3.62</td>
<td>Branch flexural failure</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>8.37</td>
<td>3.62</td>
<td>3.62</td>
<td>Branch flexural failure</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>4.35</td>
<td>17.59</td>
<td>4.35</td>
<td>Chord wall plastification</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>9.79</td>
<td>17.59</td>
<td>9.79</td>
<td>Chord wall plastification</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>17.40</td>
<td>28.9</td>
<td>17.40</td>
<td>Chord wall plastification</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>10.36</td>
<td>42.0</td>
<td>10.36</td>
<td>Chord wall plastification</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>23.3</td>
<td>59.2</td>
<td>23.3</td>
<td>Chord wall plastification</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>41.4</td>
<td>74.2</td>
<td>41.4</td>
<td>Chord wall plastification</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>39.4</td>
<td>71.5</td>
<td>39.4</td>
<td>Local yielding of branch</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>66.5</td>
<td>110.2</td>
<td>66.5</td>
<td>Local yielding of branch</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>99.1</td>
<td>140.6</td>
<td>99.1</td>
<td>Local yielding of branch</td>
</tr>
</tbody>
</table>

### Table 3-3 Nominal flexural strength of the welded joints

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Specified Weld Leg Size $L_v, L_H$ (in.)</th>
<th>Nominal Weld Flexural Strength $M_{n-ip} = F_{nw} \cdot S_{ip}$ (kip-ft)</th>
<th>Nominal Weld : Maximum Flexural Strength Ratio $M_{n-ip}/\phi M_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>$\frac{3}{16}$</td>
<td>1.160</td>
<td>0.55</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>$\frac{3}{16}$</td>
<td>1.830</td>
<td>0.51</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>$\frac{3}{16}$</td>
<td>2.34</td>
<td>0.65</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>$\frac{3}{16}$</td>
<td>4.20</td>
<td>0.97</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>$\frac{3}{16}$</td>
<td>5.06</td>
<td>0.52</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>$\frac{3}{8}$</td>
<td>11.85</td>
<td>0.68</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>$\frac{3}{16}$</td>
<td>8.16</td>
<td>0.79</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>$\frac{1}{4}$</td>
<td>12.60</td>
<td>0.54</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>$\frac{3}{8}$</td>
<td>21.5</td>
<td>0.52</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>($\frac{3}{16}$)*</td>
<td>18.88</td>
<td>0.48</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>($\frac{1}{4}$)*</td>
<td>28.4</td>
<td>0.43</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>($\frac{3}{8}$)*</td>
<td>47.5</td>
<td>0.48</td>
</tr>
</tbody>
</table>

* Value represents the specified continuous effective weld throat, $t_w$
The calculated nominal flexural strengths of the welded joints for each test specimen using the equation for the effective elastic section modulus for in-plane bending (Equation 2.28) specified in Section K4 (AISC, 2010), at the limit state of shear rupture along the plane of the effective weld throat, are summarized in Table 3-3 and given in further detail in Appendix A.

Early in the experimental program it was discovered that the above-mentioned design method was not conservative enough to ensure weld rupture for every test specimen. After three full-scale tests, two had failed by punching shear of the chord face and hence a decision was made to reduce the weld sizes on the remaining (nine) test specimens using a hand-held grinder with a metal grinding wheel. Those with \( \beta = 0.50 \) (T-0.50-34, T-0.50-23, T-0.50-17) were tested with the weld sizes in Table 3-3 but the other nine tests were tested with the reduced weld sizes summarized in Table 3-4. As an extra precaution, the 0.60 factor applied to \( F_{EXX} \) used to calculate the nominal stress of the weld metal \( (F_{nw}) \) was excluded from the weld flexural strength calculations performed prior to testing (to ensure weld-critical behavior). Detailed resistance tables, using the specified nominal properties and the final weld sizes, that summarize the predicted nominal flexural strength of the welded joints, are provided in Appendix A.2.

Table 3-4 Predicted flexural strength of welded joints after weld size reduction and excluding the factor of 0.60 applied to \( F_{EXX} \)

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Specified Weld Leg Size ( L_H, L_V ) (in.)</th>
<th>Nominal Flexural Strength of Weld ( M_{n-ip} = F_{EXX} \cdot S_{ip} ) (kip-ft)</th>
<th>Nominal Weld : Maximum Flexural Strength Ratio ( M_{n-ip}/\phi M_n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>1/8</td>
<td>1.290</td>
<td>0.62</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>1/8</td>
<td>2.04</td>
<td>0.56</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>1/8</td>
<td>2.61</td>
<td>0.72</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>1/8</td>
<td>9.06</td>
<td>0.87</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>3/16</td>
<td>15.75</td>
<td>0.68</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>1/4</td>
<td>23.9</td>
<td>0.58</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>(1/8)*</td>
<td>21.0</td>
<td>0.53</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>(3/16)*</td>
<td>35.5</td>
<td>0.53</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>(1/4)*</td>
<td>52.8</td>
<td>0.53</td>
</tr>
</tbody>
</table>

* Value represents the specified continuous effective weld throat, \( t_w \)**
3.2.1. Welding Process Investigation

As part of an investigation into the viability of automated welding for structural steel fabrication processes (and hence this experimental program), numerous trial specimens were welded at Lincoln Electric’s Automation Division in Cleveland, Ohio by a Fanuc Robot ARC Mate 100iC, adapted to perform GMAW with pulsed spray metal transfer, shown in Figure 3-2 (a). The trial specimens were representative of the types of welded joints specified in the experimental program. Specimens were mounted on chucks that rotated in coordination with the robotic arm throughout the welding process (referred to as coordinated motion). An experienced welder/robot technologist used a controller to manually input the weld path and various other process parameters such as electrode amperage and voltage, welding position, travel speed, start/stop points, electrical stickout and wire feed speed.

![Fanuc ARC Mate 100iC adapted to perform GMAW-P](image1)

![Welded trial specimen: fillet weld along branch transverse face, PJP flare-bevel-groove weld along branch longitudinal face](image2)

**Figure 3-2 Automated welding of trial specimens at Lincoln Electric’s Automation Division**

A 0.045” diameter AWS ER70S-6 (SuperArc L-59) solid wire electrode with a shielding gas mixture of 90% argon and 10% carbon dioxide supplied at a flow rate of 40 cubic feet per hour (CFH) was used to weld the trial specimens. Those with $0.25 \leq \beta \leq 0.85$ (stepped connections) were welded in the horizontal position with continuous fillet welds around the branch footprint. Specimens with $0.85 < \beta \leq 1.00$ (matched connections) were welded in the flat and vertical positions with PJP flare-bevel-groove welds along the branch longitudinal walls and fillet welds along the branch transverse walls, as
shown in Figure 3-2 (b). Root pass welds along the corner radius of the chord were required to prevent the weld metal from “blowing through” the gap along the PJP flare-bevel-groove weld.

While the welds appeared to be of good quality from a basic visual inspection, macroetch examinations were still performed whereby disks were cut normal to the longitudinal axis of the weld to observe the macrostructure of the weld cross-sections. The disks were prepared in accordance with ASTM E340-06 (ASTM, 2006) using a 10% nital etchant solution applied to the surfaces at the joints to observe the weld profile, weld/base metal fusion and root penetration. Volumetric discontinuities such as porosity and undercutting, as well as planar discontinuities such as cracks and incomplete fusion, were investigated.

The fillet weld in Figure 3-3 (a) has a desirable profile according to ANSI/AWS D1.1 (2010) with a slightly convex face and vertical/ horizontal legs of approximately the same size. Fusion with the base metal is thorough with deep, broad, finger-like penetration at the root, typical of high argon concentration shielding gases mixed with carbon dioxide (Lincoln Electric, 2004). No visible discontinuities were observed in the fillet weld. The PJP flare-bevel-groove weld in Figure 3-3 (b) also shows thorough fusion with the base metal and penetration to the root pass weld. No visible discontinuities were observed in the PJP flare-bevel-groove weld.
Based on the macroetch examination it was concluded that automated welding was a viable option for the experimental program. Several advantages became apparent including improved weld quality over semiautomatic processes, excellent fusion with the base metal, adequate root penetration, continuous electrodes, capability of welding in all positions and consistent travel speeds (and hence, consistent weld sizes).

3.3. Test Specimen Fabrication Process

The chord and branch members were all cold-formed RHS made to ASTM A500 Grade C. They were cut to the lengths specified in the fabrication drawings (see Appendix B) using a vertical band saw. Branch ends intersecting the chord face were machined normal to their longitudinal axis to reduce gaps between the connecting surfaces, as well as to avoid initial out-of-straightness upon cutting. The same was done for the chord end closest to the branch. Connecting surfaces were then polished using a hand-held grinder with a wire brush wheel to remove mill scale and other impurities prior to welding.

The test specimens were configured and then tack-welded in a rigid fixture consisting of level supports and an angle-iron bolted to a level welding table secured to the laboratory floor. Clamping helped to resist the weld contraction effects on the branch caused by the heat input during welding. Machined risers were used to support different branch sizes and to ensure the branch longitudinal centerline intersected the chord longitudinal
centerline at 90°. Due to inherent variability in the geometric properties of RHS, shimming was sometimes necessary. Once the members were clamped into the desired configuration they were tack-welded at the branch mid-wall locations on all four sides of the joint. Figure 3-4 shows a photograph of the rigid fixture (a) and tack-welded test specimens (b).

### 3.3.1. Automated Welding

The welded joints were executed by an industrial robot modified to perform GMAW at the Automation Division at Lincoln Electric’s headquarters in Cleveland, Ohio. The welding equipment used throughout fabrication includes: the Fanuc ARC Mate 120iC 10L robotic arm, Fanuc system R-30iA power supply, Lincoln Electric PowerWave 455M/STT and 655 robotic welders, and an automatic wire feeder (shown in Figure 3-5).

![Fanuc ARC Mate 120iC 10L industrial robot modified to perform GMAW-P](image1)

![PowerWave 455M/STT and 655 Robotic Welders](image2)

**Figure 3-5 Welding equipment used throughout fabrication**

A 0.035" diameter AWS ER70S-6 (SuperGlide S6) solid wire electrode with a nominal specified tensile strength of 70 ksi and a shielding gas mixture of 90% argon and 10% carbon dioxide supplied at a rate of 40 CFH was used to weld the test specimens. Welding process parameters recommended by the Lincoln Electric 2010 Welding Consumables Product Catalogue were used as a starting point, and adjusted throughout the fabrication process as necessary. To satisfy the qualification requirements of the ANSI/AWS D1.1 (2010) for prequalified welded joints, numerous
trial specimens were created and macroetched before welding the actual test specimens. The macroetch examinations were used to calibrate the welding process parameters to achieve the desired weld size, profile, fusion with the base metal and root penetration for each joint. A summary of the final welding process parameters is provided in Table 3-5. The corresponding Welding Procedure Specifications (WPS) are located in Appendix C.

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Specified Weld Leg Size $L_H, L_V$ (in.)</th>
<th>Voltage (V)</th>
<th>Wire Feed Speed (ipm.)</th>
<th>Average Travel Speed (ipm.)</th>
<th>Number of passes</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>$3/16$</td>
<td>22</td>
<td>395</td>
<td>18</td>
<td>1</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>$3/16$</td>
<td>22</td>
<td>395</td>
<td>18</td>
<td>1</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>$3/16$</td>
<td>22</td>
<td>395</td>
<td>18</td>
<td>1</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>$3/16$</td>
<td>22</td>
<td>395</td>
<td>18</td>
<td>1</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>$3/16$</td>
<td>22</td>
<td>395</td>
<td>18</td>
<td>1</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>$3/8$</td>
<td>24</td>
<td>340</td>
<td>11</td>
<td>3</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>$1/16$</td>
<td>22</td>
<td>395</td>
<td>18</td>
<td>1</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>$1/4$</td>
<td>25</td>
<td>340</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>$3/8$</td>
<td>25</td>
<td>340</td>
<td>11</td>
<td>3</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>$(3/16)^*$</td>
<td>22</td>
<td>395</td>
<td>14</td>
<td>1</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>$(1/4)^*$</td>
<td>28</td>
<td>500</td>
<td>9</td>
<td>1</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>$(3/8)^*$</td>
<td>28</td>
<td>500</td>
<td>9</td>
<td>3</td>
</tr>
</tbody>
</table>

* Value represents the specified continuous effective weld throat, $t_w$

Stepped connections were clamped to a level table and welded in the horizontal position as shown in Figure 3-6 (a). Matched connections were mounted to rotating chucks and welded in the flat position using coordinated motion, shown in Figure 3-6 (b), with fillet welds along the transverse branch walls and PJP flare-bevel-groove welds along the longitudinal branch walls. Root pass welds along the corner radii of the chord adjacent to the longitudinal branch walls were required for the matched connections.

The weld path for each connection started at the longitudinal mid-wall location of the branch, where the theoretical stress under in-plane bending moment is equal to zero
(the neutral axis), and stopped at the same location on the opposite side Figure 3-7. Since the strain and stress distribution along the transverse wall is the primary interest, it was decided to avoid start/stop points and possible discontinuities in those locations. No significant “pulling” effects were observed during fabrication; the branches remained perpendicular to the chords. For multi-pass welds, interpass cleaning using a wire brush was performed to remove additional material deposited from the welding process (i.e. slag residue).

![Figure 3-6 Automated welding at Lincoln Electric](image)

**Figure 3-6 Automated welding at Lincoln Electric**

![Figure 3-7 Weld path, sequence, direction of travel and start/stop location details](image)

**Figure 3-7 Weld path, sequence, direction of travel and start/stop location details**
Once completed, the welded joints were inspected in accordance with the visual inspection acceptance criteria in Clause 6 of ANSI/AWS D1.1 (2010). Discontinuities such as crack prohibition, undercut, porosity, weld profile and weld size were investigated. No discontinuities exceeding the allowable limits of the visual inspection acceptance criteria were observed. Figure 3-8 shows some general photographs of the completed welded joints.

![3-pass fillet welded test specimen](image1)

![Various completed test specimens](image2)

**Figure 3-8 Welded RHS-to-RHS T-connections**

The final phase of fabrication was to weld the 1-in. thick end plates to the chords. The thickness of the plates and bolt hole details were designed to resist prying forces under the maximum capacity of the actuator (77-kip). A continuous fillet weld around the perimeter of the chord with a leg size of 1/2-in. was specified.

### 3.4. Geometric Measurement Procedures

In order to perform an accurate analysis of the results from the full-scale tests, the actual geometric properties of the RHS material and as-laid welds must be known. A number of techniques described herein were used to measure the geometric properties to a high degree of accuracy.

#### 3.4.1. RHS cross-sectional dimensions

All RHS cross-sectional dimensions were measured in accordance with the recommendations for checking dimensional tolerances for cold-formed RHS made to
the ASTM A500 (2010) specification proposed by the Steel Tube Institute (STI, 1993). Cross-sections of each RHS used in the experimental program were saw-cut at least 12-in. away from the flame-cut ends of the parent tube and then machined normal to the longitudinal axis. The cross-sections were scanned and then traced in Adobe Illustrator whereby the cross-sectional area, outside dimensions and inside/outside corner radii were determined using built-in measuring functions. The wall thickness was measured using a 1.0-in. Mitutoyo Digimatic Micrometer (accurate to ±0.00005”), shown in Figure 3-9 (a).

The geometric properties recorded in Appendix E.1 and tabulated in Table 4-1 were checked with the permissible dimensional tolerances (STI, 1993) and then used to calculate the actual predicted connection, branch and weld flexural strengths.

3.4.2. Weld cross-sectional dimensions

The weld cross-sectional dimensions were measured externally at numerous locations around the weld perimeter prior to testing, which are represented by an ‘X’ in Figure 3-10. The labelling convention shown in Figure 3-10 (b) corresponds to the recorded measurements located in Appendix F.1. A fillet weld gage, Figure 3-9 (b), was used to measure the horizontal and vertical leg sizes as well as the corresponding theoretical effective throat of fillet welds prior to testing. For PJP flare-bevel-groove welds performed by GMAW, the effective throat was estimated in accordance with Table J2.2 (AISC, 2010), equal to $\frac{5}{8}$ times the average outside radius of the RHS corners (assumed to be equal to $2t$) less the greatest perpendicular dimension measured from a line flush from the base metal surface to the weld surface. These external measurements were only used to predict weld-critical behavior of the test specimens prior to testing. They were not used to calculate the predicted LRFD and nominal flexural strengths of the welded joints for the results and analyses.
After testing, the connections were cut normal to the longitudinal axis of the weld at numerous locations around the perimeter shown in Figure 3-11 (a) for test specimens with $0.25 < \beta \leq 1.00$ and Figure 3-11 (b) for those with $\beta = 0.25$. The results from the macroetch examinations are located in Appendix F.2. The horizontal and vertical leg sizes and actual weld throats were measured using the built-in measuring functions of Adobe Illustrator. Average values of the effective weld throat for the individual weld elements around the branch perimeter for each test specimen were used to recalculate the predicted LRFD and nominal flexural strengths of the welded joints using the
equation for the effective elastic section modulus for in-plane bending (AISC, 2010). These values are presented in the results and used for the analyses.

![Figure 3-11 Approximate cross-sectioned locations for macroetch examinations after testing](image)

3.5. Material Property Tests

To determine the material properties of the RHS material and as-laid weld metal, three tensile coupons were created from the same RHS stock for each size used and three from the AWS ER70S-6 (SuperGlide S6) solid wire electrode. The materials were tested to determine yield stress ($F_y$), yield strain ($\varepsilon_y$), ultimate tensile strength ($F_u$, $F_{ub}$, $F_{EXX}$), strain at rupture ($\varepsilon_u$) and Young’s Modulus ($E$).

3.5.1. Tension Tests: Base Metal

Three rectangular tensile coupons for each size of RHS used in the experimental program were created (27 in total). The coupons were saw-cut from the flat surfaces at least twelve inches from the flame-cut ends of the parent tube along the longitudinal axis, as shown in Figure 3-12: one from opposite the weld seam and the other two adjacent to it.

They were fabricated to the dimensions specified in the fabrication drawings (Figure B-13 in Appendix B) and comply with the ASTM standards (ASTM, 2008) for tension testing of metallic materials. Those dimensions were measured using the Mitutoyo
digimatic micrometer and calipers prior to testing and were recorded in the tensile coupon test data sheets found in Appendix E.2.

![Figure 3-12 RHS tensile coupon locations](image)

**3.5.2. Tension Tests: Weld Metal**

Three all-weld-metal tensile coupons were created in accordance with Clause 4 of the ANSI/AWS D1.1 (2010). Steel test plates with a nominal yield strength of 45-ksi were machined to the dimensions shown in Figure 3-13 and then tack-welded to a 10-in. length of HSS 6 x 6 x 1/4”.

The welded test plates were created using the same electrode spool, equipment and fabrication processes as those used for the welded joints tested in the experimental program. The following welding process parameters, which are the average values from Table 3-5, were used:

- Arc voltage = 24V
- Wire feed speed = 400 ipm.
- Travel speed = 15 ipm.
- Electrode shielding = 90% argon, 10% carbon dioxide
- Gas flow rate = 40 CFH
The tensile coupons were cut from the welded test plates at the University of Toronto Structural Testing Facility and fabricated to the dimensions shown in the fabrication drawings (Appendix B). Those dimensions were measured using the Mitutoyo digimatic micrometer and calipers then recorded in the tensile coupon test data sheets found in Appendix E.2.

![Figure 3-13 All-weld-metal tensile coupon welded test plate details](image)

3.6. Test Setup and Instrumentation

Since no experiments of this same type had been performed at the University of Toronto in the past, the entire test setup assembly had to be designed. The first stage was to outline the desired behavior characteristics of the test setup as a whole, which are: rigid support frame for the actuator (near-zero deflection under the actuator's capacity), pure in-plane bending moment acting on the connection with simple supports...
(pin-and-roller) for the chord ends. Next, the instrumentation was selected to measure the non-uniform distribution of normal strain around the branch perimeter adjacent to the welded joint, distribution of normal strain around the branch perimeter beyond the disturbed regions (constant stress regions), branch deflection profile, chord wall deformation profile and displacements of various components on the test setup. Using the available materials and resources, the test setup was designed and then assembled in the University of Toronto Structural Testing Facility. An overall view of the general test setup assembly for the full-scale experiments is shown in Figure 3-14, with the structural components, equipment and instrumentation labelled.

![Figure 3-14 East elevation of the general test setup assembly for full-scale experiments](image)

**3.6.1. Rigid Support Frame and MTS Actuator**

Two W12x87 vertical columns spaced at 48-in. centers were bolted to the laboratory floor, each with two 2-in. diameter post-tensioned high-strength steel anchor rods. The post-tensioned anchor rods prevented the rigid steel support frame from translating horizontally under the capacity of the actuator due to the friction between the column base plates and laboratory floor. Horizontal bracing between the columns consisted of
four 1-in. diameter high-strength threaded steel rods installed concentrically with the actuator. The diagonal tension bracing consisted of two 12 x 1 x 60" long steel plates welded to the column flanges and the compression brace consisted of a HSS 4 x 4 x $\frac{1}{2}$" bolted to the column flanges.

A 77-kip, 10-in. stroke capacity portable MTS actuator was calibrated prior to the experimental program by a MTS technician. During testing, the actuator was displacement-controlled by a specialised MTS system operated by trained laboratory personnel. All of the test specimens were loaded at an initial rate of 0.0002 in/s until nonlinear behavior was observed. The MTS system records the axial force and displacement of the actuator via a built-in load cell and linearly varying differential transformer (LVDT), respectively. Four ASTM A325 1-in. bolts were used to mount the actuator horizontally to the rigid steel support frame with a small crane mounted to the foremost vertical column to support the self-weight of the actuator. No lateral support was provided because the actuator was always in quasi-static, monotonic tension (pulling the branch towards the frame) for every test and hence buckling out-of-plane was not a concern. Since the ends of the actuator have internal spherical bearings, it self-aligns under load to the connected ends. This was a helpful feature in maintaining nearly pure in-plane loading for every test.

3.6.2. Supports and Point Load Device

The test specimen chords were simply-supported by a pin on one end and a roller on the other. Both supports were welded to rigid steel base plates that were secured to the laboratory floor with two 2-in. diameter post-tensioned high-strength steel anchor rods.

The pin support in Figure 3-15 (a) consisted of a clevis welded to two rigid plates: the rigid base plate and the vertical chord end plate connector. The latter is connected to the test specimen using four ASTM A325 1-in. bolts, hand-tightened with a wrench. A cylindrical bearing joint connected the components of the clevis. Hence, the specimen was restricted from translation in all directions but free to rotate about the z-axis (out-of-plane axis).
The roller support was essentially a steel cylinder sandwiched between two steel plates with guiderails for the chord. During testing, the bolts and latch shown in Figure 3-15 (b) were removed to allow the chord to translate in the horizontal direction. No vertical restraints were required since the roller was subjected to compression only.

![Support conditions for the test specimens](image)

(a) Pin support  
(b) Roller support

**Figure 3-15 Support conditions for the test specimens**

The final structural component of the test setup was the point load device (Figure 3-16), designed to transfer the load from the actuator to a single point along the height of the branch, distributed across its width. The round bar on the point load device plate was cupped inside a socket that was welded to plates straddling the branch member at the desired moment arm (vertical distance from the connection to the application of the
point load). This allowed the branch to deflect in the horizontal and vertical axes while keeping the actuator horizontal, and hence the line of action of the applied force.

### 3.6.3. Strain Gages

To measure the non-uniform distribution of normal strain around the branch footprint, numerous strain gages oriented along the longitudinal axis of the branch were installed (Figure 3-17). The strain gages were placed approximately $\frac{9}{16}$ in. above the fillet welds to avoid the high strain region immediately adjacent to the weld toe caused by the notch effect (Cassidy, 1993). Theoretically, under pure in-plane bending moment, the distribution of strain around the branch footprint on both sides of the chord longitudinal centerline is symmetric. Hence, strain gages were installed only around half the branch perimeter. An additional gage was placed at the theoretical zero stress region on the opposite longitudinal mid-wall of the branch to monitor any significant out-of-plane effects throughout testing. Four strain gages were placed at the mid-wall locations around the branch perimeter in the constant stress region which is approximately three times the branch width above the connection (Mehrotra & Govil, 1972). This was another way of monitoring any significant out-of-plane effects throughout testing.

![Figure 3-17 Strain gages adjacent to the welded joint, oriented along the branch longitudinal axis](image)

(a) Test specimens with $0.25 < \beta \leq 1.00$  
(b) Test specimens with $\beta = 0.25$

For test specimens with $0.25 < \beta \leq 1.00$, 14 strain gages were installed around the branch footprint (18 in total, including those in the constant stress region). An overall view of the strain gage locations, orientations, spacing around the branch footprint and
Effective Weld Properties for RHS-to-RHS Moment T-Connections

labelling format is shown in Figure 3-18. All of the strain gages measured unidirectional strain along the longitudinal axis of the branch with exception to SG-1S and SG-1N, which measured strain in three directions: 0°, 45° and 90° to the longitudinal axis of the branch (known as rosette strain gages).

For test specimens with $\beta = 0.25$, 10 strain gages were installed around the branch footprint (14 in total, including those in the constant stress region). Since the overall branch width was only 2-in., it was physically impossible to fit 14 strain gages around half of the branch footprint. An overall view of the strain gage locations, orientations, spacing around the branch footprint and labelling format is shown in Figure 3-19. All of the strain gages measured unidirectional strain along the longitudinal axis of the branch with the exception of SG-1S and SG-1N, which were rosette strain gages and measured strain in three directions.
Effective Weld Properties for RHS-to-RHS Moment T-Connections

Figure 3-18 General strain gage details for test specimens with $0.25 < \beta \leq 1.00$
Figure 3-19 General strain gage details for test specimens with $\beta = 0.25$

(a) Global view of strain gage locations and orientations

(b) Spacing of strain gages adjacent to the welded joint around the branch footprint

(c) Strain gage labelling format adjacent to the welded joint
3.6.4. LED Targets

To determine the actual applied forces and moment at the connection, branch deflection and chord wall deformation profiles throughout the experiments, a K610 optical camera that records the coordinates of strobing light emitting diode (LED) targets was used. The LED targets were mounted to the test specimens and test setup assembly in several locations, shown in Figure 3-20. Their coordinates in the x, y and z axes were recorded 10 times every second (10 Hz) for the duration of each experiment.

Numerous LED targets were mounted to the branch along its height. The data was used to plot the branch deformation profile from the base of the connection to the point of application of the load. Data from certain targets was only used for real-time monitoring during tests to ensure that no significant, undesirable out-of-plane effects were occurring. Hence, some of the data from those targets is not included in the analysis.
Drawings with detailed information about the test setup and LED target locations can be found in Appendix D.

Four LED targets were placed along the longitudinal centerline of the chord adjacent to the branch walls on each side for test specimens with $0.25 \leq \beta \leq 0.85$ subject to the limit state of chord face plastification, as shown in Figure 3-20 (a). The initial intention for spacing the targets was to have them extend slightly beyond the theoretical transverse yield line in the chord face, spaced consistently in proportion to the calculated distance from the branch wall to the yield line. However, as $\beta$ increased, it became physically impossible to space the targets so closely together and therefore the spacing of the targets is somewhat arbitrary. The data was used to plot chord face deformation profiles from the initial unloaded state to failure.

For test specimens with $\beta = 1.00$, three LED targets were mounted to the vertical chord side wall on the compression side of the connection, as shown in Figure 3-20 (b). Spaced 2-in. from the top and bottom of the chord and 2-in. between each other, the data recorded from the LED targets was used to plot the chord side wall out-of-plane deformation profiles from the initial unloaded state to failure.

Although the actuator appeared to remain horizontal and in-plane for most tests, the actual force components ($\vec{F}$) in the x, y and z directions were calculated using the coordinate readings from the LED targets and force readings from the MTS actuator. A position vector ($\vec{r}$) between two LED targets on the MTS actuator and corresponding unit vector ($\vec{u}$) was calculated and then multiplied by the magnitude of force ($F$) applied by the actuator to determine the force components at each load step. In Cartesian vector form this is expressed as:

$$\vec{F} = \vec{F}\vec{u} = F\left(\frac{\vec{r}}{r}\right) = F\left(\frac{(x_B - x_A)i + (y_B - y_A)j + (z_B - z_A)k}{\sqrt{(x_B - x_A)^2 + (y_B - y_A)^2 + (z_B - z_A)^2}}\right) = [(F_x)i, (F_y)j, (F_z)k]$$

Using force components the in-plane moment, out-of-plane moment and torsion acting on the connection could be calculated. This was done by taking a position vector ($\vec{r}_0$) directed from the centroid of the connection to a point on the line of action of the force.
and calculating the cross product (or determinant). In Cartesian vector form this is expressed as:

\[
\overrightarrow{M_0} = \overrightarrow{r_0} \times \overrightarrow{F} = \begin{bmatrix}
\vec{i} & \vec{j} & \vec{k} \\
 r_{0-x} & r_{0-y} & r_{0-z} \\
 F_x & F_y & F_z
\end{bmatrix} = [(M_{0-x})\vec{i}, (M_{0-y})\vec{j}, (M_{0-z})\vec{k}]
\]

where

\(r_{0-x}, r_{0-y}, r_{0-z}\) represent the x, y, z components of the position vector, \(\overrightarrow{r_0}\), directed from the centroid of the connection to a point on the line of action of the force.

\(F_x, F_y, F_z\) represent the x, y, z components of the force vector, \(\overrightarrow{F}\).

\(M_{0-x}, M_{0-y}, M_{0-z}\) represent the out-of-plane moment, torsion and in-plane moment (respectively) of the moment vector, \(\overrightarrow{M_0}\).

### 3.6.5. LVDTs

Although the LED targets were sufficient for measuring the branch deflection, a string-pot LVDT mounted to an aluminum frame was attached to the backside of the point load device at the height of the initial moment arm. The intention was to use the data from the string-pot LVDT in case the K610 optimal camera or its data acquisition system shut down in the middle of a test, preventing loss of data. Fortunately, this did not happen and the data from the string-pot LVDT was not used for analysis.

One horizontal 1-in. LVDT was installed at the free-end of the chord at the roller support to monitor significant horizontal displacement during testing. Another 1-in. LVDT was installed vertically on the pin support’s rigid base plate to monitor uplift. Since the anchor bolts were only installed at the front holes of the rigid base plate, there was concern that it could uplift under higher loads. However, this was not observed during any test.

### 3.7. Weld Size Reduction

After the first three tests (T-0.50-34, T-0.50-23 and T-0.50-17) it became apparent that the welds were stronger than anticipated. To ensure that the remaining nine test
specimens would be weld-critical, it was decided to reduce their weld leg size by approximately $\frac{1}{8}$-in. A hand-held grinder with a metal grinding wheel was used to carefully grind the welds to the final sizes prescribed in Table 3-4. The result was continuous weld throat sizes around the branch perimeter and fillet welds with near-triangular cross-sections.

Figure 3-21 Comparison of welded joint before and after reducing the weld size for T-0.75-23

Figure 3-22 Comparison of welded joint before and after reducing the weld size for T-1.00-23

Figure 3-21 shows a comparison of the appearance of test specimen T-0.75-23 before and after grinding. The originally specified weld size was $\frac{1}{4}$-in. and it was machined with a hand grinder to $\frac{3}{16}$-in. Figure 3-22 shows a comparison of the appearance of test specimen T-1.00-23 before and after grinding. The originally specified effective weld throat was $\frac{1}{4}$-in. and it was ground with a hand grinder to $\frac{3}{16}$-in.
Chapter 4: Experimental Results

This chapter contains the actual geometric and material properties of the RHS and as-laid welds. The strength tables from Chapter 3 are re-calculated using the average measured geometric and material properties and the predicted LRFD and nominal branch, connection and weld flexural strengths based on ANSI/AISC 360 (2010) are summarized in Appendix A.2. In addition the results and observations from full-scale tests on RHS-to-RHS moment T-connections are presented which includes the failure modes, actual flexural strength, moment-deflection relationships, normal strain distribution plots around the branch perimeter, branch deflection profiles and chord wall deformation profiles for each test specimen.

4.1. Actual Geometric and Material Properties of RHS

The geometric and material properties of the RHS are summarized in this section. Measured cross-sectional dimensions for each RHS are compared with the dimensional tolerances for cold-formed RHS made to the ASTM A500 specification (STI, 1993; ASTM, 2010). The yield strength \( (F_y, F_{yb}) \), yield strain \( \varepsilon_y \), ultimate tensile strength \( (F_u, F_{ub}) \), elongation at rupture \( \varepsilon_u \) and Young’s modulus \( E \) determined from tensile coupon (TC) tests are reported herein. The results are compared with the minimum specified requirements for cold-formed RHS made to ASTM A500 Grade C as well as with the reported values in the mill certificates provided by the steel manufacturer.

4.1.1. Geometric Properties of RHS

The cross-sectional dimensions of each RHS used in the experimental program were measured and the average values are presented in Table 4-1. The traced cross-sections of the RHS material showing the individual measurements are located in Appendix E.1. The measured geometric properties were checked with the dimensional tolerances for cold-formed RHS made to ASTM A500 specification (STI, 1993; ASTM, 2010) and a comparative analysis is presented in Table 4-2.
Table 4-1 Average measured cross-sectional dimensions of RHS

<table>
<thead>
<tr>
<th>RHS Designation</th>
<th>Height and width, H and B (in.)</th>
<th>Wall thickness, t (in.)</th>
<th>Cross-sectional area, A (in.²)</th>
<th>Outer radius (in.)</th>
<th>Inner radius (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSS 2 x 2 x 1/4&quot;</td>
<td>2.014</td>
<td>0.2266</td>
<td>1.516</td>
<td>0.482</td>
<td>0.248</td>
</tr>
<tr>
<td>HSS 4 x 4 x 1/4&quot;</td>
<td>4.017</td>
<td>0.2254</td>
<td>3.344</td>
<td>0.492</td>
<td>0.282</td>
</tr>
<tr>
<td>HSS 4 x 4 x 1/2&quot;</td>
<td>4.022</td>
<td>0.4576</td>
<td>6.108</td>
<td>0.945</td>
<td>0.476</td>
</tr>
<tr>
<td>HSS 6 x 6 x 1/4&quot;</td>
<td>6.008</td>
<td>0.2259</td>
<td>5.128</td>
<td>0.509</td>
<td>0.298</td>
</tr>
<tr>
<td>HSS 6 x 6 x 3/8&quot;</td>
<td>6.000</td>
<td>0.3422</td>
<td>7.483</td>
<td>0.772</td>
<td>0.416</td>
</tr>
<tr>
<td>HSS 6 x 6 x 1/2&quot;</td>
<td>6.006</td>
<td>0.4594</td>
<td>9.669</td>
<td>1.155</td>
<td>0.671</td>
</tr>
<tr>
<td>HSS 8 x 8 x 1/4&quot;</td>
<td>8.018</td>
<td>0.2317</td>
<td>7.065</td>
<td>0.639</td>
<td>0.398</td>
</tr>
<tr>
<td>HSS 8 x 8 x 3/8&quot;</td>
<td>7.986</td>
<td>0.3441</td>
<td>10.144</td>
<td>0.939</td>
<td>0.588</td>
</tr>
<tr>
<td>HSS 8 x 8 x 1/2&quot;</td>
<td>8.052</td>
<td>0.4560</td>
<td>13.138</td>
<td>1.356</td>
<td>0.875</td>
</tr>
</tbody>
</table>

The values in bold font represent measurements that have exceeded the allowable tolerances. It is surprising to see that the wall thickness can be so thin at some locations and yet the cross-sectional area for each RHS is approximately equal to the nominal specified area. When comparing the averages of those dimensions (instead of the maximum and minimum measurements), none of them violate the dimensional tolerance criteria. Thus, the violations should not have a significant impact on the RHS flexural performance, especially when considering the design wall thickness (AISC, 2010) instead of the specified nominal wall thickness. It is valuable to note that the minimum outside corner radii for all RHS used in the experimental program were larger than 2.00t. This geometric property is of high interest because of corner cracking which can occur when the corner radius is too small.
Table 4-2 Comparison of the measured geometric properties with the dimensional tolerances for cold-formed RHS made to ASTM A500 specification (STI, 1993; ASTM, 2010)

<table>
<thead>
<tr>
<th>RHS Designation</th>
<th>Outside dimension tolerance</th>
<th>Minimum wall thickness</th>
<th>Cross-sectional area</th>
<th>Outside corner radius</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H and B (in.)</td>
<td>t (in.)</td>
<td>A (%)</td>
<td>Maximum (in.)</td>
</tr>
<tr>
<td>HSS 2 x 2 x (\frac{1}{4})&quot;</td>
<td>+0.017</td>
<td>0.2240 (-10.4%)</td>
<td>+0.40</td>
<td>0.500 (2.21t)</td>
</tr>
<tr>
<td>HSS 4 x 4 x (\frac{1}{4})&quot;</td>
<td>+0.026</td>
<td>0.2190 (-12.4%)</td>
<td>-0.77</td>
<td>0.540 (2.40t)</td>
</tr>
<tr>
<td>HSS 4 x 4 x (\frac{1}{2})&quot;</td>
<td>+0.040</td>
<td>0.4530 (-9.40%)</td>
<td>+1.46</td>
<td>0.980 (2.14t)</td>
</tr>
<tr>
<td>HSS 6 x 6 x (\frac{1}{4})&quot;</td>
<td>+0.018</td>
<td>0.2189 (-12.4%)</td>
<td>-2.14</td>
<td>0.550 (2.43t)</td>
</tr>
<tr>
<td>HSS 6 x 6 x (\frac{3}{8})&quot;</td>
<td>-0.032</td>
<td>0.3391 (-9.57%)</td>
<td>-1.28</td>
<td>0.810 (2.37t)</td>
</tr>
<tr>
<td>HSS 6 x 6 x (\frac{1}{2})&quot;</td>
<td>+0.018</td>
<td>0.4542 (-9.16%)</td>
<td>-0.73</td>
<td>1.220 (2.66t)</td>
</tr>
<tr>
<td>HSS 8 x 8 x (\frac{1}{4})&quot;</td>
<td>+0.047</td>
<td>0.2273 (-9.08%)</td>
<td>-0.50</td>
<td>0.710 (3.06t)</td>
</tr>
<tr>
<td>HSS 8 x 8 x (\frac{3}{8})&quot;</td>
<td>-0.104</td>
<td>0.3375 (-10.0%)</td>
<td>-2.46</td>
<td>1.055 (3.07t)</td>
</tr>
<tr>
<td>HSS 8 x 8 x (\frac{1}{2})&quot;</td>
<td>+0.068</td>
<td>0.4495 (-10.1%)</td>
<td>-2.68</td>
<td>1.625 (3.56t)</td>
</tr>
</tbody>
</table>

4.1.2. RHS Material Properties

The results from 27 individual TC tests are summarized here. Three TCs for each RHS used in the experimental program were tested in accordance with the standard methods for tension testing of metallic materials (ASTM, 2008). A 225-kip capacity MTS test frame, shown in Figure 4-1(a), was used to load the TCs to failure. The initial test rate was set to 0.0002 in/s. Once the rounded yield region was passed, the test rate was increased to 0.0003 in/s. After reaching the tensile strength, the test rate was increased to 0.0006 in/s. During testing, the axial strain was measured by a MTS extensometer (Model# 632.12C-20) which was attached to the coupon at the beginning of each test.
The specified yield strength and corresponding yield strain of the material was later determined using the 0.2% offset method (ASTM, 2008). Stress-strain diagrams for each TC specimen are located in Appendix E.3. The averaged stress-strain plots are also provided and the results from those are summarized in Table 4-3.

### Table 4-3 RHS tensile coupon test results

<table>
<thead>
<tr>
<th>RHS Designation</th>
<th>$F_y$ (ksi)</th>
<th>$\varepsilon_y$ ($\times 10^3 \mu\varepsilon$)</th>
<th>$F_u$ (ksi)</th>
<th>$\varepsilon_u$ (%)</th>
<th>$E$ ($\times 10^3$ ksi)</th>
<th>$F_y/F_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSS 2 x 2 x $\frac{1}{4}$&quot;</td>
<td>59.3</td>
<td>2.27</td>
<td>67.5</td>
<td>21.3</td>
<td>26.2</td>
<td>0.879</td>
</tr>
<tr>
<td>HSS 4 x 4 x $\frac{1}{4}$&quot;</td>
<td>62.1</td>
<td>1.99</td>
<td>76.2</td>
<td>27.3</td>
<td>31.1</td>
<td>0.815</td>
</tr>
<tr>
<td>HSS 4 x 4 x $\frac{1}{2}$&quot;</td>
<td>63.9</td>
<td>2.58</td>
<td>79.1</td>
<td>26.4</td>
<td>24.8</td>
<td>0.808</td>
</tr>
<tr>
<td>HSS 6 x 6 x $\frac{1}{4}$&quot;</td>
<td>48.0</td>
<td>1.86</td>
<td>63.6</td>
<td>33.2</td>
<td>25.8</td>
<td>0.755</td>
</tr>
<tr>
<td>HSS 6 x 6 x $\frac{3}{8}$&quot;</td>
<td>50.7</td>
<td>1.94</td>
<td>61.5</td>
<td>33.9</td>
<td>26.2</td>
<td>0.824</td>
</tr>
<tr>
<td>HSS 6 x 6 x $\frac{1}{2}$&quot;</td>
<td>53.8</td>
<td>2.03</td>
<td>64.3</td>
<td>34.0</td>
<td>26.5</td>
<td>0.837</td>
</tr>
<tr>
<td>HSS 8 x 8 x $\frac{1}{4}$&quot;</td>
<td>55.4</td>
<td>2.07</td>
<td>71.5</td>
<td>27.5</td>
<td>26.9</td>
<td>0.775</td>
</tr>
<tr>
<td>HSS 8 x 8 x $\frac{3}{8}$&quot;</td>
<td>57.1</td>
<td>2.24</td>
<td>73.9</td>
<td>32.3</td>
<td>25.5</td>
<td>0.773</td>
</tr>
<tr>
<td>HSS 8 x 8 x $\frac{1}{2}$&quot;</td>
<td>59.8</td>
<td>2.24</td>
<td>73.8</td>
<td>34.2</td>
<td>26.8</td>
<td>0.810</td>
</tr>
</tbody>
</table>

*Figure 4-1 RHS tensile coupon specimens*

All of the TCs exhibited initial linear-elastic behavior up to the specified yield strength (determined by the 0.2% strain offset method) which was located in a somewhat rounded yield region. This region indicates that there is a large amount of residual...
stress in the cold-formed ASTM A500 RHS even along the flats of the walls. Another indication of high residual stresses in the material was the curvature of the TCs immediately after being extracted from the parent tube, Figure 4-1(b). Those with thinner wall thicknesses had larger curvatures.

Figure 4-2 Typical stress-strain plot from RHS tensile coupon tests

Beyond the rounded region the TCs underwent minor strain-hardening up to the tensile strength and then gradually decreased until fracture. The extensometer was removed shortly after reaching the ultimate tensile strength to avoid damage when the TC fractures. The elongation at failure was determined after the test was completed by joining the fractured pieces together and recording the change in gage length divided by the initial gage length. The TCs were ductile and generally elongated well beyond the minimum specified requirement (21% elongation), the exception being those tested for the HSS 2 x 2 x \( \frac{1}{4} \)" which had an average \( \varepsilon_u \) equal to 21.3%. Figure 4-2 shows a typical stress-strain plot with the averaged material properties of the RHS.
### Table 4-4 Comparison between the specified minimum requirements and actual material properties of RHS

<table>
<thead>
<tr>
<th>RHS Designation</th>
<th>Specified Minimum</th>
<th>Actual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( F_y ) (ksi)</td>
<td>( F_u ) (ksi)</td>
</tr>
<tr>
<td>HSS 2 x 2 x ( \frac{1}{4} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
<tr>
<td>HSS 4 x 4 x ( \frac{1}{4} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
<tr>
<td>HSS 4 x 4 x ( \frac{1}{2} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
<tr>
<td>HSS 6 x 6 x ( \frac{1}{4} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
<tr>
<td>HSS 6 x 6 x ( \frac{3}{8} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
<tr>
<td>HSS 6 x 6 x ( \frac{1}{2} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
<tr>
<td>HSS 8 x 8 x ( \frac{1}{4} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
<tr>
<td>HSS 8 x 8 x ( \frac{3}{8} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
<tr>
<td>HSS 8 x 8 x ( \frac{1}{2} )&quot;</td>
<td>50</td>
<td>62</td>
</tr>
</tbody>
</table>

A comparison between the specified minimum required material properties for cold-formed RHS made to ASTM A500 Grade C and the actual material properties recorded from the TC tests is made in Table 4-4. The actual material properties exceeded the specified nominal material properties for most sizes of RHS with the exception of the yield strength for the HSS 6 x 6 x \( \frac{1}{4} \)"", which was 4% below, and the ultimate tensile strength for the HSS 6 x 6 x \( \frac{3}{8} \)"", which was less than 1% below.

Another comparison between the material properties reported in the mill certificates provided by the product manufacturer and the actual material properties observed from the TC tests is shown in Table 4-5. It is interesting to note that the actual material properties measured in the laboratory TC tests are lower than all of those reported in the mill certificates. The original mill certificates are located in Appendix E.4.

The resistance tables in Appendix A.2 were re-calculated using the actual geometric and material properties of the RHS to determine the predicted LRFD and nominal flexural strengths of the connection and the welded joint for each test specimen.
Table 4-5 Comparison between actual material properties of RHS and those reported in the mill certificates provided by the product manufacturer

<table>
<thead>
<tr>
<th>RHS Designation</th>
<th>Mill Certificates</th>
<th>Actual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$F_y$ (ksi)</td>
<td>$F_u$ (ksi)</td>
</tr>
<tr>
<td>HSS 2 x 2 x $\frac{1}{4}''$</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>HSS 4 x 4 x $\frac{1}{4}''$</td>
<td>66.1</td>
<td>83.6</td>
</tr>
<tr>
<td>HSS 4 x 4 x $\frac{1}{2}''$</td>
<td>78.3</td>
<td>85.1</td>
</tr>
<tr>
<td>HSS 6 x 6 x $\frac{1}{4}''$</td>
<td>59.0</td>
<td>72.0</td>
</tr>
<tr>
<td>HSS 6 x 6 x $\frac{3}{8}''$</td>
<td>56.4</td>
<td>67.5</td>
</tr>
<tr>
<td>HSS 6 x 6 x $\frac{1}{2}''$</td>
<td>54.4</td>
<td>65.1</td>
</tr>
<tr>
<td>HSS 8 x 8 x $\frac{1}{4}''$</td>
<td>57.0</td>
<td>78.4</td>
</tr>
<tr>
<td>HSS 8 x 8 x $\frac{3}{8}''$</td>
<td>64.7</td>
<td>80.0</td>
</tr>
<tr>
<td>HSS 8 x 8 x $\frac{1}{2}''$</td>
<td>64.4</td>
<td>78.8</td>
</tr>
</tbody>
</table>

4.2. Actual Geometric and Material Properties of the As-Laid Welds

The geometric and material properties of the as-laid welds are presented in this section. The average measured effective weld throats for the individual weld elements around the branch footprint (north, south, east and west) are summarized. The yield strength ($F_y$), tensile strength ($F_{uu}$), elongation at failure ($\varepsilon_u$) and Young’s modulus ($E$) determined from all-weld-metal TC tests are also reported herein. The results are compared with the specified minimum requirements for AWS ER70S-6 solid wire electrodes (AWS, 2010).

4.2.1. Weld Size Measurements

The theoretical effective weld throats of the welded joints tested in the experimental program were measured externally prior to testing using a fillet weld gage to ensure that the test specimen would be weld-critical. These external measurements were not used to calculate the predicted LRFD and nominal flexural strengths of the welded joints. The average values for the external measurements of the individual weld elements are summarized in Table 4-6.
Since the current design requirements of ANSI/AISC 360 (2010) are based solely on the limit state of shear rupture along the plane of the effective weld throat, the average measured values for the vertical and horizontal weld leg sizes, \( L_V \) and \( L_H \) (respectively), are not presented in this section. Instead, they are located in Appendix F.1 for the external measurements made using the fillet weld gage and in Appendix F.2 for the measurements recorded from the macroetch examinations.

Table 4-6 Average effective weld throat sizes measured using a fillet weld gage (external measurements)

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>North Weld (in.)</th>
<th>South Weld (in.)</th>
<th>East Weld (in.)</th>
<th>West Weld (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>0.08</td>
<td>0.09</td>
<td>0.09</td>
<td>0.09</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>0.09</td>
<td>0.09</td>
<td>0.09</td>
<td>0.09</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>0.09</td>
<td>0.09</td>
<td>0.09</td>
<td>0.09</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>0.13</td>
<td>0.12</td>
<td>0.14</td>
<td>0.13</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>0.12</td>
<td>0.11</td>
<td>0.13</td>
<td>0.12</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>0.28</td>
<td>0.30</td>
<td>0.29</td>
<td>0.27</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>0.09</td>
<td>0.09</td>
<td>0.09</td>
<td>0.08</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>0.15</td>
<td>0.13</td>
<td>0.14</td>
<td>0.13</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>0.23</td>
<td>0.22</td>
<td>0.15</td>
<td>0.18</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>0.13</td>
<td>0.13</td>
<td>0.19</td>
<td>0.17</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>0.19</td>
<td>0.19</td>
<td>0.43</td>
<td>0.41</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>0.23</td>
<td>0.24</td>
<td>0.44</td>
<td>0.39</td>
</tr>
</tbody>
</table>

Note: Values in bold are PJP flare-bevel-groove welds that were measured in accordance with Table J2.2 (AISC, 2010), equal to \( 5/8 \) times the average outside radius of the RHS corners (assumed to be equal to \( 2t \)) less the greatest perpendicular dimension measured from a line flush from the base metal surface to the weld surface.

The average effective weld throats measured from the macroetch examinations for the individual weld elements of each test specimen are summarized in Table 4-7. Since these measurements represent the actual cross-sectional geometry of the welds, they were used to calculate the effective elastic section modulus for in-plane bending postulated in Table K4.1 (AISC, 2010). All of the scanned weld cross-sections are located in Appendix F.2. A consistent observation from the macroetch examinations is that the failure plane through the fillet welds of the stepped connections is, on average,
at an angle considerably larger than the commonly assumed 45°. Also, the effective weld throat measurements recorded for the individual weld elements of the stepped connections are very similar in Table 4-6 and Table 4-7.

When comparing those for the matched connections, specifically the measurements for the PJP flare-bevel-groove welds (east and west weld elements) of test specimens T-1.00-23 and T-1.00-12, there are considerable differences. It is evident that the requirements of Table J2.2 (AISC, 2010) for estimating the effective throat size of PJP groove welds not filled flush to the surface of the connected branch provide a significant over-estimation.

Table 4-7 Average effective weld throat sizes measured from the macroetch examinations

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>North Weld (in.)</th>
<th>South Weld (in.)</th>
<th>East Weld (in.)</th>
<th>West Weld (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>0.097</td>
<td>0.119</td>
<td>0.098</td>
<td>0.100</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>0.091</td>
<td>0.148</td>
<td>0.106</td>
<td>0.072</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>0.079</td>
<td>0.122</td>
<td>0.124</td>
<td>0.142</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>0.124</td>
<td>0.132</td>
<td>0.144</td>
<td>0.131</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>0.125</td>
<td>0.131</td>
<td>0.168</td>
<td>0.185</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>0.247</td>
<td>0.271</td>
<td>0.280</td>
<td>0.267</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>0.109</td>
<td>0.107</td>
<td>0.084</td>
<td>0.131</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>0.137</td>
<td>0.154</td>
<td>0.162</td>
<td>0.138</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>0.227</td>
<td>0.246</td>
<td>0.157</td>
<td>0.230</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>0.126</td>
<td>0.092</td>
<td><strong>0.094</strong></td>
<td><strong>0.149</strong></td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>0.179</td>
<td>0.274</td>
<td><strong>0.198</strong></td>
<td><strong>0.120</strong></td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>0.235</td>
<td>0.384</td>
<td><strong>0.254</strong></td>
<td><strong>0.153</strong></td>
</tr>
</tbody>
</table>

Note: Values in bold are PJP flare-bevel-groove welds that were measured along the actual plane of failure through the weld (if the cross-section was fractured through the weld). Otherwise, it was taken as the shortest length through the weld.

Another interesting difference is the effective weld throat size of the fillet welds measured on the tension side (south weld element) of the connection. Those from the macroetch examinations are considerably larger for test specimens T-1.00-23 and T-
than those from the external measurements. As shown in Figures F-16, F-18 and F-20 (Appendix F.2), the angle of inclination of the failure planes is very steep and appears to be along the vertical plane of the fusion face with the connected branch. One exception is test specimen T-1.00-34; as shown in Figure F-16 (c), the fusion with the chord is inadequate and as a result, the failure plane is through the weld.

4.2.2. As-Laid Weld Material Properties

The results from three all-weld-metal TC specimens are reported here. Each was tested in accordance with the standard methods for tension testing of metallic materials (ASTM, 2008). A 55-kip capacity MTS test frame, shown in Figure 4-3 (a), was used to load the TCs to failure. Prior to testing, the dimensions of the TCs were measured using the Mitutoyo digimatic micrometer and calipers. The measurements for the all-weld-metal TCs are located in Appendix F.3. The initial test rate was set to 0.0002 in/s. When strain hardening became apparent the rate was increased to 0.0003 in/s until failure. During testing, the axial strain was measured by a MTS extensometer (Model# 632.12F-20) which was attached to the coupon at the beginning of each test. The specified yield strength of the material was later determined using the 0.2% offset method (ASTM, 2008). Stress-strain diagrams for the individual all-weld-metal TC tests are provided in Appendix F.4 and the results are summarized in Table 4-8.

Table 4-8 All-weld-metal tensile coupon test results

<table>
<thead>
<tr>
<th>Coupon Designation</th>
<th>$F_y$ (ksi)</th>
<th>$E$ ($\times 10^3$ ksi)</th>
<th>$F_{exx}$ (ksi)</th>
<th>$\varepsilon_u$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ i ]</td>
<td>76.2</td>
<td>29.6</td>
<td>90.0</td>
<td>29.2</td>
</tr>
<tr>
<td>[ ii ]</td>
<td>75.7</td>
<td>31.6</td>
<td>86.9</td>
<td>28.3</td>
</tr>
<tr>
<td>[ iii ]</td>
<td>75.8</td>
<td>29.6</td>
<td>87.5</td>
<td>28.0</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>75.9</strong></td>
<td><strong>30.3</strong></td>
<td><strong>88.1</strong></td>
<td><strong>28.5</strong></td>
</tr>
</tbody>
</table>
All of the measured material properties exceeded the minimum requirements for AWS ER70S-6 solid wire electrodes. The tensile strength of the as-laid weld metal was 26\% stronger than the nominal specified tensile strength (70 ksi). This over-strength of the material was the main reason why the originally specified weld sizes (Table 3-3) had to be reduced. For future experiments on weld-critical connections, it would be beneficial to know the actual tensile strength of the weld metal ($F_{Eyx}$) prior to designing the test specimen weld sizes.

Every all-weld-metal TC exhibited linear-elastic behavior until the upper yield point which was followed by an immediate drop in stress and subsequent yield plateau region. Shortly after, strain hardening initiated until the tensile strength of the material was achieved. The extensometer used to measure the strain was removed shortly after reaching the ultimate tensile strength to avoid damage when the TC fractures. The elongation at failure was determined after the test was completed by joining the fractured pieces together and recording the change in gage length divided by the initial gage length. The TCs were very ductile and elongated well beyond the minimum specified requirement (22\% elongation). Figure 4-4 shows the stress-strain plot with the averaged material properties of the weld metal.
The resistance tables in Appendix A.2 were re-calculated using the actual geometric and material properties of the as-laid welds to determine the predicted LRFD and nominal flexural strengths of the welded joints.

4.3. Results from Full-Scale Tests on RHS-to-RHS Moment T-Connections

The results and observations from 12 full-scale tests on RHS-to-RHS moment T-connections are presented in this section. A summary of the actual flexural strength (or ultimate moment) of the welded joints with the predicted LRFD and nominal flexural strengths calculated using the equation for the effective elastic section modulus for in-plane bending (AISC, 2010) and the actual geometric and material properties of the RHS material and as-laid weld metal is provided in Table 4-9. The average effective weld throat measurements from the macroetch examinations summarized Table 4-7 were used instead those measured externally (prior to testing) with the fillet weld gage. Every weld-critical test specimen failed at a moment considerably higher than the predicted nominal flexural strength, which is expected since the equation for the
Effective elastic section modulus for in-plane bending is based on yield line theory and provides a lower bound design method.

Table 4-9 Predicted LRFD and nominal flexural strengths of the welded joints as well as the actual flexural strengths of the welded joints for the individual test specimens

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>$\beta$</th>
<th>Chord Wall Slenderness</th>
<th>Nominal Flexural Strength</th>
<th>LRFD Flexural Strength</th>
<th>Actual Flexural Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$M_{n-ipt}$ (kip-ft)</td>
<td>$\phi M_{n-ipt}$ (kip-ft)</td>
<td>$M_{ultimate}$ (kip-ft)</td>
</tr>
<tr>
<td>T-0.25-34</td>
<td>0.25</td>
<td>34</td>
<td>1.121</td>
<td>0.840</td>
<td>4.11</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>0.25</td>
<td>23</td>
<td>1.876</td>
<td>1.407</td>
<td>4.37</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>0.25</td>
<td>17</td>
<td>2.41</td>
<td>1.811</td>
<td>4.82</td>
</tr>
<tr>
<td>T-0.50-34†</td>
<td>0.50</td>
<td>34</td>
<td>4.81</td>
<td>4.81</td>
<td>11.71</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>0.50</td>
<td>23</td>
<td>7.31</td>
<td>5.48</td>
<td>15.25</td>
</tr>
<tr>
<td>T-0.50-17†</td>
<td>0.50</td>
<td>17</td>
<td>20.1</td>
<td>20.1</td>
<td>29.6</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>0.75</td>
<td>34</td>
<td>8.35</td>
<td>6.26</td>
<td>14.37</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>0.75</td>
<td>23</td>
<td>13.23</td>
<td>9.92</td>
<td>27.1</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>0.75</td>
<td>17</td>
<td>21.7</td>
<td>16.26</td>
<td>52.2</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>1.00</td>
<td>34</td>
<td>15.01</td>
<td>11.83</td>
<td>39.7</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>1.00</td>
<td>23</td>
<td>25.8</td>
<td>20.1</td>
<td>64.7</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>1.00</td>
<td>17</td>
<td>39.4</td>
<td>30.5</td>
<td>93.6</td>
</tr>
</tbody>
</table>

† Connection failure preceded weld rupture. Predicted LRFD and nominal flexural strengths are those for the connection instead of the welded joint.
Figure 4-5 summarizes the failure modes observed during the experimental program. Ten out of 12 test specimens failed by weld rupture. The two specimens that were not weld-critical (T-0.50-34 and T-0.50-17) failed by rupture of the chord face (or punching shear) on the tension side of the connection, after extensive chord face plastification.
These were tested early in the experimental program and indicated that the actual tensile strength of the weld metal may be considerably higher than the specified nominal strength. As a result, the remaining test specimens underwent weld size reductions to ensure weld-critical behavior, which was successfully achieved.

4.3.1. Discussion on the Performance of the Test Setup Assembly

Before describing the results for each test specimen it is worth noting some of the major changes that were made to the original test setup and instrumentation in order to optimize the performance and data acquisition for subsequent tests. The specimens were tested in the following order:

1. T-0.50-34  
2. T-0.50-23  
3. T-0.50-17  
4. T-0.75-17  
5. T-0.75-23  
6. T-0.75-34  
7. T-1.00-23  
8. T-1.00-17  
9. T-1.00-34  
10. T-0.25-34  
11. T-0.25-23  
12. T-0.25-17

Some issues were encountered during first test, T-0.50-34, and as a result the test was completed over the course of two days. The major problem was encountered on the first day of testing - the branch deflection was larger than anticipated. Consequently, the actuator reached its maximum stroke (displacement capacity) in the original test setup configuration. To limit the amount of branch deflection for subsequent tests the lever arm was decreased. It was decided not to do so for the final load stage of the first test because it could have posed some complications for the analysis. The alternative was to move the entire test specimen a further 12-in. (horizontally) away from the test setup assembly. Because of this, the K610 optical camera also had to be moved in order to “see” all of the LED targets, which meant that the LED target coordinates were completely different between both days of testing. Thus, it was decided not to use the data from the K610 optical camera for the analysis of specimen T-0.50-34. Instead, the branch deflections were measured by the MTS actuator LVDT and the original lever arm (42.5-in.) at the initial, unloaded state was used to calculate the applied moment at the connection.
Based on the behavior of the test setup assembly for T-0.50-34 it was decided to lower the actuator by nine inches. With a smaller lever arm (approximately 33.5-in.), the branch deflections were proportionately smaller to those observed previously. This, in addition to moving the specimen 12-in. (horizontally) away from the rigid steel support frame ensured that the maximum available stroke in the actuator was not exceeded in subsequent tests.

Some changes to the instrumentation were undertaken for the second test (T-0.50-23) as well as all subsequent tests. First, the LVDT attached to the point load device (intended to measure the branch deflection) was replaced by a branch string-pot LVDT. Since the branch displaces in both the horizontal and vertical directions, the original LVDT was not able to accurately measure the resultant of the displacement. The branch string-pot LVDT was successful in doing so and was used for all subsequent tests. Four additional strain gages were added to the mid-wall locations around the branch perimeter at a vertical distance from the connection equal to approximately $3 \times B_b$ (in the constant stress region). Since the data from the LED targets could not be analyzed in real-time during testing, these strain gages were an effective monitoring tool for significant out-of-plane effects. The moment-strain relationships from these strain gages are plotted in Appendix H and confirm the exceptional performance of the test setup assembly in applying nearly-pure shear and bending moment to the connections. Finally, the sampling rates of the HBM MGCplus data acquisition system and K610 optical camera were synchronized to 10 Hz (from 5Hz and 16 Hz, respectively).

The only change that was made after the second test was lowering the actuator to a height of 18.5-in. above the connection for the test specimens with $\beta = 0.25$. By decreasing the moment arm, the deflection would be limited and a higher point load applied by the actuator would be required to obtain the ultimate moment. Because of the connection parameters, large deflections were anticipated and lower ultimate strengths were predicted. This decision proved to be successful since the maximum stroke of the actuator was not reached. In addition, the out-of-plane effects acting on the connection were insignificant for all three test specimens with $\beta = 0.25$. 

4.3.2. Test Specimen T-0.25-34

T-0.25-34 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.010 in/s after the ultimate moment was reached. The welded joint failed very suddenly by rupture along a plane through the weld (see macroetch examination in Appendix F.2). Unlike the test specimens with $\beta > 0.25$, where failure was gradual after crack initiation, there was very little indication that complete failure around the tension side of the connection was imminent. This indicates that the weld has very little deformation capacity after initial failure. At this stage of the test, there was significant branch deflection and chord face deformation that were likely beyond allowable serviceability conditions. A loud “popping” noise caused by the release of stress in the welded joint was an indication of test completion. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint is 4.11 kip-ft which is 267% stronger than the predicted nominal flexural strength according to ANSI/AISC 360 (2010). Figure 4-6 shows the moment-deflection relationship for T-0.25-34. Initially, the welded joint demonstrates linear-elastic behavior up to the predicted LRFD flexural strength where it begins to gradually decrease in stiffness just before the predicted nominal flexural strength until the ultimate moment is achieved.

The significant over-strength of the welded joint is consistent with theories that smaller welds are stronger than larger welds when compared to their respective predicted nominal strengths. This may be true for welds with consistent deep root penetration which increases the overall strength of the weld due to a larger effective throat size. Smaller welds with the same root penetration as larger welds will have a proportionately larger over-strength effect since the increased effective throat size from the deep penetration is proportionately larger for the smaller weld than the larger weld.

The measured out-of-plane bending moment and torsion acting on the connection were 0.008 and 0.030 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric
moment-strain relationship measured by the upper branch strain gages in the constant stress region (8-in. above the connection), plotted in Appendix H.

Figure 4-7 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.

The strain distributions at the LRFD design and nominal strengths are nearly symmetric about the theoretical neutral axis, SG-5E, which can be expected for connections subject to in-plane bending. Also, the highest measured strains are located at the corners of the branch which is typical for connections between RHS because of the relative flexibility in the connected chord face and the stiffness of the branch corners. The magnitude of strain along the branch transverse faces decreases towards the mid-wall location (SG-1S/1N), but not nearly as much as for test specimens with larger $\beta$-ratios. These regions are less effective than the corners in resisting the applied loads, but they do have a contribution to the overall joint resistance that should not be completely neglected.

At the ultimate moment, the strain distribution becomes less symmetric and the highest strain is recorded on the tension side of the connection (SG-3S). The magnitude of normal strain at the mid-wall locations at the ultimate moment indicates that a large portion of the weld perimeter is effective in resisting the applied loads. Additionally, the branch has yielded at locations SG-3S/2N. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 6.65-in. and the corresponding maximum chord deformation was 0.332-in. vertically at the toe of the horizontal weld leg. Throughout the experiment, the branch remained elastic (negligible curvature) and therefore its rotation was primarily due to the chord face deforming under the applied loads. This is typical for semi-rigid connections with $\beta \leq 0.85$ that undergo chord face plastification at the limit state. The profiles for the branch deflection and chord face deformation are plotted in Appendix I.
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**Figure 4-6** Moment-deflection relationship for T-0.25-34

![Graph showing the moment-deflection relationship for T-0.25-34. The graph plots in-plane bending moment, $M_{ip}$ (kip-ft) against in-plane deflection (inches). The moment at ultimate load, $M_{ultimate}$, is indicated as 4.11 kip-ft. There are dashed lines representing nominal weld strength and LRFD weld strength.](image1)

**Figure 4-7** Distribution of normal strain around the branch perimeter for T-0.25-34

![Graph showing the distribution of normal strain around the branch perimeter. The strain gage designation is provided, along with the strain values for initial unloaded state, LRFD weld strength, nominal weld strength, and actual weld strength.](image2)

**Effective Weld Properties for RHS-to-RHS Moment T-Connections**
4.3.3. Test Specimen T-0.25-23

T-0.25-23 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.010 in/s after the ultimate moment was reached. The welded joint failed very suddenly by weld rupture along a plane through the weld (see macroetch examination in Appendix F.2), similar to T-0.25-34. There was very little indication that complete failure around the tension side of the connection was imminent. Because the chord wall thickness is larger than that of specimen T-0.25-34, there was less branch deflection and chord face deformation, which made predicting the instance of failure even more difficult. A loud “popping” noise caused by the release of stress in the welded joint was an indication of test completion. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint is 4.37 kip-ft which is 133% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). The over-strength effect is not as high as that of specimen T-0.25-34 because the root penetration was not as deep. Figure 4-8 shows the moment-deflection relationship for T-0.25-23. Initially, the welded joint demonstrates linear-elastic behavior up to the predicted LRFD flexural strength where it begins to gradually decrease in stiffness just before the predicted nominal flexural strength until the ultimate moment is achieved.

The measured out-of-plane bending moment and torsion acting on the connection were 0.004 and 0.016 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (8-in. above the connection), plotted in Appendix H.

Figure 4-9 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.
The strain distributions at the LRFD design and nominal strengths are nearly symmetric about the theoretical neutral axis, SG-5E, which can be expected for connections subject to in-plane bending. Also, the highest measured strains are located at the corners of the branch which is typical for connections between RHS because of the relative flexibility in the connected chord face and the stiffness of the branch corners. The magnitude of strain along the branch transverse faces decreases towards the mid-wall location (SG-1S/1N), but not nearly as much as for test specimens with larger β-ratios. These regions are less effective than the corners in resisting the applied loads, but they do have a contribution to the overall joint resistance that should not be completely neglected.

At the ultimate moment, the distribution of normal strain around the branch perimeter becomes less symmetric. The highest strains are at the corners where the branch has yielded at locations SG-3S/3N/2N. The magnitude of normal strain at the mid-wall locations at the ultimate moment indicates that a large portion of the weld perimeter is effective in resisting the applied loads. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 3.31-in. and the corresponding maximum chord deformation was 0.157-in. vertically at the toe of the horizontal weld leg. Throughout the experiment, the branch remained elastic (negligible curvature) and therefore its rotation was primarily due to the chord face deforming under the applied loads. This is typical for semi-rigid connections with $\beta \leq 0.85$ that undergo chord face plastification at the limit state. The profiles for the branch deflection and chord face deformation are plotted in Appendix I.
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Figure 4-8 Moment-deflection relationship for T-0.25-23

Figure 4-9 Distribution of normal strain around the branch perimeter for T-0.25-23
4.3.4. Test Specimen T-0.25-17

T-0.25-17 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.0008 in/s after the ultimate moment was reached. The welded joint failed very suddenly by weld rupture along a plane through the weld (see macroetch examination in Appendix F.2), similar to T-0.25-34 and T-0.25-23. There was very little indication that complete failure around the tension side of the connection was imminent. Because the chord wall thickness is greater than the other two test specimens with $\beta = 0.25$, there was less branch deflection and chord face deformation, which made predicting the instance of failure difficult. A loud “popping” noise caused by the release of stress in the welded joint was an indication of test completion. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint is 4.82 kip-ft which is 100% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). The over-strength effect of the welded joint is not as large as those of T-0.25-34 and T-0.25-23, but it is still significant. Figure 4-10 shows the moment-deflection relationship for T-0.25-17. Initially, the welded joint demonstrates linear-elastic behavior up to the predicted LRFD flexural strength where it begins to gradually decrease in stiffness just before the predicted nominal flexural strength until the ultimate moment is achieved.

The measured out-of-plane bending moment and torsion acting on the connection were 0.002 and 0.007 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (8-in. above the connection), plotted in Appendix H.

Figure 4-11 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.
The strain distributions at the LRFD design and nominal strengths are nearly symmetric about the theoretical neutral axis, SG-5E, which can be expected for connections subject to in-plane bending. Also, the highest measured strains are located around the corners of the branch which is typical for connections between RHS because of the relative flexibility in the connected chord face and the stiffness of the branch corners. The magnitude of strain along the branch transverse faces decreases towards the mid-wall location (SG-1S/1N), but not nearly as much as for test specimens with larger $\beta$-ratios. These regions are less effective than the corners in resisting the applied loads, but they do have a contribution to the overall joint resistance that should not be completely neglected.

At the ultimate moment, the distribution of normal strain around the branch perimeter becomes less symmetric. The highest strains are at the corners where the branch has yielded at locations SG-2S/3S/3N/2N and even at the branch transverse mid-wall location, SG-1N, on the compression side of the connection. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 1.200-in. and the corresponding maximum chord deformation was 0.038-in. vertically at the toe of the horizontal weld leg. Throughout the experiment there was noticeable curvature in the branch, especially at the ultimate moment. This is because the chord wall slenderness ratio is low and hence the chord face is relatively stiff in comparison to the other two specimens with $\beta = 0.25$. The profiles for the branch deflection and chord face deformation are plotted in Appendix I.
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Figure 4-10 Moment-deflection relationship for T-0.25-17

\[ M_{\text{ultimate}} = 4.82 \text{ kip-ft} \]

Figure 4-11 Distribution of normal strain around the branch perimeter for T-0.25-17
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4.3.5. Test Specimen T-0.50-34

T-0.50-34 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.004 in/s after the ultimate moment was reached. The test specimen failed by the chord face rupturing due to punching shear (classified as a connection failure instead of a weld failure) on the tension side of the connection (see macroetch examination in Appendix F.2). Cracks initiated in the chord face at the branch corners, adjacent to the horizontal toe of the weld, and then propagated inwards along the transverse face to the mid-wall location. A continuous crack eventually formed around the tension side of the connection. The failure was gradual and testing was terminated once the cracks reached the branch longitudinal mid-wall locations. At this point, the branch had deflected considerably. Photographs of the failed test specimen are shown in Appendix G. Throughout the experiment the actuator appeared to remain horizontal and the point load device worked as intended. The socket attached to the branch member straddling plates rotated around the round bar of the point load device. Also, no significant out-of-plane branch deflections were observed.

The ultimate moment resistance of the connection was 11.71 kip-ft which is 143% stronger than the predicted nominal connection flexural strength according to the ANSI/AISC 360 (2010). Figure 4-12 shows the moment-deflection relationship for T-0.50-34. Initially, the connection demonstrates linear-elastic behavior but decreases in stiffness before reaching the predicted nominal connection flexural strength (which is also equal to the predicted LRFD design strength, since $\phi = 1.00$) indicating that the chord face has already yielded.

Figure 4-13 shows the distribution of normal strain measured around the branch perimeter at the predicted and actual (ultimate) connection flexural strengths.

The strain distribution at the predicted connection flexural strength is nearly symmetric about the theoretical neutral axis, SG-7E, which can be expected for connections subject to in-plane bending moment. At this point the branch has not yielded and the magnitude of strain along the branch transverse faces decreases slightly from locations.
SG-2S/2N towards the mid-wall locations (SG-1S/1N). This non-uniform distribution of normal strain is typical for connections between RHS because of the relative flexibility in the connected chord face. It indicates that these (transverse mid-wall) regions are less effective in resisting the applied loads but should not be considered completely ineffective (in this case).

At the ultimate moment, the strain distribution becomes less symmetric. The branch has yielded at numerous locations on both sides of the connection including the corners at SG-3S/4S/3N/4N and along the transverse faces at SG-2S/1N/2N. The highest strains were measured on the compression side at SG-2N. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 11.37 inches. Throughout the experiment the branch appeared to remain elastic (negligible curvature) and therefore its rotation was primarily due to the chord face deforming under the applied load. This behavior is typical for semi-rigid connections with $\beta \leq 0.85$ that undergo chord face plastification at the limit state.
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Figure 4-12 Moment-deflection relationship for T-0.50-34

\[ M_{\text{ultimate}} = 11.71 \text{ kip-ft} \]

Branch in-plane deflection (inches)

Figure 4-13 Distribution of normal strain around the branch perimeter for T-0.50-34
4.3.6. Test Specimen T-0.50-23

T-0.50-23 was loaded at an initial test rate of approximately 0.0004 in/s which was increased throughout the experiment to a maximum rate of 0.004 in/s after the ultimate moment was reached. The test specimen failed by weld rupture along a plane through the weld (see macroetch examination in Appendix F.2), initiated by a crack forming at the southwest corner of the welded joint. After loading the test specimen further, a crack also formed on the southeast corner. The experiment was completed once the cracks at the corners propagated towards one another along the transverse face of the welded joint meeting at the mid-wall location. Once a continuous crack formed around the tension side of the connection, a loud “popping” noise caused by the release of stress in the welded joint indicated the test was complete. The failure was gradual, demonstrating the welded joint’s deformation capacity after initial failure. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint was 15.25 kip-ft which is 109% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). Figure 4-14 shows the moment-deflection relationship for T-0.50-23. Initially, the welded joint demonstrates linear-elastic behavior beyond the predicted nominal flexural strength. After that, the stiffness decreases which is possibly due to the chord face yielding under the applied moment.

The measured out-of-plane bending moment and torsion acting on the connection were 0.150 and 0.271 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (12-in. above the connection), plotted in Appendix H.

Figure 4-15 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.
The strain distributions at the LRFD design and nominal flexural strengths are nearly symmetric about the theoretical neutral axis, SG-7E, which can be expected for connections subject to in-plane bending moment. Also, the highest measured strains are located at the corners of the branch which is typical for connections between RHS because of the relative flexibility in the connected chord face and the stiffness of the branch corners. The magnitude of strain along the branch transverse faces decreases towards the mid-wall locations (SG-1S/1N). This indicates that these regions are less effective in resisting the applied loads.

At the ultimate moment, the strain distribution becomes less symmetric and the neutral axis shifts to the left. Additionally, the branch has yielded at numerous locations including the corners (SG-3S/4S/4N/3N) and along the branch transverse face on both sides of the connection (SG-2S/2N/1N). The highest strains were measured on the compression side at SG-2N. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 6.56-in. and the corresponding maximum chord face deformation was 0.396-in. vertically at the toe of the horizontal weld leg. Throughout the experiment, the branch remained elastic (negligible curvature) and therefore its rotation was primarily due to the chord face deforming under the applied loads. This is typical for semi-rigid connections with $\beta \leq 0.85$ that undergo chord face plastification at the limit state with non-slender branch members. The profiles for the branch deflection and chord face deformation are plotted in Appendix I.
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Figure 4-14 Moment-deflection relationship for T-0.50-23

\[ M_{\text{ultimate}} = 15.25 \text{ kip-ft} \]

Figure 4-15 Distribution of normal strain around the branch perimeter for T-0.50-23
4.3.7. Test Specimen T-0.50-17

T-0.50-17 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.004 in/s after the ultimate moment was reached. The test specimen failed by the chord face rupturing due to punching shear (classified as a connection failure instead of a weld failure) on the tension side of the connection (see macroetch examination in Appendix F.2). Cracks initiated in the chord face at the branch corners, adjacent to the horizontal toe of the weld, and then propagated inwards along the transverse face to the mid-wall location. A continuous crack eventually formed around the tension side of the connection. The failure was gradual and testing was terminated once the cracks reached the branch longitudinal mid-wall locations. At this point, the branch had deflected considerably. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the connection was 29.6 kip-ft which is 47.2% stronger than the predicted connection flexural strength according to the ANSI/AISC 360 (2010). Figure 4-16 shows the moment-deflection relationship for T-0.50-17. Initially, the connection demonstrates linear-elastic behavior but loses stiffness before reaching the predicted nominal connection flexural strength (which is also equal to the predicted LRFD design strength, since $\varphi = 1.00$) indicating that the chord face has yielded just prior.

The measured out-of-plane bending moment and torsion acting on the connection were 0.022 and 0.056 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (12-in. above the connection), plotted in Appendix H.

Figure 4-17 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and actual connection flexural strength (ultimate moment).
The strain distribution at the predicted LRFD design flexural strength is nearly symmetric about the theoretical neutral axis, SG-7E, which can be expected for connections subject to in-plane bending. Also, the highest measured strains are located at the corners of the branch which is typical for connections between RHS because of the relative flexibility in the connected chord face and the stiffness of the branch corners. The branch has started to yield around the corner on the compression side of the connection at locations SG-4N/3N. The magnitude of strain along the branch transverse faces decreases from the corner edge (SG-3S/3N) towards the mid-wall locations (SG-1S/1N). This indicates that these regions are less effective in resisting the applied loads, however they should not be considered completely ineffective.

At the ultimate moment, the strain distribution becomes less symmetric and the neutral axis shifts towards the tension side. The branch has yielded at numerous locations including the corners (SG-3S/4S/3N/4N/5N) and along the transverse face (SG-2S/2N) on both sides of the connection. The highest strains were measured on the compression side at SG-3N. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 6.33-in. and the corresponding maximum chord deformation was 0.437-in. vertically at the toe of the horizontal weld leg. Throughout the experiment, the branch remained elastic (negligible curvature) and therefore its rotation was primarily due to the chord face deforming under the applied loads. This is typical for semi-rigid connections with \( \beta \leq 0.85 \) that undergo chord face plastification at the limit state with non-slender branch members. The profiles for the branch deflection and chord face deformation are plotted in Appendix I.
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Figure 4-16 Moment-deflection relationship for T-0.50-17

Figure 4-17 Distribution of normal strain around the branch perimeter for T-0.50-17
4.3.8. Test Specimen T-0.75-34

T-0.75-34 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.002 in/s after the ultimate moment was reached. The test specimen failed by weld rupture along a plane through the weld (see macroetch examination in Appendix F.2), initiated by a crack forming at the southeast corner of the welded joint. After loading the test specimen further, the crack propagated along the branch transverse face over to the southwest corner. The experiment was completed once a continuous crack around the tension side of the connection formed. A loud “popping” noise caused by the release of stress in the welded joint was an indication of test completion. The failure was gradual, demonstrating the welded joint’s deformation capacity after initial failure. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint was 14.37 kip-ft which is 72% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). Figure 4-18 shows the moment-deflection relationship for T-0.75-34. The welded joint demonstrates linear-elastic behavior beyond the predicted LRFD flexural strength but its stiffness begins to decrease slightly before the predicted nominal flexural strength is achieved. This is possibly due to the chord face yielding under the applied moment.

The measured out-of-plane bending moment and torsion acting on the connection were 0.044 and 0.063 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (18-in. above the connection), plotted in Appendix H.

Figure 4-19 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.
The highest measured strains at the LRFD design and nominal strengths are located at the corners of the branch which is typical for connections between RHS because of the relative flexibility in the connected chord face and the stiffness of the branch corners. The magnitude of strain along the branch transverse faces decreases significantly towards the mid-wall location (SG-1S/1N). This indicates that these regions are less effective in resisting the applied loads. The branch has not yielded at any location around its perimeter at this stage of the test.

At the ultimate moment, the strain distribution around the branch perimeter is very erratic. The highest measured strains are at SG-4S which are still lower than the yield strain of the branch RHS material. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 2.46-in. and the corresponding maximum chord deformation was 0.284-in. vertically at the toe of the horizontal weld leg. Throughout the experiment, the branch remained elastic (negligible curvature) and therefore its rotation was primarily due to the chord face deforming under the applied loads. This is typical for semi-rigid connections with $\beta \leq 0.85$ that undergo chord face plastification at the limit state with non-slender branch members. The profiles for the branch deflection and chord face deformation are plotted in Appendix I.
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Figure 4-18 Moment-deflection relationship for T-0.75-34

![Moment-deflection relationship for T-0.75-34](image)

Figure 4-19 Distribution of normal strain around the branch perimeter for T-0.75-34

![Distribution of normal strain around the branch perimeter for T-0.75-34](image)

$M_{\text{ultimate}} = 14.37$ kip-ft

- Initial unloaded state
- LRFD weld strength
- Nominal weld strength
- Actual weld strength
4.3.9. Test Specimen T-0.75-23

T-0.75-23 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.002 in/s after the ultimate moment was reached. The test specimen failed by weld rupture along a plane through the weld (see macroetch examination in Appendix F.2), initiated by cracks forming at the southeast and southwest corners of the welded joint at nearly the same time. The experiment was completed once the cracks at the corners propagated towards one another along the transverse face of the welded joint meeting at the mid-wall location. Once a continuous crack formed around the tension side of the connection, a loud “popping” noise caused by the release of stress in the welded joint indicated the test was complete. The failure was gradual, demonstrating the welded joint’s deformation capacity after initial failure. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint was 27.1 kip-ft which is 105% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). Figure 4-20 shows the moment-deflection relationship for T-0.75-23. The welded joint demonstrates linear-elastic behavior beyond the predicted nominal flexural strength. After that, the stiffness decreases which is possibly due to the chord face yielding under the applied moment.

The measured out-of-plane bending moment and torsion acting on the connection were 0.010 and 0.019 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (18-in. above the connection), plotted in Appendix H.

Figure 4-21 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.
The strain distributions at the LRFD design and nominal strengths are somewhat uniform about the theoretical neutral axis, SG-7E, which can be expected for connections subject to in-plane bending. Also, the highest measured strains are located at the corners of the branch which is typical for connections between RHS because of the relative flexibility in the connected chord face and the stiffness of the branch corners. The magnitude of strain along the branch transverse faces decreases significantly towards the mid-wall location (SG-1S/1N). This indicates that these regions are less effective in resisting the applied loads.

At the ultimate moment, the strain distribution becomes less symmetric and the neutral axis shifts towards the compression side. The branch did not yield at any location around its perimeter throughout the test up to the ultimate moment. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 1.23-in. and the corresponding maximum chord deformation was 0.108-in. vertically at the toe of the horizontal weld leg. Throughout the experiment, the branch remained elastic (negligible curvature) and therefore its rotation was primarily due to the chord face deforming under the applied loads. This is typical for semi-rigid connections with $\beta \leq 0.85$ that undergo chord face plastification at the limit state with non-slender branch members. The profiles for the branch deflection and chord face deformation are plotted in Appendix I.
Figure 4-20 Moment-deflection relationship for T-0.75-23

Figure 4-21 Distribution of normal strain around the branch perimeter for T-0.75-23
4.3.10. Test Specimen T-0.75-17

T-0.75-17 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.002 in/s after the ultimate moment was reached. The test specimen failed by weld rupture along a plane through the weld (see macroetch examination in Appendix F.2), initiated by a crack forming at the southeast corner of the welded joint. After loading the test specimen further, a crack also formed on the southwest corner. The experiment was completed once the cracks at the corners propagated towards one another along the transverse face of the welded joint meeting at the mid-wall location. Once a continuous crack formed around the tension side of the connection, a loud “popping” noise caused by the release of stress in the welded joint indicated the test was complete. The failure was gradual, demonstrating the welded joint’s deformation capacity after initial failure. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint was 52.2 kip-ft which is 141% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). Figure 4-22 shows the moment-deflection relationship for T-0.75-17. Initially, the welded joint demonstrates linear-elastic behavior beyond the predicted nominal flexural strength. After that, the stiffness decreases which is possibly due to the chord face yielding under the applied moment.

The measured out-of-plane bending moment and torsion acting on the connection were 0.020 and 0.021 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane bending moment they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (18-in. above the connection), plotted in Appendix H.

Figure 4-23 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.
The strain distributions at the LRFD design and nominal flexural strengths are nearly symmetric about the theoretical neutral axis, SG-7E, which can be expected for connections subject to in-plane bending. Also, the highest measured strains are located at the corners of the branch which is typical for connections between RHS because of the relative flexibility in the connected chord face and the stiffness of the branch corners. The magnitude of strain along the branch transverse faces decreases significantly towards the mid-wall locations (SG-1S/1N). This indicates that these regions are less effective in resisting the applied loads.

At the ultimate moment, the strain distribution becomes less symmetric and the neutral axis shifts towards the compression side. Additionally, the branch has yielded at numerous locations on the compression side of the connection including the corner (SG-5N/4N) and on the branch transverse face (SG-2N). The highest strains were measured on the compression side at SG-4N. The branch did not yield at any location on the tension side. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

The horizontal branch deflection at the ultimate moment was 1.41-in. and the corresponding maximum chord deformation was 0.118-in. vertically at the toe of the horizontal weld leg. Throughout the experiment, the branch remained elastic (negligible curvature) and therefore its rotation was primarily due to the chord face deforming under the applied loads. This is typical for semi-rigid connections with $\beta \leq 0.85$ that undergo chord face plastification at the limit state with non-slender branch members. The profiles for the branch deflection and chord face deformation are plotted in Appendix I.
Figure 4-22 Moment-deflection relationship for T-0.75-17

Figure 4-23 Distribution of normal strain around the branch perimeter for T-0.75-17
4.3.11. Test Specimen T-1.00-34

T-1.00-34 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.0006 in/s after the ultimate moment was reached. The test specimen failed by weld rupture, initiated by a crack forming at the southwest corner of the PJP flare-bevel-groove weld. After loading the test specimen further, a crack also formed on the southeast corner. The experiment was terminated once the cracks at the corners propagated towards one another along the transverse face of the welded joint and met near the mid-wall location, forming a continuous crack around the tension side of the connection. Unlike the other tests performed on matched connections, failure occurred through the weld (Figure F-16(c) in Appendix F.2), due to inadequate fusion with the chord, as well as through the vertical fusion face with the branch (Figure F-16(d) in Appendix F.2). A loud “popping” noise caused by the release of stress in the welded joint was an indication of test completion. Failure of the welded joint was gradual, demonstrating its deformation capacity after initial failure. Photographs of the failed specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint was 39.7 kip-ft which is 164% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). Figure 4-24 shows the moment-deflection relationship for T-1.00 -34. The welded joint demonstrates linear-elastic behavior beyond the predicted LRFD and nominal flexural strengths and then its stiffness begins to decrease gradually up to the ultimate moment.

The measured out-of-plane bending moment and torsion acting on the connection were 0.095 and 0.133 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (20-in. above the connection), plotted in Appendix H.
Figure 4-25 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.

The strain distributions at the LRFD design and nominal strengths are somewhat symmetric about the theoretical neutral axis, SG-7E, which can be expected for connections subject to in-plane bending. Also, the highest measured strains are located at the corners of the branch which is typical for matched connections between RHS because the branch longitudinal walls are located directly above the webs of the chord. The result is a highly non-uniform distribution of normal strain and stress across the branch transverse faces with most of the load being transferred through the corners. As shown, the magnitude of strain along the branch transverse faces decreases significantly towards the mid-wall locations (SG-1S/1N). This indicates that these regions are much less effective in resisting the applied loads. Wall portions corresponding to SG-1S/1N can be considered to be completely ineffective since the recorded strain is approximately equal to zero.

At the ultimate moment, the strain distribution becomes less symmetric with the highest strains recorded on the tension side of the connection. At this point, the branch has yielded at the corner on the tension side of the connection at SG-3S/4S/5S but has not yielded at any location on the compression side. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

At the ultimate moment the branch deflected horizontally by 0.632-in. This is smaller than the deflections measured in previous tests where $\beta \leq 0.85$, which is expected for matched connections since they are generally stiffer than stepped connections. The corresponding maximum chord sidewall deformation, or out-of-plane “bulging”, was 0.050-in. Throughout the experiment, the branch remained relatively straight, as shown in Appendix I. The chord sidewall deformation profile is also located therein.
**Figure 4-24** Moment-deflection relationship for T-1.00-34

**Figure 4-25** Distribution of normal strain around the branch perimeter for T-1.00-34

\[ M_{\text{ultimate}} = 39.7 \text{ kip-ft} \]
4.3.12. Test Specimen T-1.00-23

T-1.00-23 was loaded at an initial test rate of approximately 0.0004 in/s which was increased throughout the experiment to a maximum rate of 0.0006 in/s after the ultimate moment was reached. The test specimen failed by weld rupture, initiated by a crack forming at the southwest corner of the PJP flare-bevel-groove weld. After loading the test specimen further, a crack also formed on the southeast corner. The experiment was terminated once the cracks at the corners propagated towards one another along the transverse face of the welded joint and met near the mid-wall location, forming a continuous crack around the tension side of the connection. The angle of inclination of the failure plane on the fillet weld was very steep, as shown in the macroetch examination (Appendix F.2), and appears to be along the vertical fusion face with the branch wall. A loud “popping” noise, caused by the release of stress in the welded joint, was an indication of test completion. Failure of the welded joint was gradual, demonstrating its deformation capacity after initial failure. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint was 64.7 kip-ft which is 150% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). Figure 4-26 shows the moment-deflection relationship for T-1.00-23. The welded joint demonstrates linear-elastic behavior beyond the predicted LRFD and nominal flexural strengths and then its stiffness begins to decrease gradually up to the ultimate moment.

The measured out-of-plane bending moment and torsion acting on the connection were 0.039 and 0.034 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. However, the moment-strain relationship measured by the upper branch strain gages in the constant stress region, plotted in Appendix H, is not perfectly symmetric. This is possibly because the upper branch strain gages are only 9.5-in. from the point of application of the load (24-in. above the connection) which may be slightly within the disturbed region. For subsequent tests on matched connections, the strain gages were installed 20-in. above the connection.
Figure 4-27 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.

The strain distributions at the LRFD design and nominal strengths are somewhat symmetric about the theoretical neutral axis, SG-7E, which can be expected for connections subject to in-plane bending. Also, the highest measured strains are located at the corners of the branch which is typical for matched connections between RHS because the branch longitudinal walls are located directly above the webs of the chord. The result is a highly non-uniform distribution of normal strain and stress across the branch transverse faces with most of the load being transferred through the corners. As shown, the magnitude of strain along the branch transverse faces decreases significantly towards the mid-wall locations (SG-1S/1N). This indicates that these regions are less effective in resisting the applied loads. Wall portions corresponding to SG-1S/1N can be considered to be completely ineffective since the recorded strain is approximately equal to zero.

At the ultimate moment, the strain distribution becomes less symmetric with the highest strains recorded on the tension side of the connection. The branch has yielded at the corner locations SG-4S/5S with the highest measurements of strain recorded at SG-5S. The branch did not yield at any location on the compression side of the connection. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

At the ultimate moment the branch deflected horizontally by 0.688-in. This is smaller than the deflections measured in previous tests where $\beta \leq 0.85$, which is expected for matched connections since they are generally stiffer than stepped connections. The corresponding maximum chord sidewall deformation, or out-of-plane “bulging”, was 0.039-in. Throughout the experiment, the branch remained relatively straight, as shown in Appendix I, however large deformations were observed in the connecting chord face on the tension side of the connection. The chord sidewall deformation profile is also located therein.
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Figure 4-26 Moment-deflection relationship for T-1.00-23

\[ M_{\text{ultimate}} = 64.7 \text{ kip-ft} \]

Branch in-plane deflection (inches)

Figure 4-27 Distribution of normal strain around the branch perimeter for T-1.00-23

- Initial unloaded state
- LRFD weld strength
- Nominal weld strength
- Actual weld strength

Strain gage designation

[Diagram showing strain distribution with labels for strain gages and moment.]
4.3.13. **Test Specimen T-1.00-17**

T-1.00-17 was loaded at an initial test rate of approximately 0.0002 in/s which was increased throughout the experiment to a maximum rate of 0.0006 in/s after the ultimate moment was reached. The test specimen failed by weld rupture, initiated by a crack forming at the southwest corner of the PJP flare-bevel-groove weld. After loading the test specimen further, a crack also formed on the southeast corner. The experiment was terminated once the cracks at the corners propagated towards one another along the transverse face of the welded joint and met near the mid-wall location, forming a continuous crack around the tension side of the connection. The angle of inclination of the failure plane on the fillet weld was very steep, as shown in the macroetch examination (Appendix F.2), and appears to be along the vertical fusion face with the branch wall. A loud “popping” noise, caused by the release of stress in the welded joint, was an indication of test completion. Failure of the welded joint was gradual, demonstrating its deformation capacity after initial failure. Photographs of the failed test specimen are shown in Appendix G.

The ultimate moment resistance of the welded joint was 93.6 kip-ft which is 138% stronger than the predicted nominal flexural strength according to the ANSI/AISC 360 (2010). Figure 4-28 shows the moment-deflection relationship for T-1.00 -17. The welded joint demonstrates linear-elastic behavior beyond the predicted LRFD and nominal flexural strengths and then its stiffness begins to decrease gradually up to the ultimate moment.

The measured out-of-plane bending moment and torsion acting on the connection were 0.535 and 0.038 kip-ft, respectively. Since these values are so small in comparison to the ultimate in-plane moment, they can be considered negligible. Thus, the connection was subjected primarily to in-plane bending. This is confirmed by the symmetric moment-strain relationship measured by the upper branch strain gages in the constant stress region (20-in. above the connection), plotted in Appendix H.
Figure 4-29 shows the distribution of normal strain measured around the branch perimeter at the predicted LRFD design and nominal flexural strengths as well as the actual flexural strength (ultimate moment) of the welded joint.

SG-3N did not work properly throughout the experiment and therefore no data is reported for that specific location on the branch. A linear interpolation between SG-4N and SG-2N is used instead.

The strain distributions at the LRFD design and nominal strengths are somewhat symmetric about the theoretical neutral axis, SG-7E, which can be expected for connections subject to pure in-plane bending. Also, the highest measured strains are located at the corners of the branch which is typical for matched connections between RHS because the branch longitudinal walls are located directly above the webs of the chord. The result is a highly non-uniform distribution of normal strain and stress across the branch transverse faces with most of the load being transferred through the corners. As shown, the magnitude of strain along the branch transverse faces decreases significantly towards the mid-wall locations (SG-1S/1N). This indicates that these regions are less effective in resisting the applied loads. Wall portions corresponding to SG-1S/1N can be considered to be completely ineffective since the recorded strain is approximately equal to zero.

At the ultimate moment, the strain distribution becomes less symmetric with the highest strains recorded on the tension side of the connection. The branch has only yielded at the corner on the tension side of the connection at SG-4S. The branch did not yield at any location on the compression side of the connection. Moment-strain relationships for the individual SGs up to the ultimate moment are plotted in Appendix H.

At the ultimate moment the branch deflected horizontally by 0.708-in. This is smaller than the deflections measured in previous tests where $\beta \leq 0.85$, which can be expected for matched connections since they are generally stiffer than stepped connections. The corresponding maximum chord sidewall deformation, or out-of-plane “bulging”, was 0.022-in. Throughout the experiment, the branch remained relatively straight however some curvature is evident at the ultimate moment when examining the branch deflection
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profile, located in Appendix I. The chord sidewall deformation profile is also located therein.

![Moment-deflection relationship for T-1.00-17](image1)

**Figure 4-28 Moment-deflection relationship for T-1.00-17**

![Distribution of normal strain](image2)

**Figure 4-29 Distribution of normal strain around the branch perimeter for T-1.00-17**
Chapter 5: Evaluation of Results

An objective of this experimental program is to verify or adjust the current effective elastic section modulus for in-plane bending defined by Equation 2.28 and postulated in Table K4.1 of ANSI/AISC 360 (2010) for RHS-to-RHS moment T-connections. Chapter 4 presented the actual geometric and material properties measured for the RHS material and as-laid weld metal, as well as the actual flexural strengths (or ultimate moments) observed from the full-scale tests performed on weld-critical test specimens. Since test specimens T-0.50-34 and T-0.50-17 failed by punching shear of the connected chord face (considered a connection failure), they are not included in the analyses performed herein. The data from the successful tests is used to plot correlations between the actual and predicted flexural strengths of the welded joints as well as to calculate the resistance factor by performing a safety index analysis (described in Section 2.3.2.3). Additionally, the analyses are repeated with the application of the \((1.00 + 0.5 \sin^{1.5} \theta)\) factor to fillet welds loaded at an angle of 90° to the longitudinal axis of individual weld elements. Finally, a modification to the effective weld properties postulated in Table K4.1 (AISC, 2010) that assumes more of the weld perimeter is effective in resisting the applied bending moment (for most geometric configurations) is proposed and evaluated using the same criteria.

5.1. Evaluation of Current Effective Weld Properties

An analysis of the current effective elastic section modulus defined by Equation 2.28 and specified in Table K4.1 (AISC, 2010) is performed herein. The actual flexural strengths for each weld-critical connection are summarized in Table 5-1 with the predicted nominal flexural strengths which are calculated using the measured geometric and material properties of the RHS and as-laid welds. The mean of the actual/ predicted strengths as well as the coefficient of variation (COV) are given and used in combination with Equation 2.34 to calculate a resistance factor, \(\varphi\), equal to 1.46 (\(\beta^+ = 4.0\) and \(\alpha = 0.55\) as discussed in Section 2.3.2.3). Since this is much larger than the
resistance factors used for fillet welds and PJP groove welds (0.75 and 0.80, respectively), the current equation for the effective elastic section modulus for in-plane bending postulated in Table K4.1 (AISC, 2010) can be deemed very conservative. The correlations are plotted without the inclusion of the fillet weld directional strength enhancement factor in Figure 5-1.

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Actual flexural strength</th>
<th>Predicted nominal flexural strength</th>
<th>Actual / Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_{\text{ultimate}}$ (kip-ft)</td>
<td>$M_{n-ip}$ (kip-ft)</td>
<td></td>
</tr>
<tr>
<td>T-0.25-34</td>
<td>4.11</td>
<td>1.121</td>
<td>3.67</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>4.37</td>
<td>1.876</td>
<td>2.33</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>4.82</td>
<td>2.41</td>
<td>2.00</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>15.25</td>
<td>7.31</td>
<td>2.09</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>14.37</td>
<td>8.35</td>
<td>1.720</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>27.1</td>
<td>13.23</td>
<td>2.05</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>52.2</td>
<td>21.7</td>
<td>2.41</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>39.7</td>
<td>15.01</td>
<td>2.65</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>64.7</td>
<td>25.8</td>
<td>2.50</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>93.6</td>
<td>39.4</td>
<td>2.38</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td>2.38</td>
</tr>
<tr>
<td>COV</td>
<td></td>
<td></td>
<td>0.223</td>
</tr>
<tr>
<td>$\varnothing$</td>
<td></td>
<td></td>
<td>1.46</td>
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</table>

The predicted nominal flexural strength was re-calculated with the inclusion of the $(1.00 + 0.5 \sin^{1.5} \theta)$ factor (or fillet weld directional strength enhancement factor). It was applied only to the weld elements that consisted of fillet welds. For test specimens with $\beta \leq 0.85$, all four sides are fillet welded and loaded normal ($\theta = 90^\circ$) to the longitudinal axis of the weld and hence the nominal flexural strength was increased by a factor of 1.5. For the matched connections, the fillet weld directional strength enhancement factor only applies to the welds along the transverse walls of the branch because there are PJP flare-bevel-groove welds along the longitudinal walls.
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(a) Actual strength vs. predicted nominal strength
(M_{n-ip})

(b) Actual strength vs. predicted LRFD strength
(\phi M_{n-ip})

Figure 5-1 Correlation with test results for RHS-to-RHS moment T-connections and excluding the 
(1.00 + 0.50 \sin^{1.5}\theta) term

(a) Actual strength vs. predicted nominal strength
(M_{n-ip})

(b) Actual strength vs. predicted LRFD strength
(\phi M_{n-ip})

Figure 5-2 Correlation with test results for RHS-to-RHS moment T-connections and including the 
(1.00 + 0.50 \sin^{1.5}\theta) term
Table 5-2 summarizes the ratio of actual/predicted flexural strengths giving a new mean and COV used to calculate a new resistance factor, $\phi$, equal to 0.96.

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Actual flexural strength $M_{\text{ultimate}}$ (kip-ft)</th>
<th>Predicted nominal flexural strength $M_{n-\text{ip}}$ (kip-ft)</th>
<th>Actual / Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>4.11</td>
<td>1.681</td>
<td>2.45</td>
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<td>T-0.25-23</td>
<td>4.37</td>
<td>2.81</td>
<td>1.553</td>
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<td>T-0.25-17</td>
<td>4.82</td>
<td>3.62</td>
<td>1.331</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>15.25</td>
<td>10.97</td>
<td>1.391</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>14.37</td>
<td>12.53</td>
<td>1.147</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>27.1</td>
<td>19.85</td>
<td>1.364</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>52.2</td>
<td>32.5</td>
<td>1.606</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>39.7</td>
<td>16.79</td>
<td>2.37</td>
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<tr>
<td>T-1.00-23</td>
<td>64.7</td>
<td>31.3</td>
<td>2.07</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>93.6</td>
<td>49.4</td>
<td>1.895</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
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<td>1.72</td>
</tr>
<tr>
<td>COV</td>
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<td></td>
<td>0.264</td>
</tr>
<tr>
<td>$\phi$</td>
<td></td>
<td></td>
<td>0.96</td>
</tr>
</tbody>
</table>

The correlations including the fillet weld directional strength enhancement factor are plotted in Figure 5-2. Since $\phi$ is still larger than the resistance factors for fillet welds and PJP groove welds, it can be applied safely with the current equation for the effective elastic section modulus for in-plane bending postulated in the Table K4.1 (AISC, 2010) for RHS-to-RHS moment T-connections. Although it may be safe for these types of connections between RHS under bending moment, it was shown in Chapter 2 that it is unsafe when applied to other types of axially-loaded connections between RHS. Thus, for consistency it would not be practical to apply the $(1.00 + 0.5 \sin^{1.5} \theta)$ to some types of RHS connections under specific loads and not to others. Instead, an investigation into modifying the current effective weld properties so that more of the weld perimeter is considered as being effective in resisting the applied loads will be performed in the next section.
If the requirements of CSA S16 (2009) are evaluated, an identical result to ANSI/AISC 360 (2010) is obtained. Although they have different resistance factors for fillet welds (equal to 0.67 and 0.75 for CSA S16 and ANSI/AISC 360 (2010), respectively), the equations come out identical as shown.

For CSA S16 (2009):

\[
\phi M_{n-ip} = 0.67 \cdot \varphi \cdot (F_{EXX} \cdot S_{ip}) \\
\phi M_{n-ip} = 0.67 \cdot 0.67 \cdot (F_{EXX} \cdot S_{ip}) \\
\phi M_{n-ip} = 0.45 \cdot (F_{EXX} \cdot S_{ip})
\]

For ANSI/AISC 360 (2010):

\[
\phi M_{n-ip} = 0.75 \cdot (F_{nw} \cdot S_{ip}) \\
\phi M_{n-ip} = 0.75 \cdot (0.60 \cdot F_{EXX} \cdot S_{ip}) \\
\phi M_{n-ip} = 0.45 \cdot (F_{EXX} \cdot S_{ip})
\]

Thus, since \( \phi \) is much larger than 0.67, the equation for the effective elastic section modulus for in-plane bending may be deemed adequately conservative for CSA S16 (2009) too.

The Eurocode, EN 1993-1-8 (CEN, 2005), uses a safety factor \( y_{M2} = 1.25 \) instead of a resistance factor and therefore the inverse \( 1 / 1.25 = 0.80 \) can be compared to the calculated resistance factor. Since \( \phi \) is greater than 0.80, the current equation for the effective elastic section modulus for in-plane bending specified in Table K4.1 (AISC, 2010) can be deemed adequately conservative for that application as well.

### 5.2. Evaluation of Modified Effective Weld Properties

As shown in the previous section, the current equation for the effective elastic section modulus for in-plane bending is excessively conservative. Thus, some modifications to the effective weld properties postulated in Table K4.1 (AISC, 2010) that allow more of the welded joint’s perimeter to be considered effective in resisting the applied moment in RHS-to-RHS T-connections will be made. In Chapter 4 the branch strain distribution plots showed that the transverse welds were effective in resisting the applied loads
beyond the limit of two times the chord wall thickness (2t). Therefore the following requirement:

“When \( \beta > 0.85 \) or \( \Theta > 50^\circ \), \( b_{eol}/2 \) shall not exceed 2t.”

will be modified to:

“When \( \beta > 0.85 \) or \( \Theta > 50^\circ \), \( b_{eol}/2 \) shall not exceed \( B_b/4 \).”

This modification to the requirement limiting the value of \( b_{eol} \) increases the effective length of the transverse welds ultimately leading to an increased predicted flexural strength. An exception is for small RHS branch member sizes, such as HSS 2 x 2 x 1/4”, where this modified requirement may actually decrease the effective length of the transverse welds. Since this size of RHS is not commonly used in large-scale construction, it is not a concern, especially because it results in an even safer design (which was shown in the previous section to be very conservative).

The tables and correlation plots from the previous section have been recalculated with the modified requirement, excluding the \((1.00 + 0.5 \sin^{1.5}\theta)\) factor, and are shown in Table 5-3 and Figure 5-3, respectively. As shown, the modified requirement provides a resistance factor equal to 0.87 which is larger than those for fillet welds and PJP groove welds and hence, the modified requirement can be deemed adequately safe for such connections for ANSI/AISC 360 (2010), CSA S16 (2009) and EN 1993-1-8 (CEN, 2005).

The modified requirement is also applicable to the equations for the effective length (Equation 2.27) and the effective elastic section modulus for out-of-plane bending (Equation 2.29). While there is no available test data on weld-critical connections between RHS loaded by branch out-of-plane bending, the data from weld-critical axially-loaded T- and X- (or Cross-) connection tests performed at the University of Toronto (Cassidy, 1993) can be re-analysed using the modified requirement to investigate whether the modified requirements remain conservative. Performing a safety index analysis on the data gives a mean value of the actual/ predicted strengths equal to 1.114 and a COV equal to 0.1408 for a calculated resistance factor, \( \varnothing \), equal to 0.82. Since this is larger than 0.75 which is required for fillet welds, the modified requirements
to the effective weld properties in Table K4.1 (AISC, 2010) proposed in this section may be deemed adequately conservative for axially-loaded RHS-to-RHS T- and X- (or Cross-) connections. Figure 5-4 shows the correlation with the test results when using the modified requirements.

The correlations including the fillet weld directional strength enhancement factor are plotted in Figure 5-5 and Figure 5-6 for axially-loaded RHS-to-RHS T- and X- (or Cross) connections and 90° RHS-to-RHS moment T-connections, respectively. Since each produces a resistance factor, $\phi$, much less than 0.75, the fillet weld directional strength enhancement factor equal to $(1.00 + 0.5 \sin^{1.5}\theta)$ should not be used for such connections in combination with the modified requirement to the effective weld properties postulated in Table K4.1 (AISC, 2010) which is proposed herein.

Table 5-3 Actual versus the predicted nominal flexural strength using the modified requirement of ANSI/AISC 360 (2010) and excluding the $(1.00 + 0.5 \sin^{1.5}\theta)$ factor

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Actual flexural strength, $M_{\text{ultimate}}$ (kip-ft)</th>
<th>Predicted nominal flexural strength, $M_{n-ip}$ (kip-ft)</th>
<th>Actual / Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>4.11</td>
<td>1.121</td>
<td>3.67</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>4.37</td>
<td>1.599</td>
<td>2.73</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>4.82</td>
<td>1.688</td>
<td>2.86</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>15.25</td>
<td>8.74</td>
<td>1.744</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>14.37</td>
<td>11.59</td>
<td>1.240</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>27.1</td>
<td>19.20</td>
<td>1.410</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>52.2</td>
<td>29.1</td>
<td>1.796</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>39.7</td>
<td>20.4</td>
<td>1.950</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>64.7</td>
<td>42.3</td>
<td>1.530</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>93.6</td>
<td>63.6</td>
<td>1.473</td>
</tr>
<tr>
<td>Mean</td>
<td>2.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>COV</td>
<td>0.386</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\phi$</td>
<td>0.87</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Effective Weld Properties for RHS-to-RHS Moment T-connections

Figure 5-3 Correlation with test results for RHS-to-RHS moment T-connections using the modified requirement of ANSI/AISC 360 (2010) and excluding the \((1.00 + 0.5 \sin^{1.5} \theta)\) factor

Figure 5-4 Correlation with test results for RHS-to-RHS axially-loaded T- and X-connections using the modified requirement of ANSI/AISC 360 (2010) and excluding the \((1.00 + 0.5 \sin^{1.5} \theta)\) factor
Effective Weld Properties for RHS-to-RHS Moment T-Connections

Figure 5-5 Correlation with test results for RHS-to-RHS moment T-connections using the modified requirement of ANSI/AISC 360 (2010) and including the \((1.00 + 0.5 \sin 1.5\theta)\) factor

Figure 5-6 Correlation with test results for RHS-to-RHS axially-loaded T- and X-connections using the modified requirement of ANSI/AISC 360 (2010) and including the \((1.00 + 0.5 \sin 1.5\theta)\) factor
Chapter 6: Conclusions and Recommendations

Based on the results from this experimental program which consisted of 12 full-scale tests on RHS-to-RHS moment T-connections designed to be weld-critical and on the analysis of data from other experimental programs consisting of tests on weld-critical connections between RHS, the following conclusions and recommendations are made:

- Current design methods for determining the effective fillet weld throat size required to develop the yield strength of the connected branch wall (referred to as prequalified weld sizes) do not agree amongst national and international codes, specifications and guidelines.
- The \((1.00 + 0.50 \sin^{1.5} \Theta)\) factor (or fillet weld directional strength enhancement factor) should not be universally applied to all connections with or between RHS, as it may result in an unsafe design. This may be because connections with RHS are inherently eccentrically-loaded (because welding can only be performed on one side of the branch wall) and secondary effects create additional tension at the fillet weld roots which may reduce the overall strength.
- Macroetch examinations of the failed welds showed that the angle of the failure plane through the weld, for stepped connections that are fillet-welded all-around the branch perimeter, is between 0 and 45 degrees to the branch fusion face.
- Macroetch examinations of the failed welds also showed that the failure plane for the transverse fillet weld elements for matched connections is along the branch fusion face (if adequate fusion is achieved).
- The distribution of normal strain around the branch perimeter adjacent to the welded joint in a RHS-to-RHS T-connection subject to branch in-plane bending is highly non-uniform.
- As the \(\beta\) –ratio for RHS-to-RHS moment T-connections decreases, the effective length of the weld element along the transverse walls of the branch increases (and vice-versa).
• The current equation for the effective elastic section modulus for in-plane bending specified in Table K4.1 of ANSI/AISC 360-10 is very conservative and can be considered a lower bound, safe design approach.

• Modifying the requirement that limits the effective width, $b_{eoi}$, in Table K4.1 (AISC, 2010) from:

  “When $\beta > 0.85$ or $\Theta > 50^\circ$, $b_{eoi}/2$ shall not exceed $2t$."

  to, “When $\beta > 0.85$ or $\Theta > 50^\circ$, $b_{eoi}/2$ shall not exceed $B_b/4$”

increases the predicted strength of welded joints in RHS-to-RHS T-, Y- and X- (or Cross-) connections subject to branch axial load or branch bending. Adopting the above modification is still conservative and generally provides a more economical design approach.

• For connections with $0.25 \leq \beta \leq 0.85$, branch rotation was mostly due to the chord face deforming under the applied load. The vertical chord face deformation extends beyond the theoretical rectilinear yield lines (as shown by the deformation profiles in Appendix I.2).

• Full-scale truss tests on weld-critical RHS-to-RHS overlapped K-connections should be performed to verify or adjust the effective weld lengths postulated in Table K4.1 of ANSI/AISC 360 (2010).

• Full-scale connection tests on weld-critical RHS-to-RHS T-connections subject to branch out-of-plane bending should be performed to verify the effective elastic section modulus for out-of-plane bending postulated in Table K4.1 of ANSI/AISC 360 (2010).

• Further investigation into the proportional strength enhancement effect that consistent deep root penetration has on small fillet welds, relative to larger fillet weld sizes, would be beneficial.

• Further investigation into the secondary effects induced at the root of eccentrically-loaded (one-sided) fillet welds and the effect on the strength should be performed.
REFERENCES


APPENDIX A  Sample Calculations and Resistance Tables
### A.1 Sample Calculations

**Stepped Connection, T-0.75-23 (imperial units):**

<table>
<thead>
<tr>
<th>Branch properties:</th>
<th>HSS 6 x 6 x 3/8&quot;</th>
<th>cold-formed ASTM A500 Grade C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside dimensions</td>
<td>Nominal wall thickness</td>
<td>Design wall thickness</td>
</tr>
<tr>
<td>$H_b = B_b = 6$</td>
<td>$t_{b,nom} = 0.375$</td>
<td>$t_b = 0.93t_{b,nom} = 0.349$</td>
</tr>
</tbody>
</table>

$F_{y_b} = 50 \text{ ksi}, F_{u_b} = 62 \text{ ksi}$

$S_b = 13.100 \text{ in}^3, Z_b = 15.800 \text{ in}^3$

**Chord properties: | HSS 8 x 8 x 3/8" | cold-formed ASTM A500 Grade C |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside dimensions</td>
<td>Nominal wall thickness</td>
<td>Design wall thickness</td>
</tr>
<tr>
<td>$H = B = 8$</td>
<td>$t_{nom} = 0.375$</td>
<td>$t = 0.93t_{nom} = 0.349$</td>
</tr>
</tbody>
</table>

$F_y = 50 \text{ ksi}, F_u = 62 \text{ ksi}$

**Check the limits of applicability of Table K3.2, AISC 360-10:**

<table>
<thead>
<tr>
<th>Branch angle</th>
<th>$\theta$</th>
<th>$\geq 90^\circ$</th>
<th>90</th>
<th>ok</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord wall slenderness</td>
<td>$B/t$</td>
<td>$\leq 35$</td>
<td>23</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td>$H/t$</td>
<td></td>
<td>23</td>
<td>ok</td>
</tr>
<tr>
<td>Branch wall slenderness</td>
<td>$B_b/t_b$</td>
<td>$\leq 35, \frac{E}{F_{y_b}} \leq 1.25$</td>
<td>17</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td>$H_b/t_b$</td>
<td></td>
<td>17</td>
<td>ok</td>
</tr>
<tr>
<td>Width ratio</td>
<td>$B_b/B$</td>
<td>$\geq 0.25$</td>
<td>0.75</td>
<td>ok</td>
</tr>
<tr>
<td>Aspect ratio</td>
<td>0.5</td>
<td>$\leq H_b/B_b \leq 2.0$</td>
<td>1.0</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\leq H/B \leq 2.0$</td>
<td>1.0</td>
<td>ok</td>
</tr>
<tr>
<td>Material strength</td>
<td>$F_y$</td>
<td>$\leq 52 \text{ ksi}$</td>
<td>50 ksi</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td>$F_{y_b}$</td>
<td></td>
<td>50 ksi</td>
<td>ok</td>
</tr>
<tr>
<td>Ductility</td>
<td>$F_y/F_u$</td>
<td>$\leq 0.8$</td>
<td>0.74</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td>$F_{y_b}/F_{u_b}$</td>
<td>Gr. C acceptable</td>
<td>0.74</td>
<td>ok</td>
</tr>
</tbody>
</table>

Therefore, Table K3.2 is applicable for calculating the connection flexural strength.
Appendix A.1: Sample Calculations

K3. HSS-to-HSS moment connections, AISC 360-10:

\[ \beta = \frac{B_p}{B} = \frac{6}{8} = 0.75 < 0.85 \]  
Governed by the limit state of chord wall plastification

\[ \therefore M_n = F_y t^2 H_b \left[ \frac{1}{2\eta} + \frac{2}{\sqrt{1 - \beta}} + \frac{\eta}{(1 - \beta)} \right] Q_f \]

\[ \eta = \frac{l_b}{B} = \frac{H_b}{sin90^\circ} = \frac{6}{sin90^\circ} = 0.75 \]

\[ Q_f = 1 \quad \text{for chord (connecting surface) in tension} \]

\[ M_n = 50 \times (0.349)^2 \times 6 \times \left[ \frac{1}{2 \times 0.75} + \frac{2}{\sqrt{1 - 0.75}} + \frac{0.75}{(1 - 0.75)} \right] \times 1.0 \]

\[ \therefore M_n = 280 \text{ kip} \cdot \text{in} = 23.3 \text{ kip} \cdot \text{ft} \]

\[ \therefore \Phi M_n = 1.00 \times 23.3 = 23.3 \text{ kip} \cdot \text{ft} \quad \text{LRFD connection flexural strength} \]

Branch flexural strength, Section F7 AISC 360-10:

\[ \lambda_p = 1.12 \frac{E}{F_y} = 1.12 \times \sqrt{\frac{29,000}{50}} = 27.0 \]

\[ \frac{b_b}{t_b} = \frac{(B_b - 4t_b)}{t_b} = \frac{6 - 4 \times 0.349}{0.349} = 13.19 < \lambda_p \]

Therefore, the flange/ web of the branch are considered to be compact and is subject to the limit state of yielding (plastic moment)

\[ \therefore M_{n-b} = M_{p-b} = F_{yb} Z_b = 50 \times 15.800 = 790 \text{ kip} \cdot \text{in} = 65.8 \text{ kip} \cdot \text{ft} \]

\[ \therefore \Phi M_{n,b} = 0.90 \times 65.8 = 59.2 \text{ kip} \cdot \text{ft} \quad \text{LRFD branch flexural strength} \]

Therefore the maximum flexural strength of test specimen T-0.75-23 is 23.3 kip-ft and is governed by the limit state of chord wall plastification.

Weld-critical design calculations, Section K4 AISC 360-10:

The weld size is determined through trial and error until the nominal weld-to-LRFD connection strength ratio is equal to or less than 1.00. Try a fillet weld size of \[ \frac{1}{4} " \]:

137
Appendix A.1: Sample Calculations

t_w = 0.707 \times 0.250 = 0.1768" 

M_{n-ip} = F_{nw}S_{ip} 

F_{nw} = 0.6 \cdot F_{exx} = 0.6 \times 70 = 42 \text{ ksi} 

b_{eol} = \frac{10}{B/t} \left( \frac{F_yt}{F_yb} \right) B_p = \frac{10}{8/0.349} \times \left( \frac{50 \times 0.349}{50 \times 0.349} \right) \times 6 = 2.618" \leq B_b 

\frac{b_{eol}}{2} = 1.309" > 2t = 0.698" 

\therefore b_{eol} = 1.396" 

S_{ip} = \frac{t_w}{3} \left( \frac{H_b}{\sin \theta} \right)^2 + t_w b_{eol} \left( \frac{H_b}{\sin \theta} \right) = \frac{0.1768}{3} \times \left( \frac{6}{\sin 90^\circ} \right)^2 + 0.1768 \times 1.396 \times \left( \frac{6}{\sin 90^\circ} \right) 

S_{ip} = 3.601 \text{ in}^3 

\therefore M_{n-ip} = 42 \times 3.601 = 151.3 \text{ kip \cdot in} 

\therefore M_{n-ip} = 12.60 \text{ kip \cdot ft} \quad \text{nominal weld flexural strength} 

\frac{M_{n-ip}}{\Phi M_n} = \frac{12.60}{21.48} = 0.59 < 1.0 \quad \therefore \text{a.k.}
Appendix A.1: Sample Calculations

**Matched Connection, T-1.00-34 (imperial units):**

Branch properties: HSS 8 x 8 x \( \frac{1}{4} \)”, cold-formed ASTM A500 Grade C

<table>
<thead>
<tr>
<th>Outside dimensions</th>
<th>Nominal wall thickness</th>
<th>Design wall thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_b = B_b = 8 )</td>
<td>( t_{b,nom} = 0.250 )</td>
<td>( t_b = 0.93t_{b,nom} = 0.2325 )</td>
</tr>
</tbody>
</table>

\( F_{yb} = 50 \) ksi, \( F_{ub} = 62 \) ksi

\( s_b = 17.700 \) in\(^3\), \( Z_b = 20.500 \) in\(^3\)

Chord properties: HSS 8 x 8 x \( \frac{1}{4} \)”, cold-formed ASTM A500 Grade C

<table>
<thead>
<tr>
<th>Outside dimensions</th>
<th>Nominal wall thickness</th>
<th>Design wall thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H = B = 8 )</td>
<td>( t_{nom} = 0.250 )</td>
<td>( t = 0.93t_{nom} = 0.2325 )</td>
</tr>
</tbody>
</table>

\( F_y = 50 \) ksi, \( F_u = 62 \) ksi

Check the limits of applicability of Table K3.2, AISC 360-10:

<table>
<thead>
<tr>
<th>Branch angle</th>
<th>( \theta )</th>
<th>( \geq 90^\circ )</th>
<th>90</th>
<th>ok</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chord wall slenderness</td>
<td>( B/t )</td>
<td>( \leq 35 )</td>
<td>34</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td>( H/t )</td>
<td></td>
<td>34</td>
<td>ok</td>
</tr>
<tr>
<td>Branch wall slenderness</td>
<td>( B_b/t_b )</td>
<td>( \leq 35 )</td>
<td>34</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td>( H_b/t_b )</td>
<td>( \leq 1.25 \frac{E}{F_{yb}} )</td>
<td>34</td>
<td>ok</td>
</tr>
<tr>
<td>Width ratio</td>
<td>( B_b/B )</td>
<td>( \geq 0.25 )</td>
<td>1.00</td>
<td>ok</td>
</tr>
<tr>
<td>Aspect ratio</td>
<td>0.5</td>
<td>( \leq H_b/B_b \leq 2.0 )</td>
<td>1.0</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \leq H/B \leq 2.0 )</td>
<td>1.0</td>
<td>ok</td>
</tr>
<tr>
<td>Material strength</td>
<td>( F_y )</td>
<td>( \leq 52 ) ksi</td>
<td>50 ksi</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td>( F_{yb} )</td>
<td></td>
<td>50 ksi</td>
<td>ok</td>
</tr>
<tr>
<td>Ductility</td>
<td>( F_y/F_u )</td>
<td>( \leq 0.8 )</td>
<td>0.81</td>
<td>ok</td>
</tr>
<tr>
<td></td>
<td>( F_{yb}/F_{ub} )</td>
<td>Grade C is acceptable</td>
<td>0.81</td>
<td>ok</td>
</tr>
</tbody>
</table>

Therefore, Table K3.2 is applicable for calculating the connection flexural strength.
Appendix A.1: Sample Calculations

K3. HSS-to-HSS moment connections, AISC 360-10:

\[ \beta = \frac{B_0}{B} = \frac{8}{8} = 1.00 > 0.85 \]

\[ \therefore M_n \text{ is the lesser of } 0.5 F_y^* t (H_b + 5t)^2 \quad \text{and} \quad F_y B \left[ Z_b - \left(1 - \frac{b_{eol}}{B_b}\right) B_b H_b t_b \right] \]

\[ F_y^* = F_y \quad \text{for T-connections} \]

\[ b_{eol} = \frac{10}{B} \left( \frac{F_y t}{F_{y b}} \right) B_b = \frac{10}{8/0.2325} \times \left( \frac{50 \times 0.2325}{50 \times 0.2325} \right) \times 8 = 2.325 < B_b \]

**Limit State: Sidewall local yielding**

\[ M_n = 0.5 F_y^* t (H_b + 5t)^2 \]

\[ M_n = 0.5 \times 50 \times 0.2325 \times (8 + 5 \times 0.2325)^2 = 488 \text{ kip \cdot in} = 40.7 \text{ kip \cdot ft} \]

\[ \therefore \emptyset M_n = 1.00 \times 40.7 = 40.7 \text{ kip \cdot ft} \]

**Limit State: Local yielding of branch due to uneven load distribution**

\[ M_n = F_{y b} B \left[ Z_b - \left(1 - \frac{b_{eol}}{B_b}\right) B_b H_b t_b \right] \]

\[ M_n = 50 \times \left[ 20.500 - \left(1 - \frac{2.325}{8}\right) \times 8 \times 8 \times 0.2325 \right] = 497 \text{ kip \cdot in} = 41.4 \text{ kip \cdot ft} \]

\[ \therefore \emptyset M_n = 0.95 \times 41.4 = 39.4 \text{ kip \cdot ft} \quad \leftarrow \text{goes\n}

**Branch flexural strength, Section F7 AISC 360-10:**

\[ \lambda_r = 1.40 \sqrt{\frac{E}{F_y}} = 1.40 \times \sqrt{\frac{29000}{50}} = 33.7 \]

\[ \frac{B_b}{t_b} = \frac{(B_b - 4t_b)}{t_b} = \frac{(8 - 4 \times 0.2325)}{0.2325} = 30.4 < \lambda_r \]

\[ \lambda_p < 30.4 < \lambda_r \]

Therefore, the flange/ web of the branch are considered to be noncompact and are subject to the limit states of yielding (plastic moment), flange local buckling, and web local buckling.
Appendix A.1: Sample Calculations

Yielding (plastic moment):

\[ M_{n-b} = M_{p-b} = F_{y b} Z_b = 50 \times 20.500 = 1,025 \text{ kip} \cdot \text{in} = 85.4 \text{ kip} \cdot \text{ft} \]

Flange local buckling:

\[ M_{n-b} = M_{p-b} - (M_{p-b} - F_{y b} S_b) \left( 3.57 \frac{h_b}{t_f} \sqrt{\frac{F_{y b}}{E}} - 4.0 \right) \leq M_{p-b} \]

\[ M_{n-b} = 1,025 - (1,025 - 50 \times 17.7) \left( 3.57 \frac{7.07}{0.2325} \sqrt{\frac{50}{29,000}} - 4.0 \right) = 954 \text{ kip} \cdot \text{in} \leq M_{p-b} \]

\[ M_{n-b} = 79.5 \text{ kip} \cdot \text{ft} \leq M_{p-b} \hspace{1cm} \leftarrow \text{governs} \]

Web local buckling:

\[ M_{n-b} = M_{p-b} - (M_{p-b} - F_{y b} S_{xb}) \left( 0.305 \frac{h_b}{t_w} \sqrt{\frac{F_{y b}}{E}} - 0.738 \right) \leq M_{p-b} \]

\[ M_{n-b} = 1,025 - (1,025 - 50 \times 17.7) \left( 0.305 \frac{7.07}{0.2325} \sqrt{\frac{50}{29,000}} - 0.738 \right) = 1,074 \text{ kip} \cdot \text{in} > M_{p-b} \]

\[ \therefore \phi M_{n,b} = 0.90 \times 79.5 = 71.6 \text{ kip} \cdot \text{ft} \hspace{1cm} \text{LRFD branch flexural strength} \]

Therefore the maximum flexural strength of test specimen T-1.00-34 is 39.4 kip-ft and is governed by the limit state of local yielding of the branch due to uneven load distribution.

Weld-critical design calculations, Section K4 AISC 360-10:

The weld size is determined through trial and error until the nominal weld-to-LRFD connection strength ratio is equal to or less than 1.00. Try a continuous effective weld throat of \( \frac{3}{16} \)" around the branch footprint (fillet welds along the transverse branch face and PJP flare-bevel-groove welds along the longitudinal face of the branch:

\[ t_w = 0.1875" \]

\[ M_{n-\ell p} = F_{nw} S_{lp} \]
Appendix A.1: Sample Calculations

\[ F_{nw} = 0.6 \cdot F_{EXX} = 0.6 \times 70 = 42 \text{ ksi} \]

\[
\begin{align*}
    b_{eol} &= \frac{10}{B/t} \left( \frac{F_t}{F_{yb}t_b} \right) B_B = \frac{10}{8/0.2325} \times \left( \frac{50 \times 0.2325}{50 \times 0.2325} \right) \times 8 = 2.325" \leq B_b \\
    \frac{b_{eol}}{2} &= 1.162" > 2t = 0.465" \\
    \therefore b_{eol} &= 0.930"
\end{align*}
\]

\[
S_{ip} = \frac{t_w}{3} \left( \frac{H_b}{\sin \theta} \right)^2 + t_w b_{eol} \left( \frac{H_b}{\sin \theta} \right) = 0.1875 \times \left( \frac{8}{\sin 90^\circ} \right)^2 + 0.1875 \times 0.930 \times \left( \frac{8}{\sin 90^\circ} \right)
\]

\[ S_{ip} = 4.00 + 1.395 \]

\[ S_{ip} = 5.395 \text{ in}^3 \]

\[ \therefore M_{n-ip} = 42 \times 5.395 = 226.6 \text{ kip} \cdot \text{in} \]

\[ \therefore M_{n-ip} = 18.88 \text{ kip} \cdot \text{ft} \text{ nominal weld flexural strength} \]

\[ \frac{M_{n-ip}}{\phi M_n} = \frac{18.88}{36.21} = 0.52 < 1.0 \quad \therefore \alpha. k. \]

Calculate the LRFD weld strength:

\[ \therefore \phi M_{n-ip} = 42 \times \left[ (0.75 \times 1.395) + (0.80 \times 4.00) \right] = 178.3 \text{ kip} \cdot \text{in} \]

\[ \therefore \phi M_{n-ip} = 14.86 \text{ kip} \cdot \text{ft} \text{ LRFD weld flexural strength} \]
### A.2 Resistance Tables

Table A-1 LRFD connection flexural strength calculations (using specified nominal geometric and material properties)

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Nominal Chord Dimensions (inches)</th>
<th>Nominal Branch Dimensions (inches)</th>
<th>Branch-to-Chord Width Ratio ( \beta )</th>
<th>Chord Wall Slenderness Ratio ( H/t )</th>
<th>Effective Width ( b_{col} )</th>
<th>Connection LRFD Moment Resistance ( \phi M_n )</th>
<th>Connection Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>8</td>
<td>0.250</td>
<td>2</td>
<td>0.25</td>
<td>34</td>
<td>-</td>
<td>2.09</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>8</td>
<td>0.375</td>
<td>2</td>
<td>0.25</td>
<td>23</td>
<td>-</td>
<td>4.71</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>8</td>
<td>0.500</td>
<td>2</td>
<td>0.25</td>
<td>17</td>
<td>-</td>
<td>8.37</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>8</td>
<td>0.250</td>
<td>4</td>
<td>0.50</td>
<td>34</td>
<td>-</td>
<td>4.35</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>8</td>
<td>0.375</td>
<td>4</td>
<td>0.50</td>
<td>23</td>
<td>-</td>
<td>9.79</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>8</td>
<td>0.500</td>
<td>4</td>
<td>0.50</td>
<td>17</td>
<td>-</td>
<td>17.40</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>8</td>
<td>0.250</td>
<td>6</td>
<td>0.75</td>
<td>34</td>
<td>-</td>
<td>10.36</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>8</td>
<td>0.375</td>
<td>6</td>
<td>0.75</td>
<td>23</td>
<td>-</td>
<td>23.3</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>8</td>
<td>0.500</td>
<td>6</td>
<td>0.75</td>
<td>17</td>
<td>-</td>
<td>41.4</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>8</td>
<td>0.250</td>
<td>8</td>
<td>1.00</td>
<td>34</td>
<td>2.325</td>
<td>39.4</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>8</td>
<td>0.375</td>
<td>8</td>
<td>1.00</td>
<td>23</td>
<td>3.488</td>
<td>66.5</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>8</td>
<td>0.500</td>
<td>8</td>
<td>1.00</td>
<td>17</td>
<td>4.650</td>
<td>99.1</td>
</tr>
</tbody>
</table>

\( ^{1} \text{Calculated in accordance with AISC 360-10, Table K3.2} \)

**Material Properties:**

- \( F_y = 50 \text{ ksi (nominal)} \) ASTM A500, Grade C
- \( F_u = 62 \text{ ksi (nominal)} \) - Minimum yield strength and tensile range specified by AWS D1.1/D1.1M:2010, Table 3.1 for ASTM A500 Grade C
- \( t, t_p = 0.93 t_{nom} \) (Design wall thickness for electric-resistance-welded HSS [AISC 360-10, Section B4.2])

**Limit State:**

- CP = Chord wall plastification \( \varphi = 1.00 \) (LRFD)
- CS = Sidewall local yielding (chord side wall failure) \( \varphi = 1.00 \) (LRFD)
- EW = Local yielding of branch due to uneven load distribution (effective width failure) \( \varphi = 0.95 \) (LRFD)
### Table A-2 Limits of applicability of Table A-1

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Grade</th>
<th>Chord Wall Slenderness $H/t \leq 35$</th>
<th>Branch Wall Slenderness $B/B \geq 0.25$</th>
<th>Width Ratio $\rho$</th>
<th>Aspect Ratio $0.5 \leq H_b/B_b \leq 2.0$</th>
<th>Material Strength $F_y \leq 52$ ksi</th>
<th>Ductility $F_y/F_u \leq 0.8$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>34</td>
<td>9</td>
<td>0.25</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>23</td>
<td>9</td>
<td>0.25</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>17</td>
<td>9</td>
<td>0.25</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>34</td>
<td>17</td>
<td>0.50</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>23</td>
<td>17</td>
<td>0.50</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>17</td>
<td>9</td>
<td>0.50</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>34</td>
<td>26</td>
<td>0.75</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>23</td>
<td>17</td>
<td>0.75</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>17</td>
<td>13</td>
<td>0.75</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>34</td>
<td>34</td>
<td>1.00</td>
<td>1</td>
<td>50</td>
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<tr>
<td>T-1.00-23</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>23</td>
<td>23</td>
<td>1.00</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>ASTM A500 Gr. C</td>
<td>90</td>
<td>17</td>
<td>17</td>
<td>1.00</td>
<td>1</td>
<td>50</td>
</tr>
</tbody>
</table>

Notes: ASTM A500 Grade C is acceptable

T-1.00-23 exceeds the branch slenderness requirements, however the connections are weld-critical and branch slenderness is not a concern.
Table A-3 LRFD branch flexural strength calculations (using specified nominal geometric and material properties)

<table>
<thead>
<tr>
<th>Nominal Branch Dimensions (inches)</th>
<th>Width-to-thickness ratio</th>
<th>Section Classification for Local Buckling[^1]</th>
<th>Elastic Section Modulus, $S_b$ (inches $^3$)</th>
<th>Plastic Section Modulus, $Z_b$ (inches $^3$)</th>
<th>Nominal Flexural Strength - Yielding[^1] $M_{p_b} = M_{p-b}$ (kip-ft)</th>
<th>Nominal Flexural Strength - Flange Local Buckling[^1] $M_{p_b}$ (kip-ft)</th>
<th>Nominal Flexural Strength - Web Local Buckling[^1] $M_{p-b}$ (kip-ft)</th>
<th>LRFD Flexural Strength $\phi \ M_{p_b}$ (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_b, H_b$</td>
<td>$t_{b,nom}$</td>
<td>$b / t_b$</td>
<td>compact</td>
<td>0.745</td>
<td>0.964</td>
<td>4.0</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>0.250</td>
<td>4.6</td>
<td>compact</td>
<td>3.900</td>
<td>4.690</td>
<td>19.5</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>0.250</td>
<td>13.2</td>
<td>compact</td>
<td>5.950</td>
<td>7.700</td>
<td>32.1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>4</td>
<td>0.500</td>
<td>4.6</td>
<td>compact</td>
<td>9.540</td>
<td>11.20</td>
<td>46.7</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>0.250</td>
<td>21.8</td>
<td>compact</td>
<td>13.100</td>
<td>15.80</td>
<td>65.8</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>0.375</td>
<td>13.2</td>
<td>compact</td>
<td>16.10</td>
<td>19.80</td>
<td>82.5</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>6</td>
<td>0.500</td>
<td>8.9</td>
<td>compact</td>
<td>17.70</td>
<td>20.50</td>
<td>85.4</td>
<td>79.5</td>
<td>85.4</td>
</tr>
<tr>
<td>8</td>
<td>0.250</td>
<td>30.4</td>
<td>non-compact</td>
<td>24.90</td>
<td>29.40</td>
<td>122.5</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>0.375</td>
<td>18.9</td>
<td>compact</td>
<td>31.20</td>
<td>37.50</td>
<td>156.3</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>8</td>
<td>0.500</td>
<td>13.2</td>
<td>compact</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

[^1] AISC 360-10, Section B4 "Member Properties"
[^1] AISC 360-10, Section F7 "Square and Rectangular HSS and Box-Shaped Members"

\[
E = 29,000 \text{ ksi}
\]
\[
F_y = 50 \text{ ksi}
\]
\[
\phi = 0.00 \text{ (LRFD)}
\]

---

Compression Element Members Subject to Flexure

<table>
<thead>
<tr>
<th>Limiting width-to-thickness ratios</th>
<th>Flanges</th>
<th>Webs</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda_p$</td>
<td>27.0</td>
<td>58.3</td>
</tr>
<tr>
<td>$\lambda_s$</td>
<td>33.7</td>
<td>137.3</td>
</tr>
</tbody>
</table>
### Table A-4 Original weld design for test specimens (including the factor of 0.60 on $F_{EXX}$ and using specified nominal geometric and material properties)

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Weld Type</th>
<th>Leg Size of Fillet Weld</th>
<th>Effective Weld Throat</th>
<th>$b_{col}$</th>
<th>Weld Section Modulus</th>
<th>Nominal Weld Strength</th>
<th>LRFD Weld Strength</th>
<th>LRFD Connection Flexural Strength</th>
<th>Weld : Connection Strength Ratio $M_{n-up} / \phi M_{n}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.75-34</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>0.581</td>
<td>0.331</td>
<td>1.16</td>
<td>0.87</td>
<td>2.09</td>
<td>0.55</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>1.308</td>
<td>0.524</td>
<td>1.83</td>
<td>1.37</td>
<td>3.62</td>
<td>0.51</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>1.000</td>
<td>0.670</td>
<td>2.34</td>
<td>1.76</td>
<td>3.02</td>
<td>0.05</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>0.930</td>
<td>1.200</td>
<td>4.20</td>
<td>3.15</td>
<td>4.35</td>
<td>0.97</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>1.395</td>
<td>1.447</td>
<td>5.06</td>
<td>3.80</td>
<td>9.79</td>
<td>0.52</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>F (all-around)</td>
<td>3/8</td>
<td>0.2652</td>
<td>1.860</td>
<td>3.387</td>
<td>11.85</td>
<td>8.89</td>
<td>17.40</td>
<td>0.68</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>0.930</td>
<td>2.331</td>
<td>8.16</td>
<td>6.12</td>
<td>10.36</td>
<td>0.79</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>F (all-around)</td>
<td>1/4</td>
<td>0.1768</td>
<td>1.395</td>
<td>3.601</td>
<td>12.60</td>
<td>9.45</td>
<td>23.31</td>
<td>0.54</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>F (all-around)</td>
<td>3/8</td>
<td>0.2652</td>
<td>1.860</td>
<td>6.141</td>
<td>21.49</td>
<td>16.12</td>
<td>41.44</td>
<td>0.52</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>F, PJP (transverse, sides)</td>
<td>-</td>
<td>3/16</td>
<td>0.930</td>
<td>5.395</td>
<td>18.88</td>
<td>14.86</td>
<td>39.36</td>
<td>0.48</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>F, PJP (transverse, sides)</td>
<td>-</td>
<td>1/4</td>
<td>1.395</td>
<td>8.123</td>
<td>28.43</td>
<td>22.26</td>
<td>66.54</td>
<td>0.43</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>F, PJP (transverse, sides)</td>
<td>-</td>
<td>3/8</td>
<td>1.860</td>
<td>13.580</td>
<td>47.53</td>
<td>37.05</td>
<td>99.11</td>
<td>0.48</td>
</tr>
</tbody>
</table>

**Note:** Weld sizes comply with AISC 360-10, Table J2 3 "Minimum Effective Throat of PJP Groove Welds" and Table J9 4 "Minimum Size of Fillet Welds"

- $F_{EOV} = 70$ ksi (nominal) FR70-S electrodes
- $\phi = 0.75$ for fillet welds
- $\phi = 0.80$ for partial-joint-penetration groove welds
- $F =$ Fillet weld
- PJP = Partial joint penetration flare-bevel-groove weld
### Table A-5 Final weld design for test specimens (excluding the factor of 0.60 on $F_{EXX}$ and using specified nominal geometric and material properties)

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Weld Type</th>
<th>Leg Size of Fillet Weld</th>
<th>Effective Weld Throat $t_w$</th>
<th>$b_{eq}$</th>
<th>Weld Section Modulus $S_p$</th>
<th>Nominal Weld Strength $M_{n-p}$</th>
<th>LRFD Weld Strength $\varphi M_{n-p}$</th>
<th>LRFD Connection Flexural Strength $\varphi M_n$</th>
<th>Weld : Connection Strength Ratio $M_{n-p} / \varphi M_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>F (all-around)</td>
<td>1/8</td>
<td>0.0884</td>
<td>0.581</td>
<td>0.221</td>
<td>1.29</td>
<td>0.97</td>
<td>2.09</td>
<td>0.62</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>F (all-around)</td>
<td>1/8</td>
<td>0.0884</td>
<td>1.308</td>
<td>0.349</td>
<td>2.04</td>
<td>1.53</td>
<td>3.62</td>
<td>0.56</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>F (all-around)</td>
<td>1/8</td>
<td>0.0884</td>
<td>1.860</td>
<td>0.447</td>
<td>2.61</td>
<td>1.95</td>
<td>3.62</td>
<td>0.72</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>0.930</td>
<td>1.200</td>
<td>7.00</td>
<td>5.25</td>
<td>4.35</td>
<td>1.61</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>1.395</td>
<td>1.447</td>
<td>8.44</td>
<td>6.33</td>
<td>9.79</td>
<td>0.86</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>F (all-around)</td>
<td>3/8</td>
<td>0.2652</td>
<td>1.860</td>
<td>3.387</td>
<td>19.76</td>
<td>14.82</td>
<td>17.40</td>
<td>1.14</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>F (all-around)</td>
<td>1/8</td>
<td>0.0884</td>
<td>0.930</td>
<td>1.554</td>
<td>9.06</td>
<td>6.80</td>
<td>10.36</td>
<td>0.87</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>F (all-around)</td>
<td>3/16</td>
<td>0.1326</td>
<td>1.395</td>
<td>2.701</td>
<td>15.75</td>
<td>11.82</td>
<td>23.31</td>
<td>0.68</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>F (all-around)</td>
<td>1/4</td>
<td>0.1768</td>
<td>1.860</td>
<td>4.094</td>
<td>23.88</td>
<td>17.91</td>
<td>41.44</td>
<td>0.58</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>F, PJP (transverse, sides)</td>
<td>-</td>
<td>1/8</td>
<td>0.930</td>
<td>3.597</td>
<td>20.98</td>
<td>16.51</td>
<td>39.36</td>
<td>0.53</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>F, PJP (transverse, sides)</td>
<td>-</td>
<td>3/16</td>
<td>1.395</td>
<td>6.093</td>
<td>35.54</td>
<td>27.82</td>
<td>66.54</td>
<td>0.53</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>F, PJP (transverse, sides)</td>
<td>-</td>
<td>1/4</td>
<td>1.860</td>
<td>9.053</td>
<td>52.81</td>
<td>41.16</td>
<td>99.11</td>
<td>0.53</td>
</tr>
</tbody>
</table>

**Note:** Weld sizes comply with AISC 360-10, Table J2.3 "Minimum Effective Throat of PJP Groove Welds" and Table J2.4 "Minimum Size of Fillet Welds"
Appendix A.2: Resistance Tables

Table A-6 LRFD connection flexural strength calculations (using the measured geometric and material properties)

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Measured Chord Dimensions (inches)</th>
<th>Measured Branch Dimensions (inches)</th>
<th>Branch-to-Chord Width Ratio $\beta$</th>
<th>Chord Wall Slenderness Ratio $H/t$</th>
<th>Effective Width $b_{eoi}$ (inches)</th>
<th>Nominal connection flexural strength, $M_p$ (kip-ft)</th>
<th>LRFD connection flexural strength, $\varphi M_p$ (kip-ft)</th>
<th>Connection Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>8.02 0.23</td>
<td>2.01 0.23</td>
<td>0.25</td>
<td>35</td>
<td>-</td>
<td>2.31</td>
<td>2.31</td>
<td>CP</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>7.99 0.34</td>
<td>2.01 0.23</td>
<td>0.25</td>
<td>23</td>
<td>-</td>
<td>5.26</td>
<td>5.26</td>
<td>CP</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>8.05 0.46</td>
<td>2.01 0.23</td>
<td>0.25</td>
<td>18</td>
<td>-</td>
<td>9.69</td>
<td>9.69</td>
<td>CP</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>8.02 0.23</td>
<td>4.02 0.23</td>
<td>0.50</td>
<td>35</td>
<td>-</td>
<td>4.81</td>
<td>4.81</td>
<td>CP</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>7.99 0.34</td>
<td>4.02 0.23</td>
<td>0.50</td>
<td>23</td>
<td>-</td>
<td>10.96</td>
<td>10.96</td>
<td>CP</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>8.05 0.46</td>
<td>4.02 0.23</td>
<td>0.50</td>
<td>18</td>
<td>-</td>
<td>20.1</td>
<td>20.1</td>
<td>CP</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>8.02 0.23</td>
<td>6.01 0.23</td>
<td>0.75</td>
<td>35</td>
<td>-</td>
<td>11.39</td>
<td>11.39</td>
<td>CP</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>7.99 0.34</td>
<td>6.00 0.34</td>
<td>0.75</td>
<td>23</td>
<td>-</td>
<td>26.0</td>
<td>26.0</td>
<td>CP</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>8.05 0.46</td>
<td>6.01 0.46</td>
<td>0.75</td>
<td>18</td>
<td>-</td>
<td>47.1</td>
<td>47.1</td>
<td>CP</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>8.02 0.23</td>
<td>8.02 0.23</td>
<td>1.00</td>
<td>35</td>
<td>2.317</td>
<td>45.0</td>
<td>45.0</td>
<td>CS</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>7.99 0.34</td>
<td>7.99 0.34</td>
<td>1.00</td>
<td>23</td>
<td>3.441</td>
<td>77.1</td>
<td>77.1</td>
<td>CS</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>8.05 0.46</td>
<td>8.05 0.46</td>
<td>1.00</td>
<td>18</td>
<td>4.560</td>
<td>121.3</td>
<td>121.3</td>
<td>CS</td>
</tr>
</tbody>
</table>

Limit State:
- CP = Chord wall plastification
- CS = Sidewall local yielding (chord side wall failure)
- EW = Local yielding of branch due to uneven load distribution (effective width failure)

$\varphi = 1.00$ (LRFD)

$\Phi = 0.95$ (LRFD)
### Table A-7 Effective elastic section modulus for individual weld lengths (using the measured geometric and material properties)

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Average Effective Throat of North Weld, $t_{thn}$ (inches)</th>
<th>Effective Elastic Section Modulus of North Weld, $S_{eln}$ (inches$^2$)</th>
<th>Average Effective Throat of South Weld, $t_{ths}$ (inches)</th>
<th>Effective Elastic Section Modulus of South Weld, $S_{els}$ (inches$^2$)</th>
<th>Average Effective Throat of East Weld, $t_{the}$ (inches)</th>
<th>Effective Elastic Section Modulus of East Weld, $S_{ele}$ (inches$^2$)</th>
<th>Average Effective Throat of West Weld, $t_{thw}$ (inches)</th>
<th>Effective Elastic Section Modulus of West Weld, $S_{elw}$ (inches$^2$)</th>
<th>Weld Effective Elastic Section Modulus $S_{ew}$ (inches$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>0.097</td>
<td>0.054</td>
<td>0.119</td>
<td>0.067</td>
<td>0.098</td>
<td>0.066</td>
<td>0.100</td>
<td>0.068</td>
<td>0.254</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>0.091</td>
<td>0.116</td>
<td>0.148</td>
<td>0.106</td>
<td>0.106</td>
<td>0.072</td>
<td>0.072</td>
<td>0.049</td>
<td>0.426</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>0.079</td>
<td>0.145</td>
<td>0.122</td>
<td>0.223</td>
<td>0.124</td>
<td>0.084</td>
<td>0.142</td>
<td>0.096</td>
<td>0.548</td>
</tr>
<tr>
<td>T-0.50-34</td>
<td>0.124</td>
<td>0.230</td>
<td>0.133</td>
<td>0.217</td>
<td>0.144</td>
<td>0.388</td>
<td>0.131</td>
<td>0.362</td>
<td>1.217</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>0.125</td>
<td>0.346</td>
<td>0.131</td>
<td>0.362</td>
<td>0.168</td>
<td>0.452</td>
<td>0.185</td>
<td>0.499</td>
<td>1.660</td>
</tr>
<tr>
<td>T-0.50-17</td>
<td>0.247</td>
<td>0.905</td>
<td>0.271</td>
<td>0.993</td>
<td>0.280</td>
<td>0.754</td>
<td>0.267</td>
<td>0.719</td>
<td>3.370</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>0.109</td>
<td>0.304</td>
<td>0.107</td>
<td>0.299</td>
<td>0.084</td>
<td>0.508</td>
<td>0.131</td>
<td>0.786</td>
<td>1.896</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>0.137</td>
<td>0.565</td>
<td>0.155</td>
<td>0.638</td>
<td>0.162</td>
<td>0.973</td>
<td>0.138</td>
<td>0.828</td>
<td>3.004</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>0.227</td>
<td>1.246</td>
<td>0.246</td>
<td>1.349</td>
<td>0.157</td>
<td>0.941</td>
<td>0.230</td>
<td>1.305</td>
<td>4.921</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>0.126</td>
<td>0.470</td>
<td>0.092</td>
<td>0.340</td>
<td>0.094</td>
<td>1.003</td>
<td>0.149</td>
<td>1.594</td>
<td>3.407</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>0.179</td>
<td>0.983</td>
<td>0.275</td>
<td>1.509</td>
<td>0.198</td>
<td>2.099</td>
<td>0.120</td>
<td>1.277</td>
<td>5.867</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>0.235</td>
<td>1.727</td>
<td>0.384</td>
<td>2.818</td>
<td>0.254</td>
<td>2.741</td>
<td>0.153</td>
<td>1.655</td>
<td>8.942</td>
</tr>
</tbody>
</table>

- $F_{exc}= 88.1$ ksi
- $F = $ Fillet weld
- $PJP = $ Partial joint penetration flare-bevel-groove weld
- $\varphi = 0.75$ for fillet welds
- $\varphi = 0.80$ for partial-joint-penetration groove welds
Table A-8 Actual flexural strength and predicted nominal/ LRFD weld flexural strengths using the measured geometric and material properties

<table>
<thead>
<tr>
<th>Experimental Designation</th>
<th>Predicted Nominal Weld Strength $M_{n-ip}$ (kip-ft)</th>
<th>Predicted LRFD Weld Strength $\varphi M_{n-ip}$ (kip-ft)</th>
<th>Actual Flexural Strength of Weld $M_u$ (kip-ft)</th>
<th>LRFD Connection Moment Resistance $\varphi M_n$ (kip-ft)</th>
<th>Weld : Connection Capacity Ratio $M_{n-ip} / \varphi M_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-0.25-34</td>
<td>1.121</td>
<td>0.84</td>
<td>4.11</td>
<td>2.31</td>
<td>0.48</td>
</tr>
<tr>
<td>T-0.25-23</td>
<td>1.876</td>
<td>1.407</td>
<td>4.37</td>
<td>4.86</td>
<td>0.39</td>
</tr>
<tr>
<td>T-0.25-17</td>
<td>2.41</td>
<td>1.811</td>
<td>4.82</td>
<td>4.86</td>
<td>0.50</td>
</tr>
<tr>
<td>T-0.50-34*</td>
<td>5.36</td>
<td>4.02</td>
<td>11.71</td>
<td>4.81</td>
<td>1.11</td>
</tr>
<tr>
<td>T-0.50-23</td>
<td>7.31</td>
<td>5.48</td>
<td>15.26</td>
<td>10.96</td>
<td>0.67</td>
</tr>
<tr>
<td>T-0.50-17*</td>
<td>14.85</td>
<td>11.14</td>
<td>29.6</td>
<td>20.1</td>
<td>0.74</td>
</tr>
<tr>
<td>T-0.75-34</td>
<td>8.35</td>
<td>6.26</td>
<td>14.37</td>
<td>11.39</td>
<td>0.73</td>
</tr>
<tr>
<td>T-0.75-23</td>
<td>13.23</td>
<td>9.92</td>
<td>27.1</td>
<td>26.0</td>
<td>0.51</td>
</tr>
<tr>
<td>T-0.75-17</td>
<td>21.7</td>
<td>16.26</td>
<td>52.2</td>
<td>47.1</td>
<td>0.46</td>
</tr>
<tr>
<td>T-1.00-34</td>
<td>15.01</td>
<td>11.83</td>
<td>39.7</td>
<td>45.0</td>
<td>0.33</td>
</tr>
<tr>
<td>T-1.00-23</td>
<td>25.8</td>
<td>20.1</td>
<td>64.7</td>
<td>77.1</td>
<td>0.34</td>
</tr>
<tr>
<td>T-1.00-17</td>
<td>39.4</td>
<td>30.5</td>
<td>93.6</td>
<td>121.3</td>
<td>0.32</td>
</tr>
</tbody>
</table>

* Connection failure preceded weld rupture
APPENDIX B  Fabrication Drawings
Figure B-1 Test specimen T-0.25-34 fabrication drawings
Figure B-2 Test specimen T-0.25-23 fabrication drawings
Figure B-3 Test specimen T-0.25-17 fabrication drawings

NOTES:
1. Unless otherwise specified, all unit are in inches
2. All welds to be performed with ER70S-6 solid wire electrodes
3. All RHS material is of ASTM A500 specification, Grade B/C
4. 45 ksi steel plate
5. Machine RHS end normal to longitudinal axis
   * Machine to \( \frac{1}{4} \)" using a hand grinder
Figure B-4 Test specimen T-0.50-34 fabrication drawings
Figure B-5 Test specimen T-0.50-23 fabrication drawings
Figure B-6 Test specimen T-0.50-17 fabrication drawings
Figure B-7 Test specimen T-0.75-34 fabrication drawings

**Bill of Material**

<table>
<thead>
<tr>
<th>No.</th>
<th>Material</th>
<th>Length</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>HSS 6 x 6 x 1/4&quot;</td>
<td>48&quot;</td>
<td>Branch</td>
</tr>
<tr>
<td>1</td>
<td>HSS 8 x 8 x 1/4&quot;</td>
<td>84&quot;</td>
<td>Chord</td>
</tr>
<tr>
<td>1</td>
<td>14 x 14 x 1&quot; Plate</td>
<td>-</td>
<td>End-plate</td>
</tr>
</tbody>
</table>

**Notes:**

1. Unless otherwise specified, all unit are in inches
2. All welds to be performed with ER70S-6 solid wire electrodes
3. All RHS material is of ASTM A500 specification, Grade B/C
4. 45 ksi steel plate
5. Machine RHS end normal to longitudinal axis

* Machine to 1/8" using a hand grinder
**Figure B-8 Test specimen T-0.75-23 fabrication drawings**

**BILL OF MATERIAL**

<table>
<thead>
<tr>
<th>No.</th>
<th>Material</th>
<th>Length</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>HSS 6 x 6 x ( \frac{3}{8} )&quot;</td>
<td>48&quot;</td>
<td>Branch</td>
</tr>
<tr>
<td>1</td>
<td>HSS 8 x 8 x ( \frac{3}{4} )&quot;</td>
<td>84&quot;</td>
<td>Chord</td>
</tr>
<tr>
<td>1</td>
<td>14 x 14 x 1&quot; Plate</td>
<td>-</td>
<td>End-plate</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Unless otherwise specified, all units are in inches
2. All welds to be performed with ER70S-6 solid wire electrodes
3. All RHS material is of ASTM A500 specification, Grade B/C
4. 45 ksi steel plate
5. Machine RHS end normal to longitudinal axis
   * Machine to \( \frac{3}{16} \)" using a hand grinder
Figure B-9 Test specimen T-0.75-17 fabrication drawings
Figure B-10 Test specimen T-1.00-34 fabrication drawings
Figure B-11 Test specimen T-1.00-23 fabrication drawings
Figure B-12 Test specimen T-1.00-17 fabrication drawings
Appendix B: Fabrication Drawings

**RHS Tensile Coupon Dimensions (inches)**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>Gage length = 2.000 ± 0.005</td>
</tr>
<tr>
<td>A</td>
<td>Length of reduced section, min. = 2.250</td>
</tr>
<tr>
<td>W</td>
<td>Width = 0.500 ± 0.010</td>
</tr>
<tr>
<td>R</td>
<td>Radius of fillet, min. = 0.500</td>
</tr>
<tr>
<td>T</td>
<td>Thickness = varies</td>
</tr>
<tr>
<td>B, C, L</td>
<td>Depend on the grip device dimensions</td>
</tr>
</tbody>
</table>

**All-Weld-Metal Tensile Coupon Dimensions (inches)**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>Gage length = 2.000 ± 0.005</td>
</tr>
<tr>
<td>D</td>
<td>Diameter = 0.500 ± 0.010</td>
</tr>
<tr>
<td>r</td>
<td>Radius of fillet, min. = 3/8</td>
</tr>
<tr>
<td>A</td>
<td>Length of reduced section, min. = 2-1/4</td>
</tr>
</tbody>
</table>

---

**Figure B-13 Tensile coupon specimen fabrication drawings**
APPENDIX C  Welding Procedure Specifications
Figure C-1 Welding procedure specification for 3/16" fillet welds in stepped connections
Figure C-2 Welding procedure specification for 1/4" fillet welds in stepped connections
Figure C-3 Welding procedure specification for 3/8” fillet welds in stepped connections
Figure C-4 Welding procedure specification for 3/16” PJP flare-bevel-groove welds in matched connections
Figure C-5 Welding procedure specification for 1/4" PJP flare-bevel-groove welds in matched connections
Figure C-6 Welding procedure specification for 3/8" PJP flare-bevel-groove welds in matched connections
APPENDIX D  Test Setup and Instrumentation
Figure D-1 General test setup assembly in the University of Toronto Structural Testing Facility
Figure D-2 General instrumentation layout for test specimens

- LED targets installed on all test specimens
- LED targets installed on specimens with $\beta \leq 0.85$
- LED targets installed on specimens with $\beta > 0.85$
- LVDT locations
- String-pot LVDT locations

NOTES:

- Height varies
- Spacing varies

~52"
APPENDIX E  RHS Geometric and Mechanical Properties
Appendix E.1: Geometric Properties of RHS

E.1 Geometric Properties of RHS

HSS 2 x 2 x 1/4” Geometric Measurements

**Outside Dimensions**

- $B_{avg} = H_{avg} = 2.014”$

**Wall Thickness**

- $t_{avg} = 0.2266”$

**Outer Corner Radius**

- Average = 0.482”

**Inner Corner Radius**

- Average = 0.248”

Cross-sectional area = 1.516 in²
HSS 4 x 4 x \( \frac{1}{4}'' \) Geometric Measurements

**Outside Dimensions**

\[ 4.022 \quad 4.026 \quad 4.019 \]
\[ 4.010 \quad 4.016 \quad 4.011 \]

\[ B_{\text{avg}} = H_{\text{avg}} = 4.017'' \]

**Wall Thickness**

\[ 0.2266 \]
\[ 0.2285 \quad 0.2190 \]

\[ t_{\text{avg}} = 0.2254'' \]

**Outer Corner Radius**

\[ 0.490 \quad 0.480 \]
\[ 0.540 \quad 0.460 \]

Average = 0.492"

**Inner Corner Radius**

\[ 0.300 \quad 0.280 \]
\[ 0.300 \quad 0.250 \]

Average = 0.282"

Cross-sectional area = 3.344 in\(^2\)
Appendix E.1: Geometric Properties of RHS

HSS 4 x 4 x 1/2" Geometric Measurements

Outside Dimensions

Wall Thickness

$B_{avg} = H_{avg} = 4.022"$

$t_{avg} = 0.4576"$

Outer Corner Radius

Inner Corner Radius

Average = 0.945"

Average = 0.476"

Cross-sectional area = 6.108 in$^2$
Appendix E.1: Geometric Properties of RHS

HSS 6 x 6 x $\frac{1}{4}$" Geometric Measurements

**Outside Dimensions**

- $B_{avg} = H_{avg} = 6.008"$

**Wall Thickness**

- $t_{avg} = 0.2259"$

**Outer Corner Radius**

- Average = 0.509"

**Inner Corner Radius**

- Average = 0.298"

Cross-sectional area = 5.128 in$^2$
Appendix E.1: Geometric Properties of RHS

HSS 6 x 6 x 3/8” Geometric Measurements

**Outside Dimensions**

```
6.025  5.998  6.003
5.968  6.017  5.991
```

- \( B_{\text{avg}} = H_{\text{avg}} = 6.000'' \)

**Wall Thickness**

```
0.3416
0.3448
0.3391
```

- \( t_{\text{avg}} = 0.3422'' \)

**Outer Corner Radius**

```
0.760  0.810
0.750  0.770
```

- Average = 0.772”

**Inner Corner Radius**

```
0.410  0.450
0.375  0.430
```

- Average = 0.416”

Cross-sectional area = 7.483 in²
Appendix E.1: Geometric Properties of RHS

HSS $6 \times 6 \times \frac{1}{2}''$ Geometric Measurements

**Outside Dimensions**

$$B_{avg} = H_{avg} = 6.006''$$

**Wall Thickness**

$$t_{avg} = 0.4594''$$

**Outer Corner Radius**

Average = 1.155''

**Inner Corner Radius**

Average = 0.671''

Cross-sectional area = 9.669 in$^2$
Appendix E.1: Geometric Properties of RHS

HSS 8 x 8 x \( \frac{1}{4} \) " Geometric Measurements

**Outside Dimensions**

8.013 8.047 8.012

8.021

8.012

8.000

**Wall Thickness**

0.2299

0.2367

0.2327

0.2273

**Outer Corner Radius**

0.710 0.625

0.630 0.590

**Inner Corner Radius**

0.450 0.390

0.380 0.370

Average = 0.639"

Average = 0.398"

Cross-sectional area = 7.065 in²
Appendix E.1: Geometric Properties of RHS

HSS 8 x 8 x \(3/8\) " Geometric Measurements

**Outside Dimensions**

- \(B_{avg} = H_{avg} = 7.986\)"

**Wall Thickness**

- \(t_{avg} = 0.3441\)"

**Outer Corner Radius**

- \(0.900 \quad 1.055\)
- \(0.910 \quad 0.890\)

- Average = 0.939"

**Inner Corner Radius**

- \(0.590 \quad 0.600\)
- \(0.560 \quad 0.600\)

- Average = 0.588"

Cross-sectional area = 10.144 in\(^2\)
Appendix E.1: Geometric Properties of RHS

HSS 8 x 8 x \( \frac{1}{2} \) " Geometric Measurements

**Outside Dimensions**

\[ B_{\text{avg}} = H_{\text{avg}} = 8.052" \]

**Wall Thickness**

\[ t_{\text{avg}} = 0.4560" \]

**Outer Corner Radius**

Average = 1.356"

**Inner Corner Radius**

Average = 0.875"

Cross-sectional area = 13.138 in\(^2\)
## Appendix E.2: RHS Tensile Coupon Test Data Forms

### E.2 RHS Tensile Coupon Test Data Forms

**UNIVERSITY OF TORONTO**

**DEPARTMENT OF CIVIL ENGINEERING**

**STRUCTURES LABORATORY**

**TENSILE COUPON TEST DATA**

**PROJECT:** Effective weld lengths of RHS-to-RHS moment T-connections

**OPERATOR:** Xiaoming Sun

**DATE:** Feb 13, 2012

**SUPERVISOR:** J.A. Packer

**TIME:** 01:35 pm – 02:00 pm

<table>
<thead>
<tr>
<th>SPECIMEN NO</th>
<th>2 x 3 x 1/4&quot; (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GAUGE LENGTH</td>
<td>1.8625 in</td>
</tr>
<tr>
<td>&quot;AFTER RUPTURE&quot;</td>
<td>2.4181 in</td>
</tr>
<tr>
<td>MACHINED WIDTH</td>
<td>0.50952 in</td>
</tr>
<tr>
<td>AVERAGE/MIN</td>
<td>0.5617 in</td>
</tr>
<tr>
<td>THICKNESS</td>
<td>0.23318 in</td>
</tr>
<tr>
<td>AVERAGE/MIN</td>
<td>0.23449 in</td>
</tr>
<tr>
<td>SECTION AREA</td>
<td>0.4484 in²</td>
</tr>
</tbody>
</table>

**LENGTH L:** 18.203 in

**REMARKS:**

**EQUIPMENT:** MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

**CONTROL:** RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate

**SUPPLEMENTARY DATA:** Relax at 0.40% & 0.44% Strain for 2 minutes
Appendix E.2: RHS Tensile Coupon Test Data Forms

UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY
TENSILE COUPTON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 12, 2013

SUPERVISOR: J.A. Packer
TIME: 02:10 pm - 02:35 pm

SPECIMEN NO: 3 x 3 x 7/4" (ii)
GAUGE LENGTH: 1.9805 in
"AFTER RUPTURE: 2 3/35 in
MACHINED WIDTH: 0.58666 in
0.51610 in
0.51050 in
AVERAGE/MIN: 0.51615 in
THICKNESS: 0.33165 in
0.33160 in
0.33166 in
0.33160 in
AVERAGE/MIN: 0.33160 in
LENGTH L: 18 7/16 in
SECTION AREA: 0.12574 in²
WIDTH W: 0.33155 & 1.9640 in

TEST DATA:
YIELD LOAD T_Y:
STATIC YIELD LOAD T_st:
ULTIMATE LOAD T_u:
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS σ_y:
STATIC YIELD STRESS σ_st:
ULTIMATE STRESS σ_u:

EQUIPMENT: MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL:
RANGE 1: STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
RANGE 2: STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.42% & 0.44% strain for 2 minutes.
## Appendix E.2: RHS Tensile Coupon Test Data Forms

### Tensile Coupon Test Data

**PROJECT:** Effective weld lengths of RHS-to-RHS moment T-connections

**OPERATOR:** Xiaoming Sun

**SUPERVISOR:** J.A. Packer

**DATE:** Feb 13, 2013

**TIME:** 2:40 pm - 3:05 pm

### Specimen Details

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<thead>
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<tbody>
<tr>
<td>Gauge Length</td>
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<tr>
<td>&quot;After Rupture&quot;</td>
<td>0.395 in</td>
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<tr>
<td>Machined Width</td>
<td>0.500 in</td>
</tr>
<tr>
<td>Thickness</td>
<td>0.5096 in</td>
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<tr>
<td>Average/Min</td>
<td>0.2359 in</td>
</tr>
<tr>
<td>Test Data</td>
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</tr>
<tr>
<td>Yield Load $T_y$</td>
<td>$\sigma_y$</td>
</tr>
<tr>
<td>Static Yield Load $T_{st}$</td>
<td>$\sigma_{st}$</td>
</tr>
<tr>
<td>Ultimate Load $T_u$</td>
<td></td>
</tr>
<tr>
<td>Max. Strain at Rupture</td>
<td>$\epsilon_{ru}$</td>
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<tr>
<td>Young's Modulus</td>
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<tr>
<td>Strain Hardening Modulus</td>
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<tr>
<td>Yield Stress $\sigma_y$</td>
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<tr>
<td>Static Yield Stress $\sigma_{st}$</td>
<td></td>
</tr>
<tr>
<td>Ultimate Stress $\sigma_u$</td>
<td></td>
</tr>
</tbody>
</table>

### Equipment

- MTS 1000 kN test machine
- MTS clip gauge
- Mitutoyo digital micrometer
- Mitutoyo absolute digimatic calipers

### Control

- Range 1: Strain/Stroke/Load Rate: 0.00015 in/s to yield, 0.0004 in/s to ultimate
- Range 2: Strain/Stroke/Load Rate: 0.0006 in/s to rupture

### Supplementary Data

- Relax at 0.4% & 0.4% strain for 2 minutes
Appendix E.2: RHS Tensile Coupon Test Data Forms

UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
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TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 14, 2012

SUPERVISOR: J.A. Packer
TIME: 11:20 am - 12:00 pm

SPECIMEN NO: 4 x 4 x 1/4" (a)
GAGE LENGTH: 1.8685 in
" AFTER RUPTURE: 2.4695 in
MACHINED WIDTH: 0.54630 in
0.50625 in
0.50630 in

AVERAGE/MIN:
THICKNESS:
0.5660 in
0.5665 in
0.5695 in
0.5670 in
0.5682 in
0.5684 in
0.5695 in
0.5690 in

AVERAGE/MIN:
SECTION AREA:
GRIP P: 7 7/8 in x 1 3/4 in
LENGTH L: 73.504 in
GRIP P: 7 7/8 in x 1 3/4 in
LENGTH L: 73.504 in
W: 1.0615 in

TEST DATA:
YIELD LOAD Ty:
STATIC YIELD LOAD Ty:
ULTIMATE LOAD Tu:
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS σ_y:
STATIC YIELD STRESS σ_y:
ULTIMATE STRESS σ_u:

EQUIPMENT: MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL:
1. STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2. STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.427 % strain for 2 minutes.

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STRUCTURES LABORATORY
TENSILE COUPON TEST DATA

PROJECT : Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR : Xiaoming Sun
DATE: Feb 14, 2012

SUPERVISOR: J.A. Packer
TIME: 01:30 pm - 02:05 pm

SPECIMEN NO : 4x4xY要用 (*)(1)
GAUGE LENGTH : 1.9685 in
" AFTER RUPTURE: 0.508 in
MACHINED WIDTH : 0.50814 in
0.50827 in
0.50866 in
AVERAGE/MIN : 0.50814 in
THICKNESS : 0.025 in
0.028 in
0.02811 in

LENGTH L : 18.036 in
GRIP P : 3.175 in
GRIP T : 3.165 in
SECTION AREA : 0.11625 in
WIDTH W : 1.0335 in

TEST DATA :
YIELD LOAD Ty :
STATIC YIELD LOAD Tst1 :
ULTIMATE LOAD Tu :
MAX. STRAIN AT RUPTURE :
YOUNG'S MODULUS :
STRAIN HARDENING MODULUS:
YIELD STRESS σy :
STATIC YIELD STRESS σst :
ULTIMATE STRESS σu :

EQUIPMENT : MTS 1.000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL : RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
STRAIN/STROKE/LOAD RATE: 0.0008 in/s to rupture
SUPPLEMENTARY DATA: Relax at 0.42% & 0.45% strain for 2 minutes.
Appendix E.2: RHS Tensile Coupon Test Data Forms

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TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 14, 2012

SUPERVISOR: J.A. Packer
TIME: 02:10 pm - 02:40 pm

SPECIMEN NO: 4 x 4 x 3/4" (in)
GAUGE LENGTH: 1.9875 in
" AFTER RUPTURE: 2.5165 in
MACHINED WIDTH: 0.92386 in
0.50846 in
0.30833 in
AVERAGE/MIN: 0.50138 in
THICKNESS: 0.22132 in
0.22132 in
0.24714 in
AVERAGE/MIN: 0.22146 in
LENGTH L: 19.616 in
SECTION AREA: 0.51546 in

TEST DATA:
YIELD LOAD T_y:
STATIC YIELD LOAD T_st:
ULTIMATE LOAD T_u:
MAX. STRAIN AT RUPTURE: με
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS σ_y:
STATIC YIELD STRESS σ_st:
ULTIMATE STRESS σ_u:

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
RANGE 2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.42% & 0.45% strain for 2 minutes

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Appendix E.2: RHS Tensile Coupon Test Data Forms

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TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
SUPERVISOR: J.A. Packer

DATE: Feb 08, 2012
TIME: 08:10am - 08:45am

SPECIMEN NO.: 4 x 4 x 3/4" (L)
GAUGE LENGTH: 0.060" in
" AFTER RUPTURE: 3.4146" in
MACHINED WIDTH: 0.5089 in
AVERAGE/MIN: 0.5089 in
THICKNESS: 6.45935 in
AVERAGE/MIN: 6.45935 in
SECTION AREA: 0.36664 in²
LENGTH L: 13.234 in

REMARKS:

TEST DATA:
YIELD LOAD T_y:
STATIC YIELD LOAD T_y:
ULTIMATE LOAD T_u:
MAX. STRAIN AT RUPTURE: 
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS σ_y:
STATIC YIELD STRESS σ_y:
ULTIMATE STRESS σ_u:

EQUIPMENT: MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: relax at 0.42% and 0.46% strain, held for 2 minutes

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Appendix E.2: RHS Tensile Coupon Test Data Forms

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DEPARTMENT OF CIVIL ENGINEERING
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TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
SUPERVISOR: J.A. Packer

DATE: Feb 07, 2012
TIME: 4:00 pm - 4:30 pm

SPECIMEN NO: 4 x 4 x 7/8" (h)
GAUGE LENGTH: 1,5074 in
" AFTER RUPTURE: 2,4925 in
MACHINED WIDTH: 0.50613 in
0.50613 in
0.50835 in
AVERAGE/MIN: 0.50637 in
THICKNESS: 0.4550 in
0.4550 in
0.4550 in
LENGTH L: 18.875 in
GRIP P: 1.0386 & 1.0402 in
SECTION AREA: 0.93204 in²
W: 3.68 in

TEST DATA:
YIELD LOAD Tₜₚ:
STATIC YIELD LOAD Tₛₚ:
ULTIMATE LOAD Tₜ:
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS σₚ:
STATIC YIELD STRESS σₛₚ:
ULTIMATE STRESS σₜ:

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relaxed at 0.40% & 0.46% strain for 2 minutes
Appendix E.2: RHS Tensile Coupon Test Data Forms

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TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun

SUPERVISOR: J.A. Packer

DATE: Feb 06, 2019

TIME: 10:50 am - 11:20 am

SPECIMEN NO: 4 x 4 x 7/8" (U)

GAUGE LENGTH: 1.85 in

" AFTER RUPTURE: 0.463" in

MACHINED WIDTH: 0.6043 in

AVERAGE/MIN: 0.5054 in

THICKNESS: 0.4600 in

AVERAGE/MIN: 0.4605 in

SECTION AREA: 0.33087 in²

TEST DATA:

YIELD LOAD T_y:

STATIC YIELD LOAD T_st:

ULTIMATE LOAD T_u:

MAX. STRAIN AT RUPTURE:

YOUNG'S MODULUS:

STRAIN HARDENING MODULUS:

YIELD STRESS T_y:

STATIC YIELD STRESS T_st:

ULTIMATE STRESS T_u:

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate

RANGE 2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA:

Relax at 0.42", 0.44" strain for 2 minutes

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DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY

TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 14, 2012

SUPERVISOR: J.A. Packer
TIME: 09:20 am - 10:00 am

SPECIMEN NO: 0x6x 1/4 (L)
GAUGE LENGTH: 1.9656 in

"AFTER RUPTURE: 2.6451 in
MACHINED WIDTH: 0.5566 in
0.5088 in
AVERAGE/MIN: 0.5033
THICKNESS: 0.2291
0.2291
0.2286 in
LENGTH L: 13.936 in
AVERAGE/MIN: 0.2286 in
0.2286 in
SECTION AREA: 0.1168 in²
WIDTH W: 1.054 in 1.0246 in

TEST DATA:
YIELD LOAD Ty:
STATIC YIELD LOAD Tst:
ULTIMATE LOAD Tu:
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS σy:
STATIC YIELD STRESS σst:
ULTIMATE STRESS σu:

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.42% ε 0.45% strain for 2 minutes.
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TENSILE COUpron TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun

SUPERVISOR: J.A. Packer
TIME: 3:50 pm - 04:30 pm

SPECIMEN NO: Gx6x1¼" (1)
GAUGE LENGTH: 1.9856 in
"AFTER RUPTURE: 2.6861 in
MACHINED WIDTH: 0.2682 in
0.2682 in
0.2682 in
AVERAGE/MIN: 0.2682 in
THICKNESS: 0.2280 in
0.2280 in
0.2280 in
LENGTH L: 15.976 in
REMARKS:
AVERAGE/MIN: 0.2280 in
SECTION AREA: 0.7464 in²
WIDTH W: 1.8472 in

TEST DATA:
YIELD LOAD $T_y$:
STATIC YIELD LOAD $T_{st}$:
ULTIMATE LOAD $T_u$:
MAX. STRAIN AT RUPTURE $\varepsilon_r$:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS $\sigma_y$:
STATIC YIELD STRESS $\sigma_{st}$:
ULTIMATE STRESS $\sigma_u$:

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1: STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
RANGE 2: STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.42% and 0.45% strain for 2 minutes

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Appendix E.2: RHS Tensile Coupon Test Data Forms

UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY
TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 14, 2012
SUPERVISOR: J.A. Packer
TIME: 10:35 am - 11:10 am

SPECIMEN NO: 6 x 6 x 1/4" (in)
GAUGE LENGTH: 1.0625 in
" AFTER RUPTURE: 2.60 in
MACHINED WIDTH: 0.5875 in
0.5919 in
0.5856 in
AVERAGE/MIN: 0.5919 in
THICKNESS: 0.2234 in
0.2274 in
0.2234 in
LENGTH L: 19 in
AVERAGE/MIN: 0.2234 in
SECTION AREA: 0.4619 in²
WIDTH W: 1.0545 in
3.0455 in

TEST DATA:
YIELD LOAD Ty:
STATIC YIELD LOAD Tst:
ULTIMATE LOAD Tu:
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
YIELD STRESS 0:
STATIC YIELD STRESS 0st:
ULTIMATE STRESS 0u:

EQUIPMENT: MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL:
1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.4% @ 0.45% for 2 minutes
UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY
TENSILE COUPON TEST DATA

<table>
<thead>
<tr>
<th>PROJECT</th>
<th>Effective weld lengths of RHS-to-RHS moment T-connections</th>
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<tbody>
<tr>
<td>OPERATOR</td>
<td>Xiaoming Guan</td>
</tr>
<tr>
<td>SUPERVISOR</td>
<td>J.A. Packer</td>
</tr>
<tr>
<td>DATE</td>
<td>Feb 10, 2010</td>
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<tr>
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<table>
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<th>6 x 6 x 3/8&quot; (1)</th>
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</thead>
<tbody>
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<td>GAUGE LENGTH</td>
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<tr>
<td>&quot;AFTER RUPTURE&quot;</td>
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<tr>
<td>MACHINED WIDTH</td>
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<tr>
<td>SECTION AREA</td>
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<tr>
<td>LENGTH L</td>
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<tr>
<td>WIDTH W</td>
<td>0.5954 in</td>
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<td>REMARKS:</td>
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<table>
<thead>
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<tr>
<td>YIELD LOAD T_y</td>
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<tr>
<td>STATIC YIELD LOAD T_y</td>
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<td>ULTIMATE LOAD T_u</td>
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<td>MAX. STRAIN AT RUPTURE :</td>
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<td>YOUNG'S MODULUS</td>
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<td>STRAIN HARDENING MODULUS:</td>
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<td>YIELD STRESS T_y</td>
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<tr>
<td>STATIC YIELD STRESS T_y</td>
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<tr>
<td>ULTIMATE STRESS T_u</td>
<td></td>
</tr>
</tbody>
</table>

| EQUIPMENT | MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers |

<table>
<thead>
<tr>
<th>CONTROL</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>RANGE 1 STRAIN/STROKE/LOAD RATE:</td>
<td>0.00015 in/s to yield, 0.00004 in/s to ultimate</td>
</tr>
<tr>
<td>STRAIN/STROKE/LOAD RATE:</td>
<td>0.0006 in/s to rupture</td>
</tr>
</tbody>
</table>

| SUPPLEMENTARY DATA | Relax at 0.42% to 0.43% strain for 2 minutes |

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### Appendix E.2: RHS Tensile Coupon Test Data Forms

**UNIVERSITY OF TORONTO**

**DEPARTMENT OF CIVIL ENGINEERING**

**STRUCTURES LABORATORY**

**TENSILE COUPLON TEST DATA**

**PROJECT:** Effective weld lengths of RHS-to-RHS moment T-connections

**WORKER:** Xiaoming Sun

**SUPERVISOR:** J.A. Packer

**DATE:** Feb 4, 00:19

**TIME:** 02:05 pm - 02:45 pm

---

**SPECIMEN NO:** 6x6x3/8” (ii)

**GAUGE LENGTH:** 1.9686 in

"AFTER RUPTURE:** 2.6331 in

**MACHINED WIDTH:**

- AVERAGE/MIN: 0.6028 in
- AVERAGE: 0.6678 in
- MIN: 0.5978 in

**THICKNESS:**

- AVERAGE/MIN: 0.3302 in
- AVERAGE: 0.3392 in
- MIN: 0.2925 in

**LENGTH L:** 19.096 in

**REMARKS:**

**SECTION AREA:** 0.13756 in²

**WIDTH W:** 1.9682 in

---

**TEST DATA**

- **YIELD LOAD** $T_y$
- **STATIC YIELD LOAD** $T_{st}$
- **ULTIMATE LOAD** $T_u$
- **MAX. STRAIN AT RUPTURE** $\varepsilon_{rupture}$
- **YOUNG'S MODULUS**
- **STRAIN HARDENING MODULUS**
- **YIELD STRESS** $\sigma_y$
- **STATIC YIELD STRESS** $\sigma_{st}$
- **ULTIMATE STRESS** $\sigma_u$

---

**EQUIPMENT:** MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

**CONTROL:**

- **RANGE 1** STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
- **RANGE 2** STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

**SUPPLEMENTARY DATA:**

- Relax at 0.40% & 0.43% strain for 2 minutes
UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY

TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
SUPERVISOR: J.A. Packer
DATE: Feb. 10, 2019
TIME: 11:30am - 12:00pm

SPECIMEN NO: Gx6 x 3/8 (A)

GAUGE LENGTH: 1.963 in
"AFTER RUPTURE: 0.467 in
MACHINED WIDTH: 0.50646 in
AVERAGE/MIN: 0.5062 in
THICKNESS: 0.34043 in
AVERAGE/MIN: 0.34035 in
SECTION AREA: 0.77236 in²

TEST DATA:
YIELD LOAD T_y:
STATIC YIELD LOAD T_st:
ULTIMATE LOAD T_u:
MAX. STRAIN AT RUPTURE: 0.019
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS σ_y:
STATIC YIELD STRESS σ_st:
ULTIMATE STRESS σ_u:

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
RANGE 2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Delay at 0.4 in, 0.43 strain for 2 minutes.

REMARKS:
LENGTH L: 19.063 in
**UNIVERSITY OF TORONTO**

**DEPARTMENT OF CIVIL ENGINEERING**

**STRUCTURES LABORATORY**

**TENSILE COUPON TEST DATA**

**PROJECT**: Effective weld lengths of RHS-to-RHS moment T-connections

<table>
<thead>
<tr>
<th>OPERATOR</th>
<th>Xuejiao Sun</th>
<th>DATE: 19-12-2012</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUPERVISOR</td>
<td>J.A. Packer</td>
<td>TIME: 01:30 pm - 2:00 pm</td>
</tr>
</tbody>
</table>

**SPECIMEN NO**: 6x6x1/2" (L)

<table>
<thead>
<tr>
<th>GAUGE LENGTH</th>
<th>1.9665 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>0.35 in</td>
</tr>
<tr>
<td>Thickness</td>
<td>0.0643 in</td>
</tr>
<tr>
<td>MAchine Width</td>
<td>0.5365 in</td>
</tr>
<tr>
<td>AVERAGE/IN</td>
<td>0.5643 in</td>
</tr>
<tr>
<td>Thickness</td>
<td>0.0643 in</td>
</tr>
</tbody>
</table>

**GAGE LENGTH**: 1.9665 in

**TEST DATA**

<table>
<thead>
<tr>
<th>YIELD LOAD T_y</th>
<th>STATIC YIELD LOAD T_st</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULTIMATE LOAD T_u</td>
<td></td>
</tr>
<tr>
<td>MAX STRAIN AT RUPTURE</td>
<td></td>
</tr>
<tr>
<td>YOUNG'S MODULUS</td>
<td></td>
</tr>
<tr>
<td>STRAIN HARDENING MODULUS</td>
<td></td>
</tr>
<tr>
<td>YIELD STRESS ( \sigma_y )</td>
<td></td>
</tr>
<tr>
<td>STATIC YIELD STRESS ( \sigma_{st} )</td>
<td></td>
</tr>
<tr>
<td>ULTIMATE STRESS ( \sigma_u )</td>
<td></td>
</tr>
</tbody>
</table>

**EQUIPMENT**: MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digital calipers

**CONTROL**: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate

2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

**SUPPLEMENTARY DATA**: Relax at 6.407, 0.437 strain for 2 minutes.
Appendix E.2: RHS Tensile Coupon Test Data Forms

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
SUPervisOR: J.A. Packer
DATE: Feb. 03, 2019
TIME: 2:10 pm - 3:45 pm

SPECIMEN NO: 6 x 6 x 1 1/2" (ii)
GAUGE LENGTH: 1.50 ± .05 in
"AFTER RUPTURE: 2.66 ± .07 in
MACHINED WIDTH: 0.89 ± 0.02 in
0.85 ± 0.02 in
0.80 ± 0.02 in
AVERAGE/MIN: 0.80 ± 0.02 in
THICKNESS: 0.16 ± 0.02 in
0.15 ± 0.02 in
0.14 ± 0.02 in
AVERAGE/MIN: 0.15 ± 0.02 in
SECTION AREA: 0.297 in²

TEST DATA:
YIELD LOAD $T_y$: 
STATIC YIELD LOAD $T_{sy}$:
ULTIMATE LOAD $T_u$: 
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS $\sigma_y$: 
STATIC YIELD STRESS $\sigma_{st}$:
ULTIMATE STRESS $\sigma_u$: 

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
RANGE 2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Preheated at 0.46°C, 0.46°C strain for 2 minutes
Appendix E.2: RHS Tensile Coupon Test Data Forms

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STRUCTURES LABORATORY
TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 07, 2012

SUPERVISOR: J.A. Packer
TIME: 3:00 PM - 3:30 PM

SPECIMEN NO: G x G x 1/2 (in)
GAUGE LENGTH: 1.9685 in
"AFTER RUPTURE: 0.653 in
MACHINED WIDTH: 0.384 in
0.00344 in
0.50046 in
AVERAGE/MIN: 0.50631 in
THICKNESS: 0.45616 in
0.45866 in
0.45499 in
LENGTH L: 18 in
REMARKS:
AVERAGE/MIN: 0.45864 in
GRIP P: 3.7486 ± 3.639 in
SECTION AREA: 0.23066 in²
WIDTH W: 1.063 ± 1.0208 in

TEST DATA:
YIELD LOAD T_y:
STATIC YIELD LOAD T_st:
ULTIMATE LOAD T_u:
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS σ_y:
STATIC YIELD STRESS σ_st:
ULTIMATE STRESS σ_u:

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL:
1. RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2. RANGE 2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relaxed at 0.40% & 0.46% strain for 2 minutes.
### Appendix E.2: RHS Tensile Coupon Test Data Forms

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**DEPARTMENT OF CIVIL ENGINEERING**

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**TENSILE COTON TEST DATA**

**PROJECT:** Effective weld lengths of RHS-to-RHS moment T-connections

**OPERATOR:** Xiaoming Sun

**DATE:** Feb 19, 2012

**SUPERVISOR:** J.A. Packer

**TIME:** 11:25 am - 12:00 pm

<table>
<thead>
<tr>
<th>SPECIMEN NO:</th>
<th>8 x 8 x 4' (L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GAUGE LENGTH</td>
<td>1.9685 in</td>
</tr>
<tr>
<td>&quot; AFTER RUPTURE</td>
<td>2.5859 in</td>
</tr>
<tr>
<td>MACHINED WIDTH</td>
<td>0.50298 in</td>
</tr>
<tr>
<td>0.36792 in</td>
<td></td>
</tr>
<tr>
<td>0.36399 in</td>
<td></td>
</tr>
<tr>
<td>AVERAGE/MIN:</td>
<td>0.505646 in</td>
</tr>
<tr>
<td>THICKNESS:</td>
<td>0.25627 in</td>
</tr>
<tr>
<td>0.23316 in</td>
<td></td>
</tr>
<tr>
<td>0.23101 in</td>
<td></td>
</tr>
<tr>
<td>LENGTH:</td>
<td>18.031 in</td>
</tr>
<tr>
<td>REMARKS:</td>
<td></td>
</tr>
<tr>
<td>AVERAGE/MIN:</td>
<td>0.23571 in</td>
</tr>
<tr>
<td>GRIP:</td>
<td>7.3165 in</td>
</tr>
<tr>
<td>7.3165 in</td>
<td></td>
</tr>
<tr>
<td>SECTION AREA:</td>
<td>0.11860 in</td>
</tr>
<tr>
<td>WIDTH:</td>
<td>1.0359 in</td>
</tr>
<tr>
<td>1.0359 in</td>
<td></td>
</tr>
</tbody>
</table>

**TEST DATA**

<table>
<thead>
<tr>
<th>YIELD LOAD $T_y$:</th>
</tr>
</thead>
<tbody>
<tr>
<td>STATIC YIELD LOAD $T_{st}$:</td>
</tr>
<tr>
<td>ULTIMATE LOAD $T_u$:</td>
</tr>
<tr>
<td>MAX. STRAIN AT RUPTURE:</td>
</tr>
<tr>
<td>YOUNG'S MODULUS:</td>
</tr>
<tr>
<td>STRAIN HARDENING MODULUS:</td>
</tr>
<tr>
<td>YIELD STRESS $\sigma_y$:</td>
</tr>
<tr>
<td>STATIC YIELD STRESS $\sigma_{st}$:</td>
</tr>
<tr>
<td>ULTIMATE STRESS $\sigma_u$:</td>
</tr>
</tbody>
</table>

**EQUIPMENT:**
- MTS 1,000 kN test machine
- MTS clip gauge
- Minutoyo digital micrometer
- Minutoyo absolute digimatic calipers

**CONTROL:**
- RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
- RANGE 2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

**SUPPLEMENTARY DATA:**
- Relax at 0.40%, & 0.43% strain for 3 minutes

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DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY

TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 12, 2012

SUPERVISOR: J.A. Packer
TIME: 09:30am - 09:55 am

SPECIMEN NO : B x B x 1/16(1)
GAUGE LENGTH : 4.9675 in
" AFTER RUPTURE : 2.6480 in
MACHINED WIDTH : 0.56065 in
0.50645 in
AVERAGE/MIN : 0.50173 in
THICKNESS : 0.23250 in
0.23250 in
AVERAGE/MIN : 0.23282 in
SECTION AREA : 0.1820 in

TEST DATA:
YIELD LOAD \( T_y \):
STATIC YIELD LOAD \( T_{st} \):
ULTIMATE LOAD \( T_u \):
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS \( \sigma_y \):
STATIC YIELD STRESS \( \sigma_{st} \):
ULTIMATE STRESS \( \sigma_u \):

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL:
1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.40\% & 0.43\% strain for 2 minutes

LENGTH L : 15.125/16 in
GRIP P : 7/16, 7/8 in
WIDTH W : 1.0250, 1.0455 in

REMARKS:

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### Appendix E.2: RHS Tensile Coupon Test Data Forms

**UNIVERSITY OF TORONTO**  
**DEPARTMENT OF CIVIL ENGINEERING**  
**STRUCTURES LABORATORY**

**TENSILE COUPON TEST DATA**

**PROJECT:** Effective weld lengths of RHS-to-RHS moment T-connections

<table>
<thead>
<tr>
<th>OPERATOR</th>
<th>Xiaoming Sun</th>
<th>DATE: Feb 12, 2010</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUPERVISOR</td>
<td>J.A. Packer</td>
<td>TIME: 10:30 am - 11:00 am</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SPECIMEN NO</th>
<th>8 x 6 x 1/4&quot; (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GAUGE LENGTH</td>
<td>3.9685 in</td>
</tr>
<tr>
<td>&quot; AFTER RUPTURE</td>
<td>2.3542 in</td>
</tr>
<tr>
<td>MACHINED WIDTH</td>
<td>0.30285 in</td>
</tr>
<tr>
<td>AVERAGE/MIN</td>
<td>0.25804 in</td>
</tr>
<tr>
<td>THICKNESS</td>
<td>0.23228 in</td>
</tr>
<tr>
<td>AVERAGE/MIN</td>
<td>0.23224 in</td>
</tr>
<tr>
<td>SECTION AREA</td>
<td>0.089 in</td>
</tr>
</tbody>
</table>

**TEST DATA**

<table>
<thead>
<tr>
<th>YIELD LOAD $T_y$</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>STATIC YIELD LOAD $T_{st1}$</td>
<td></td>
</tr>
<tr>
<td>ULTIMATE LOAD $T_u$</td>
<td></td>
</tr>
<tr>
<td>MAX. STRAIN AT RUPTURE</td>
<td></td>
</tr>
<tr>
<td>YOUNG'S MODULUS</td>
<td></td>
</tr>
<tr>
<td>STRAIN HARDENING MODULUS</td>
<td></td>
</tr>
<tr>
<td>YIELD STRESS $\sigma_y$</td>
<td></td>
</tr>
<tr>
<td>STATIC YIELD STRESS $\sigma_{st1}$</td>
<td></td>
</tr>
<tr>
<td>ULTIMATE STRESS $\sigma_u$</td>
<td></td>
</tr>
</tbody>
</table>

**EQUIPMENT**

MTS 1,000 kN test machine, MTG clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

**CONTROL**

- RANGE 1: STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
- RANGE 2: STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

**SUPPLEMENTARY DATA:** Relax at 0.10% and 0.13% strain for 2 minutes.
Appendix E.2: RHS Tensile Coupon Test Data Forms

UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY
TENSILE COUPON TEST DATA

PROJECT : Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR : Xiaoming Sun
DATE: Feb 10, 2012
SUPERVISOR: J.A. Packer
TIME: 10:50 am - 11:25 am

SPECIMEN NO : 2 x 8 x 2/8 in. (G)
GAGE LENGTH : 1.5625 in
"AFTER RUPTURE : 2.5512 in
MACHINED WIDTH : 0.5039 in
0.5015 in
0.5012 in
AVERAGE/MIN : 0.5034 in
THICKNESS : 0.3449 in
0.3459 in
0.3459 in
AVERAGE/MIN : 0.3448 in
LENGTH L : 19.419 in
SECTION AREA : 0.4156 in^2
WIDTH W : 1.0634 in

TEST DATA :

YIELD LOAD T_y :
STATIC YIELD LOAD T_{st1} :
ULTIMATE LOAD T_u :
MAX. STRAIN AT RUPTURE : 
YOUNGS MODULUS :
STRAIN HARDENING MODULUS :
YIELD STRESS \sigma_y :
STATIC YIELD STRESS \sigma_{st1} :
ULTIMATE STRESS \sigma_u : 

EQUIPMENT : MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.40 \% & 0.43 \% strain for 2 minutes.
Appendix E.2: RHS Tensile Coupon Test Data Forms

---

**UNIVERSITY OF TORONTO**  
**DEPARTMENT OF CIVIL ENGINEERING**  
**STRUCTURES LABORATORY**  
**TENSILE COUPON TEST DATA**

**PROJECT:** Effective weld lengths of RHS-to-RHS moment T-connections  
**OPERATOR:** Xiaoming Sun  
**DATE:** Feb 10, 2019  
**SUPERVISOR:** J.A. Packer  
**TIME:** 09:16 a.m. - 09:40 a.m.

**SPECIMEN NO:** 3 x 8 x 2  
**GAUGE LENGTH:** 136.14 in  
**"AFTER RUPTURE:"** 2.65  
**MACHINED WIDTH:** 0.0565  
**AVERAGE/MIN:** 0.0565  
**THICKNESS:** 344.42 in  
**AVERAGE/MIN:** 344.42 in  
**SECTION AREA:** 0.137  4 in

**TEST DATA:**  
**YIELD LOAD T_y:**  
**STATIC YIELD LOAD T_st:**  
**ULTIMATE LOAD T_u:**  
**MAX. STRAIN AT RUPTURE:**  
**YOUNG'S MODULUS:**  
**STRAIN HARDENING MODULUS:**  
**YIELD STRESS \sigma_y:**  
**STATIC YIELD STRESS \sigma_{st}:**  
**ULTIMATE STRESS \sigma_u:**

**EQUIPMENT:** MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

**CONTROL:**  
1. STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate  
2. STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

**SUPPLEMENTARY DATA:** Relax at 0.417 \& 0.457, strain for 2 minutes.
Appendix E.2: RHS Tensile Coupon Test Data Forms

UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY

TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 10, 2012
SUPERVISOR: J.A. Packer
TIME: 10:00 am - 10:25 am

SPECIMEN NO.: 8 x 8 x 1/8" (Cu)
GAUGE LENGTH: 1.528 in
"AFTER RUPTURE": 1.528 in
MACHINED WIDTH: 0.5038 in
0.5108 in
0.5038 in
0.5098 in

THICKNESS: 0.3413 in
0.3427 in
0.3411 in
0.3431 in
0.3435 in

LENGTH L: 19.437 in
GRIP P: 1.937 in
Grip: 1.937 in
SECTION AREA: 0.1763 in^2
WIDTH W: 1.024 in
1.025 in

TEST DATA:
YIELD LOAD T_y:
STATIC YIELD LOAD T_y:
ULTIMATE LOAD T_u:
MAX. STRAIN AT RUPTURE:
YOUNGS MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS $\sigma_y$:
STATIC YIELD STRESS $\sigma_{st}$:
ULTIMATE STRESS $\sigma_u$:

EQUIPMENT: MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate
2 STRAIN/STROKE/LOAD RATE: 0.0005 in/s to rupture

SUPPLEMENTARY DATA: Relax at 0.48 % e 0.48 % strain for 2 minutes.
### Appendix E.2: RHS Tensile Coupon Test Data Forms

#### UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY
TENSILE COUPON TEST DATA

**PROJECT:** Effective weld lengths of RHS-to-RHS moment T-connections

**OPERATOR:** Xianning Sun  
**DATE:** Feb 07, 2012

**SUPERVISOR:** J.A. Packer  
**TIME:** 11:30 am - 12:15 pm

<table>
<thead>
<tr>
<th>SPECIMEN NO:</th>
<th>$8 \times 8 \times \frac{3}{4}$' (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GAUGE LENGTH</td>
<td>1.9669 in</td>
</tr>
<tr>
<td>AFTER RUPTURE</td>
<td>0.6413 in</td>
</tr>
<tr>
<td>MACHINED WIDTH</td>
<td>0.50939 in</td>
</tr>
<tr>
<td>AVERAGE/MIN</td>
<td>0.51024 in</td>
</tr>
<tr>
<td>THICKNESS</td>
<td>0.4533 in</td>
</tr>
<tr>
<td>AVERAGE/MIN</td>
<td>0.4533 in</td>
</tr>
<tr>
<td>SECTION AREA</td>
<td>0.24406 in</td>
</tr>
</tbody>
</table>

#### GAUGE LENGTH C

<table>
<thead>
<tr>
<th>LENGTH L</th>
<th>10 in</th>
</tr>
</thead>
</table>

#### WIDTH W

<table>
<thead>
<tr>
<th>GRIP P</th>
<th>1.7480 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRIP P'</td>
<td>1.6850 in</td>
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</table>

#### REMARKS:

- MTS 1000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers
- **CONTROL:**  
  - RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.0004 in/s to ultimate  
  - RANGE 2 STRAIN/STROKE/LOAD RATE: 0.0006 in/s to rupture

#### SUPPLEMENTARY DATA:
- Relaxed at 0.40% ± 0.46% strain for 2 minutes
UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY
TENSILE COUPON TEST DATA

PROJECT: Effective weld lengths of RHS-to-RHS moment T-connections

OPERATOR: Xiaoming Sun
DATE: Feb 08, 2012

SUPERVISOR: J.A. Packer
TIME: 11:25 am - 12:00 pm

SPECIMEN NO.: a x 8 x \( \frac{1}{2} \) (w)
GAUGE LENGTH: 1.9535 in

"AFTER RUPTURE: 0.6516 in
MACHINED WIDTH: 0.5043 in
0.5065 in
0.5063 in
AVERAGE/MIN: 0.5063 in
THICKNESS: 0.4535 in
0.4580 in
0.4533 in
AVERAGE/MIN: 0.4580 in
SECTION AREA: 0.3314 in²

TEST DATA:
YIELD LOAD \( T_y \):
STATIC YIELD LOAD \( T_{st1} \):
ULTIMATE LOAD \( T_u \):
MAX. STRAIN AT RUPTURE:
YOUNG'S MODULUS:
STRAIN HARDENING MODULUS:
YIELD STRESS \( \sigma_y \):
STATIC YIELD STRESS \( \sigma_{st} \):
ULTIMATE STRESS \( \sigma_u \):

EQUIPMENT: MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

CONTROL: RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00015 in/s to yield, 0.00014 in/s to ultimate
2 STRAIN/STROKE/LOAD RATE: 0.00016 in/s to rupture

SUPPLEMENTARY DATA: Relocated at 0.40°, 0.43°, strain for 2 minutes

GRIP P
MACHINED WIDTH B
GRIP P
THICKNESS T
LENGTH L = 12.016 in
GRIP P = 3.6890 in

REMARKS:
Appendix E.2: RHS Tensile Coupon Test Data Forms

**Tensile Coupon Test Data**

- **Project:** Effective weld lengths of RHS-to-RHS moment T-connections
- **Operator:** Xiaoming Sun
- **Supervisor:** J.A. Packer
- **Date:** Feb 08, 2012
- **Time:** 2:15 pm to 2:55 pm

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>( 8 \times 8 \times \frac{1}{4}'' \text{ in} )</th>
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<tbody>
<tr>
<td>Gauge Length</td>
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<tr>
<td>After rupture</td>
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<tr>
<td>Machined Width</td>
<td>0.5082 in ( \pm 0.0005 \text{ in} )</td>
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<tr>
<td>Average/Min.</td>
<td>0.50512 in</td>
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<tr>
<td>Thickness</td>
<td>0.45623 in ( \pm 0.0005 \text{ in} )</td>
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<tr>
<td>Average/Min.</td>
<td>0.45662 in</td>
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<tr>
<td>Length</td>
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</tr>
<tr>
<td>Section Area</td>
<td>0.33457 in</td>
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</table>

**Test Data**

- **Yield Load** \( T_y \)
- **Static Yield Load** \( T_{st} \)
- **Ultimate Load** \( T_u \)
- **Max. Strain at Rupture** \( \nu_c \)
- **Youngs Modulus**
- **Strain Hardening Modulus**
- **Yield Stress** \( \sigma_y \)
- **Static Yield Stress** \( \sigma_{st} \)
- **Ultimate Stress** \( \sigma_u \)

**Equipment:** MTS 1,000 kN test machine, MTS clip gauge, Mitutoyo digital micrometer, Mitutoyo absolute digimatic calipers

**Control:**
- **Range 1:** Strain/Stoke/Load Rate: 0.00015 in/s to yield, 0.0004 in/s to ultimate
- **Range 2:** Strain/Stoke/Load Rate: 0.0006 in/s to rupture

**Supplementary Data:** Relax at 0.47% and 0.46% strain for 2 minutes.
Appendix E.3: RHS Mechanical Properties

E.3 RHS Mechanical Properties

**Tensile Coupon Results for HSS 2 x 2 x 1/4"**

- $F_{y, \text{avg}} = 59.3 \text{ ksi}$
- $\varepsilon_{y, \text{avg}} = 2267 \, (\mu\varepsilon)$
- $F_{u, \text{avg}} = 67.5 \text{ ksi}$
- $\varepsilon_{u, \text{avg}} = 212867 \, (\mu\varepsilon)$

**Coupon [iii]**

**Coupon [ii]**

**Coupon [i]**

**Tensile Coupon Results for HSS 2 x 2 x 1/4"**

- $F_{y, [i]} = 59.6 \text{ ksi}$
- $\varepsilon_{y, [i]} = 2265 \, (\mu\varepsilon)$
- $F_{u, [i]} = 66.2 \text{ ksi}$
- $\varepsilon_{u, [i]} = 228400 \, (\mu\varepsilon)$

**Coupon [i]**

**0.2% Offset**
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 2 x 2 x $\frac{1}{4}$

$F_y,\{\text{ii}\} = 59.4$ ksi
$\varepsilon_y,\{\text{ii}\} = 2866 (\mu\varepsilon)$
$F_u,\{\text{ii}\} = 67.8$ ksi
$\varepsilon_u,\{\text{ii}\} = 193400 (\mu\varepsilon)$

Tensile Coupon Results for HSS 2 x 2 x $\frac{1}{4}$

$F_y,\{\text{iii}\} = 58.9$ ksi
$\varepsilon_y,\{\text{iii}\} = 1874 (\mu\varepsilon)$
$F_u,\{\text{iii}\} = 68.4$ ksi
$\varepsilon_u,\{\text{iii}\} = 216800 (\mu\varepsilon)$
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 4 x 4 x 1/4"

- $F_{y, \text{avg}} = 62.1$ ksi
- $\varepsilon_{y, \text{avg}} = 1994$ (µε)
- $F_{u, \text{avg}} = 76.2$ ksi
- $\varepsilon_{u, \text{avg}} = 273467$ (µε)

Tensile Coupon Results for HSS 4 x 4 x 1/4"

- $F_{y, [i]} = 64.1$ ksi
- $\varepsilon_{y, [i]} = 1966$ (µε)
- $F_{u, [i]} = 77.1$ ksi
- $\varepsilon_{u, [i]} = 255800$ (µε)

Coupon [i] - Coupon [iii]
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 4 x 4 x $\frac{1}{4}$

- $F_y, \text{[ii]} = 61.1$ ksi
- $\varepsilon_y, \text{[ii]} = 2019$ (µε)
- $F_u, \text{[ii]} = 75.7$ ksi
- $\varepsilon_u, \text{[ii]} = 286200$ (µε)

Tensile Coupon Results for HSS 4 x 4 x $\frac{1}{4}$

- $F_y, \text{[iii]} = 61$ ksi
- $\varepsilon_y, \text{[iii]} = 1998$ (µε)
- $F_u, \text{[iii]} = 75.9$ ksi
- $\varepsilon_u, \text{[iii]} = 278400$ (µε)
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 4 x 4 x \( \frac{1}{2} \)"

\[ F_y, \text{avg} = 63.9 \text{ ksi} \]
\[ \varepsilon_y, \text{avg} = 2577 \ (\mu\varepsilon) \]
\[ F_u, \text{avg} = 79.1 \text{ ksi} \]
\[ \varepsilon_u, \text{avg} = 263700 \ (\mu\varepsilon) \]

Coupon [ii]  Coupon [i]  Coupon [iii]
Tensile Coupon Results for HSS 4 x 4 x $\frac{1}{2}$ "

$F_y, [\text{ii}] = 65$ ksi
$\varepsilon_y, [\text{ii}] = 2590$ ($\mu$e)
$F_u, [\text{ii}] = 80.7$ ksi
$\varepsilon_u, [\text{ii}] = 266700$ ($\mu$e)

Tensile Coupon Results for HSS 4 x 4 x $\frac{1}{2}$ "

$F_y, [\text{iii}] = 63.2$ ksi
$\varepsilon_y, [\text{iii}] = 2431$ ($\mu$e)
$F_u, [\text{iii}] = 76.8$ ksi
$\varepsilon_u, [\text{iii}] = 266600$ ($\mu$e)
Tensile Coupon Results for HSS 6 x 6 x $\frac{1}{4}$"

**Coupon [i]**
- $F_y, avg = 48$ ksi
- $\epsilon_y, avg = 1858$ (\(\mu\varepsilon\))
- $F_u, avg = 63.6$ ksi
- $\epsilon_u, avg = 332200$ (\(\mu\varepsilon\))

**Coupon [ii]**
- $F_y, [ii] = 49.1$ ksi
- $\epsilon_y, [ii] = 1877$ (\(\mu\varepsilon\))
- $F_u, [ii] = 64.2$ ksi
- $\epsilon_u, [ii] = 344000$ (\(\mu\varepsilon\))

**Coupon [iii]**

---

Appendix E.3: RHS Mechanical Properties
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 6 x 6 x $\frac{1}{4}$

- $F_y_{[ii]} = 49.7$ ksi
- $\epsilon_y_{[ii]} = 1828$ (µε)
- $F_u_{[ii]} = 65.8$ ksi
- $\epsilon_u_{[ii]} = 331000$ (µε)

Tensile Coupon Results for HSS 6 x 6 x $\frac{1}{4}$

- $F_y_{[iii]} = 45.1$ ksi
- $\epsilon_y_{[iii]} = 1869$ (µε)
- $F_u_{[iii]} = 60.7$ ksi
- $\epsilon_u_{[iii]} = 321600$ (µε)
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 6 x 6 x 3/8"

- $F_y, \text{avg} = 50.7 \text{ ksi}
- $\varepsilon_y, \text{avg} = 1937 (\mu\varepsilon)
- $F_u, \text{avg} = 61.5 \text{ ksi}
- $\varepsilon_u, \text{avg} = 339133 (\mu\varepsilon)$

Tensile Coupon Results for HSS 6 x 6 x 3/8"

- $F_y, [i] = 51.1 \text{ ksi}
- $\varepsilon_y, [i] = 2044 (\mu\varepsilon)
- $F_u, [i] = 61.8 \text{ ksi}
- $\varepsilon_u, [i] = 326200 (\mu\varepsilon)$
Tensile Coupon Results for HSS 6 x 6 x $\frac{3}{8}$" Fy, [ii] = 49.8 ksi
εy, [ii] = 1885 (µε)
Fu, [ii] = 60.9 ksi
εu, [ii] = 336200 (µε)

Tensile Coupon Results for HSS 6 x 6 x $\frac{3}{8}$" Fy, [iii] = 51.2 ksi
εy, [iii] = 1882 (µε)
Fu, [iii] = 61.8 ksi
εu, [iii] = 355000 (µε)
Tensile Coupon Results for HSS 6 x 6 x 1/2 "

- $F_{y, avg} = 53.8$ ksi
- $\varepsilon_{y, avg} = 2033$ (µε)
- $F_{u, avg} = 64.3$ ksi
- $\varepsilon_{u, avg} = 339467$ (µε)

Coupon [i]
Coupon [ii]
Coupon [iii]

Tensile Coupon Results for HSS 6 x 6 x 1/2 "

- $F_{y, [i]} = 53.9$ ksi
- $\varepsilon_{y, [i]} = 2159$ (µε)
- $F_{u, [i]} = 65.2$ ksi
- $\varepsilon_{u, [i]} = 325500$ (µε)

Coupon [i]
0.2% Offset
Tensile Coupon Results for HSS 6 x 6 x \(\frac{1}{2}\)"

For Coupon [ii]:
- \(F_y, [\text{ii}] = 52.6 \text{ ksi}\)
- \(\varepsilon_y, [\text{ii}] = 1999 \text{ (\mu\epsilon)}\)
- \(F_u, [\text{ii}] = 63 \text{ ksi}\)
- \(\varepsilon_u, [\text{ii}] = 353300 \text{ (\mu\epsilon)}\)

For Coupon [iii]:
- \(F_y, [\text{iii}] = 54.8 \text{ ksi}\)
- \(\varepsilon_y, [\text{iii}] = 1941 \text{ (\mu\epsilon)}\)
- \(F_u, [\text{iii}] = 64.8 \text{ ksi}\)
- \(\varepsilon_u, [\text{iii}] = 339600 \text{ (\mu\epsilon)}\)
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 8 x 8 x 1/4" 

- $F_y, avg = 55.4$ ksi
- $\epsilon_y, avg = 2068$ (µε)
- $F_u, avg = 71.5$ ksi
- $\epsilon_u, avg = 274667$ (µε)

Coupon [i]

Coupon [ii]

Coupon [iii]

Tensile Coupon Results for HSS 8 x 8 x 1/4"

- $F_y, [i] = 58$ ksi
- $\epsilon_y, [i] = 2206$ (µε)
- $F_u, [i] = 72.6$ ksi
- $\epsilon_u, [i] = 288200$ (µε)

Coupon [i]

0.2% Offset
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 8 x 8 x 1/4"

\[ F_{y,\{\text{ii}\}} = 55.7 \text{ ksi} \]
\[ \varepsilon_{y,\{\text{ii}\}} = 2161 \text{ (µε)} \]
\[ F_{u,\{\text{ii}\}} = 72.3 \text{ ksi} \]
\[ \varepsilon_{u,\{\text{ii}\}} = 294400 \text{ (µε)} \]

Tensile Coupon Results for HSS 8 x 8 x 1/4"

\[ F_{y,\{\text{iii}\}} = 52.5 \text{ ksi} \]
\[ \varepsilon_{y,\{\text{iii}\}} = 1837 \text{ (µε)} \]
\[ F_{u,\{\text{iii}\}} = 69.7 \text{ ksi} \]
\[ \varepsilon_{u,\{\text{iii}\}} = 241400 \text{ (µε)} \]
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 8 x 8 x 3/8"

- $F_{y, avg} = 57.1$ ksi
- $\varepsilon_{y, avg} = 2243$ (µε)
- $F_{u, avg} = 73.9$ ksi
- $\varepsilon_{u, avg} = 322633$ (µε)

Tensile Coupon Results for HSS 8 x 8 x 3/8"

- $F_{y, [i]} = 57.9$ ksi
- $\varepsilon_{y, [i]} = 2171$ (µε)
- $F_{u, [i]} = 73.2$ ksi
- $\varepsilon_{u, [i]} = 316400$ (µε)
Tensile Coupon Results for HSS 8 x 8 x $3/8$

$F_y, \{ii\} = 55.8$ ksi  
$\varepsilon_y, \{ii\} = 2341$ (µε)  
$F_u, \{ii\} = 73.5$ ksi  
$\varepsilon_u, \{ii\} = 335400$ (µε)

Tensile Coupon Results for HSS 8 x 8 x $3/8$

$F_y, \{iii\} = 57.7$ ksi  
$\varepsilon_y, \{iii\} = 2217$ (µε)  
$F_u, \{iii\} = 75.1$ ksi  
$\varepsilon_u, \{iii\} = 316100$ (µε)

Coupon [ii]  
0.2% Offset

Coupon [iii]  
0.2% Offset
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 8 x 8 x $\frac{1}{2}$\textquoteright

\[
\begin{align*}
F_y, \text{avg} &= 59.8 \text{ ksi} \\
\varepsilon_y, \text{avg} &= 2244 (\mu \varepsilon) \\
F_u, \text{avg} &= 73.8 \text{ ksi} \\
\varepsilon_u, \text{avg} &= 341167 (\mu \varepsilon)
\end{align*}
\]

Tensile Coupon Results for HSS 8 x 8 x $\frac{1}{2}$\textquoteright

\[
\begin{align*}
F_y, \text{[i]} &= 61.2 \text{ ksi} \\
\varepsilon_y, \text{[i]} &= 2319 (\mu \varepsilon) \\
F_u, \text{[i]} &= 74 \text{ ksi} \\
\varepsilon_u, \text{[i]} &= 342900 (\mu \varepsilon)
\end{align*}
\]
Appendix E.3: RHS Mechanical Properties

Tensile Coupon Results for HSS 8 x 8 x \( \frac{1}{2} \)"

- \( F_{y, [\text{ii}]} = 59.9 \text{ ksi} \)
- \( \varepsilon_{y, [\text{ii}]} = 2010 \) (µε)
- \( F_{u, [\text{ii}]} = 72.9 \text{ ksi} \)
- \( \varepsilon_{u, [\text{ii}]} = 347000 \) (µε)

Tensile Coupon Results for HSS 8 x 8 x \( \frac{1}{2} \)"

- \( F_{y, [\text{iii}]} = 58.1 \text{ ksi} \)
- \( \varepsilon_{y, [\text{iii}]} = 2404 \) (µε)
- \( F_{u, [\text{iii}]} = 74.6 \text{ ksi} \)
- \( \varepsilon_{u, [\text{iii}]} = 333600 \) (µε)
## E.4 Mill Certificates from the Steel Fabricator

### Material Test Report

**Sold to**
Atlas ABC Corporation
1855 East 122nd Street
CHICAGO IL 60653
USA

**Shipped to**
Structures Laboratory
University of Toronto
35 St. George Street
TORONTO ON M5S 1A4
CANADA

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<tr>
<th>Material: 8.0x8.0x250x40°750x(3x3) V.CH-AKA500-V</th>
<th>Material No: 80080250</th>
<th>Made in: USA</th>
<th>Melted in: USA</th>
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### Sales order: 676142

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<th>Al</th>
<th>Cu</th>
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### CHARPY Test Results

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<th>Sample Size</th>
<th>Absorbed Energy1</th>
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<th>Absorbed Energy3</th>
<th>Avg FT-LBS</th>
<th>Shear Area1</th>
<th>Shear Area2</th>
<th>Shear Area3</th>
<th>Avg %</th>
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<td>61</td>
<td>56</td>
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**Material Note:**

**Sales Or. Note:**

---

### Material: 8.0x8.0x375x40°750x(3x3) V.CH-AKA500-V

### Sales order: 676142

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### CHARPY Test Results

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<th>Sample Size</th>
<th>Absorbed Energy1</th>
<th>Absorbed Energy2</th>
<th>Absorbed Energy3</th>
<th>Avg FT-LBS</th>
<th>Shear Area1</th>
<th>Shear Area2</th>
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<th>Avg %</th>
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<td>61</td>
<td>70</td>
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**Material Note:**

**Sales Or. Note:**

---

**Authorized by Quality Assurance:**

The results reported on this report represent the actual attributes of the material furnished and indicate full compliance with all applicable specification and contract requirements. CE calculated using the AWS D1.1 method.
Appendix E.4: Mill Certificates from the Steel Fabricator

Material: 8.0x8.0x500x40°0)(2x2V.CH-A/A500-V  
Material No: 80080500  
Made In: USA  
Melted In: USA

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<thead>
<tr>
<th>Test FL Ibs Temp</th>
<th>Sample Size</th>
<th>Absorbed Energy1 FT-LBS</th>
<th>Absorbed Energy2 FT-LBS</th>
<th>Absorbed Energy3 FT-LBS</th>
<th>Shear Avg.</th>
<th>Shear Area1 %</th>
<th>Shear Area2 %</th>
<th>Shear Area3 %</th>
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</thead>
<tbody>
<tr>
<td>20</td>
<td>0F</td>
<td>10x10 mm</td>
<td>110</td>
<td>123</td>
<td>101</td>
<td>90</td>
<td>100</td>
<td>80</td>
</tr>
</tbody>
</table>

Material Note:  
Sales Or. Note:

Authorized by Quality Assurance:

The results reported on this report represent the actual attributes of the material furnished and indicate full compliance with all applicable specification and contract requirements.

'011.1 method.'
### Appendix E.4: Mill Certificates from the Steel Fabricator

**MATERIAL TEST REPORT**

**Sold to:**
Atlas ABC Corporation
1655 East 122nd Street
CHICAGO IL 60633
USA

**Shipped to:**
Structures Laboratory
University of Toronto
35 St. George Street
TORONTO ON MSS 1A4
CANADA

<table>
<thead>
<tr>
<th>Material: 4.0x4.0x25.0x0.070x(3x3)()V.CH-AKA-000-V</th>
<th>Material No: 40040250</th>
<th>Made in: USA</th>
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</thead>
<tbody>
<tr>
<td>Made in: USA</td>
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<td></td>
</tr>
<tr>
<td>Melted in: USA</td>
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<td></td>
</tr>
</tbody>
</table>

#### Material Test Results

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<th>Charpy Test Results</th>
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<td>53</td>
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**Material Note:**
Sales Or.Note:

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<th>Made in: USA</th>
</tr>
</thead>
<tbody>
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<td></td>
</tr>
<tr>
<td>Melted in: USA</td>
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<td></td>
</tr>
</tbody>
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#### Material Test Results

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<tr>
<th>Test Fl_LbsTemp</th>
<th>Sample Size</th>
<th>Absorbed Energy1 FT-LBS</th>
<th>Absorbed Energy2 FT-LBS</th>
<th>Charpy Test Results</th>
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</thead>
<tbody>
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**Material Note:**
Sales Or.Note:

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</tr>
<tr>
<td>Melted in: USA</td>
<td></td>
<td></td>
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#### Material Test Results

<table>
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<tr>
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<th>Sample Size</th>
<th>Absorbed Energy1 FT-LBS</th>
<th>Absorbed Energy2 FT-LBS</th>
<th>Charpy Test Results</th>
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<tbody>
<tr>
<td>20°F</td>
<td>80°F</td>
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<td>55</td>
<td>53</td>
</tr>
</tbody>
</table>

**Material Note:**
Sales Or.Note:

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**Page 1 of 2**

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Appendix E.4: Mill Certificates from the Steel Fabricator

---

**ATLAS TUBE**

**MATERIAL TEST REPORT**

<table>
<thead>
<tr>
<th>Heat No</th>
<th>C</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>Si</th>
<th>Al</th>
<th>Cu</th>
<th>Cb</th>
<th>Mo</th>
<th>Ni</th>
<th>Cr</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>761281</td>
<td>0.190</td>
<td>0.770</td>
<td>0.010</td>
<td>0.008</td>
<td>0.019</td>
<td>0.042</td>
<td>0.004</td>
<td>0.008</td>
<td>0.023</td>
<td>0.054</td>
<td>0.002</td>
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**Material**

6 X 6 X .250

**Yield**

658990 Psi

**Tensile**

72020 Psi

**Elt.2in**

35.0 %

**Certification**

ASTM A560-10A GRADE B&C

---

Authorized by Quality Assurance: [Signature]

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Page: 1 of 1

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## Appendix E.4: Mill Certificates from the Steel Fabricator

![Atlas Tube Canada ULC logo](image-url) Atlas Tube Canada ULC  
200 Clark St.  
Hamrow, Ontario, Canada  
N0R 1G0  
Tel: 519-736-3641  
Fax: 519-736-3537

### MATERIAL TEST REPORT

<table>
<thead>
<tr>
<th>Heat No</th>
<th>C</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>Si</th>
<th>Al</th>
<th>Cu</th>
<th>Cb</th>
<th>Mo</th>
<th>Ni</th>
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<th>V</th>
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<td>0.008</td>
<td>0.019</td>
<td>0.042</td>
<td>0.007</td>
<td>0.004</td>
<td>0.008</td>
<td>0.023</td>
<td>0.054</td>
<td>0.002</td>
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<table>
<thead>
<tr>
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<th>Tensile</th>
<th>Els.2in</th>
<th>Certification</th>
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</thead>
<tbody>
<tr>
<td>6 X 6 X .375</td>
<td>666450 Psi</td>
<td>667470 Psi</td>
<td>34.5 %</td>
<td>ASTM A500-10A GRADE B&amp;C</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Heat No</th>
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<th>P</th>
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<th>Si</th>
<th>Al</th>
<th>Cu</th>
<th>Cb</th>
<th>Mo</th>
<th>Ni</th>
<th>Cr</th>
<th>V</th>
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</thead>
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<tr>
<td>781251</td>
<td>0.190</td>
<td>0.770</td>
<td>0.010</td>
<td>0.008</td>
<td>0.019</td>
<td>0.042</td>
<td>0.007</td>
<td>0.004</td>
<td>0.008</td>
<td>0.023</td>
<td>0.054</td>
<td>0.002</td>
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</tbody>
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<table>
<thead>
<tr>
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<th>Yield</th>
<th>Tensile</th>
<th>Els.2in</th>
<th>Certification</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 X 6 X .375</td>
<td>666450 Psi</td>
<td>667470 Psi</td>
<td>34.5 %</td>
<td>CSA G40.21-04 50W CLASS C</td>
</tr>
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</table>

Authorized by Quality Assurance:

![Quality Assurance Signature](signature-url)
### Appendix E.4: Mill Certificates from the Steel Fabricator

**Material Test Report**

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<thead>
<tr>
<th>Heat No</th>
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<th>S</th>
<th>Si</th>
<th>Al</th>
<th>Cu</th>
<th>Cb</th>
<th>Mo</th>
<th>Ni</th>
<th>Cr</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>606455</td>
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<td>0.810</td>
<td>0.008</td>
<td>0.007</td>
<td>0.013</td>
<td>0.050</td>
<td>0.050</td>
<td>0.006</td>
<td>0.004</td>
<td>0.015</td>
<td>0.046</td>
<td>0.002</td>
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<table>
<thead>
<tr>
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<th>Yield</th>
<th>Tensile</th>
<th>Elsn2in</th>
<th>Certification</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 X 6 X .500</td>
<td>664370 Psi</td>
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<td>35.3 %</td>
<td>ASTM A600-10A GRADE B&amp;C</td>
</tr>
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</table>

Authorized by Quality Assurance: ____________________________

Page: 1 Of 1
APPENDIX F   Weld Geometric and Mechanical Properties
## Appendix F.1: External Weld Measurements

### Table F-1 Weld measurements for the east weld element of specimen T-0.25-34

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SE</td>
<td>0.12</td>
<td>0.16</td>
<td>0.10</td>
</tr>
<tr>
<td>6SE</td>
<td>0.14</td>
<td>0.15</td>
<td>0.09</td>
</tr>
<tr>
<td>7E</td>
<td>0.16</td>
<td>0.16</td>
<td>0.09</td>
</tr>
<tr>
<td>6NE</td>
<td>0.15</td>
<td>0.14</td>
<td>0.09</td>
</tr>
<tr>
<td>5NE</td>
<td>0.16</td>
<td>0.14</td>
<td>0.10</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.14</strong></td>
<td><strong>0.15</strong></td>
<td><strong>0.09</strong></td>
</tr>
</tbody>
</table>

### Table F-2 Weld measurements for the west weld element of specimen T-0.25-34

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SW</td>
<td>0.14</td>
<td>0.12</td>
<td>0.08</td>
</tr>
<tr>
<td>6SW</td>
<td>0.16</td>
<td>0.14</td>
<td>0.09</td>
</tr>
<tr>
<td>7W</td>
<td>0.16</td>
<td>0.13</td>
<td>0.10</td>
</tr>
<tr>
<td>6NW</td>
<td>0.16</td>
<td>0.12</td>
<td>0.10</td>
</tr>
<tr>
<td>5NW</td>
<td>0.15</td>
<td>0.13</td>
<td>0.11</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.15</strong></td>
<td><strong>0.13</strong></td>
<td><strong>0.09</strong></td>
</tr>
</tbody>
</table>

### Table F-3 Weld measurements for the north weld element of specimen T-0.25-34

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3NW</td>
<td>0.13</td>
<td>0.10</td>
<td>0.07</td>
</tr>
<tr>
<td>2NW</td>
<td>0.15</td>
<td>0.12</td>
<td>0.07</td>
</tr>
<tr>
<td>1N</td>
<td>0.14</td>
<td>0.12</td>
<td>0.08</td>
</tr>
<tr>
<td>2NE</td>
<td>0.15</td>
<td>0.13</td>
<td>0.08</td>
</tr>
<tr>
<td>3NE</td>
<td>0.12</td>
<td>0.10</td>
<td>0.08</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.13</strong></td>
<td><strong>0.11</strong></td>
<td><strong>0.08</strong></td>
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</tbody>
</table>

### Table F-4 Weld measurements for the south weld element of specimen T-0.25-34

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<th>Location</th>
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<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
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</thead>
<tbody>
<tr>
<td>3SW</td>
<td>0.11</td>
<td>0.12</td>
<td>0.09</td>
</tr>
<tr>
<td>2SW</td>
<td>0.13</td>
<td>0.12</td>
<td>0.09</td>
</tr>
<tr>
<td>1S</td>
<td>0.15</td>
<td>0.14</td>
<td>0.10</td>
</tr>
<tr>
<td>2SE</td>
<td>0.15</td>
<td>0.14</td>
<td>0.10</td>
</tr>
<tr>
<td>3SE</td>
<td>0.16</td>
<td>0.12</td>
<td>0.09</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.14</strong></td>
<td><strong>0.13</strong></td>
<td><strong>0.09</strong></td>
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</tbody>
</table>
### Table F-5 Weld measurements for the east weld element of specimen T-0.25-23

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SE</td>
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<td>0.13</td>
<td>0.08</td>
</tr>
<tr>
<td>6SE</td>
<td>0.14</td>
<td>0.13</td>
<td>0.09</td>
</tr>
<tr>
<td>7E</td>
<td>0.15</td>
<td>0.14</td>
<td>0.10</td>
</tr>
<tr>
<td>6NE</td>
<td>0.16</td>
<td>0.15</td>
<td>0.09</td>
</tr>
<tr>
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<td>0.14</td>
<td>0.13</td>
<td>0.08</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.14</strong></td>
<td><strong>0.13</strong></td>
<td><strong>0.09</strong></td>
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</tbody>
</table>

### Table F-6 Weld measurements for the west weld element of specimen T-0.25-23

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
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</thead>
<tbody>
<tr>
<td>5SW</td>
<td>0.12</td>
<td>0.11</td>
<td>0.08</td>
</tr>
<tr>
<td>6SW</td>
<td>0.14</td>
<td>0.13</td>
<td>0.09</td>
</tr>
<tr>
<td>7W</td>
<td>0.16</td>
<td>0.11</td>
<td>0.09</td>
</tr>
<tr>
<td>6NW</td>
<td>0.13</td>
<td>0.11</td>
<td>0.10</td>
</tr>
<tr>
<td>5NW</td>
<td>0.12</td>
<td>0.11</td>
<td>0.10</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.13</strong></td>
<td><strong>0.12</strong></td>
<td><strong>0.09</strong></td>
</tr>
</tbody>
</table>

### Table F-7 Weld measurements for the north weld element of specimen T-0.25-23

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3NW</td>
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<td>0.12</td>
<td>0.08</td>
</tr>
<tr>
<td>2NW</td>
<td>0.16</td>
<td>0.12</td>
<td>0.09</td>
</tr>
<tr>
<td>1N</td>
<td>0.18</td>
<td>0.14</td>
<td>0.10</td>
</tr>
<tr>
<td>2NE</td>
<td>0.16</td>
<td>0.12</td>
<td>0.10</td>
</tr>
<tr>
<td>3NE</td>
<td>0.12</td>
<td>0.11</td>
<td>0.08</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.15</strong></td>
<td><strong>0.12</strong></td>
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### Table F-8 Weld measurements for the south weld element of specimen T-0.25-23

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</tr>
</thead>
<tbody>
<tr>
<td>3SW</td>
<td>0.11</td>
<td>0.12</td>
<td>0.09</td>
</tr>
<tr>
<td>2SW</td>
<td>0.14</td>
<td>0.12</td>
<td>0.09</td>
</tr>
<tr>
<td>1S</td>
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<td>0.10</td>
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<tr>
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<td>0.15</td>
<td>0.11</td>
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<tr>
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<td>0.15</td>
<td>0.09</td>
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<tr>
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### Table F-9 Weld measurements for the east weld element of specimen T-0.25-17

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<td>0.11</td>
</tr>
<tr>
<td>7E</td>
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<td>0.16</td>
<td>0.10</td>
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<tr>
<td>6NE</td>
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<td>0.12</td>
<td>0.10</td>
</tr>
<tr>
<td>5NE</td>
<td>0.13</td>
<td>0.15</td>
<td>0.08</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.12</strong></td>
<td><strong>0.14</strong></td>
<td><strong>0.10</strong></td>
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</tbody>
</table>

### Table F-10 Weld measurements for the west weld element of specimen T-0.25-17

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<th>Vertical Leg</th>
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<th>Theoretical Throat</th>
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</thead>
<tbody>
<tr>
<td>5SW</td>
<td>0.12</td>
<td>0.14</td>
<td>0.08</td>
</tr>
<tr>
<td>6SW</td>
<td>0.11</td>
<td>0.15</td>
<td>0.10</td>
</tr>
<tr>
<td>7W</td>
<td>0.12</td>
<td>0.16</td>
<td>0.11</td>
</tr>
<tr>
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<td>0.16</td>
<td>0.09</td>
</tr>
<tr>
<td>5NW</td>
<td>0.08</td>
<td>0.15</td>
<td>0.08</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.11</strong></td>
<td><strong>0.15</strong></td>
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</table>

### Table F-11 Weld measurements for the north weld element of specimen T-0.25-17

<table>
<thead>
<tr>
<th>Location</th>
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<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3NW</td>
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<td>0.13</td>
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<td>0.14</td>
<td>0.09</td>
</tr>
<tr>
<td>1N</td>
<td>0.16</td>
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</tr>
<tr>
<td>2NE</td>
<td>0.15</td>
<td>0.14</td>
<td>0.09</td>
</tr>
<tr>
<td>3NE</td>
<td>0.12</td>
<td>0.13</td>
<td>0.09</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.14</strong></td>
<td><strong>0.13</strong></td>
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</table>

### Table F-12 Weld measurements for the south weld element of specimen T-0.25-17

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<th>Theoretical Throat</th>
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<tbody>
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<tr>
<td>2SW</td>
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<td>0.09</td>
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<tr>
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<td>0.13</td>
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</tr>
<tr>
<td>3SE</td>
<td>0.14</td>
<td>0.13</td>
<td>0.08</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.13</strong></td>
<td><strong>0.12</strong></td>
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</table>
### Appendix F.1: External Weld Measurements

#### Table F-13 Weld measurements for the east weld element of specimen T-0.50-34

<table>
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</thead>
<tbody>
<tr>
<td>5SE</td>
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<td>0.16</td>
<td>0.11</td>
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<tr>
<td>6SE</td>
<td>0.20</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td>7E</td>
<td>0.24</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td>6NE</td>
<td>0.22</td>
<td>0.24</td>
<td>0.15</td>
</tr>
<tr>
<td>5NE</td>
<td>0.22</td>
<td>0.24</td>
<td>0.15</td>
</tr>
<tr>
<td><strong>Average</strong></td>
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#### Table F-14 Weld measurements for the west weld element of specimen T-0.50-34

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</tr>
</thead>
<tbody>
<tr>
<td>5SW</td>
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<td>0.16</td>
<td>0.11</td>
</tr>
<tr>
<td>6SW</td>
<td>0.28</td>
<td>0.14</td>
<td>0.10</td>
</tr>
<tr>
<td>7W</td>
<td>0.23</td>
<td>0.24</td>
<td>0.16</td>
</tr>
<tr>
<td>6NW</td>
<td>0.20</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td>5NW</td>
<td>0.19</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.22</strong></td>
<td><strong>0.19</strong></td>
<td><strong>0.13</strong></td>
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</table>

#### Table F-15 Weld measurements for the north weld element of specimen T-0.50-34

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</tr>
</thead>
<tbody>
<tr>
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<td>0.14</td>
</tr>
<tr>
<td>2NW</td>
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<td>0.18</td>
<td>0.13</td>
</tr>
<tr>
<td>1N</td>
<td>0.20</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td>2NE</td>
<td>0.20</td>
<td>0.19</td>
<td>0.14</td>
</tr>
<tr>
<td>3NE</td>
<td>0.20</td>
<td>0.16</td>
<td>0.11</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.20</strong></td>
<td><strong>0.19</strong></td>
<td><strong>0.13</strong></td>
</tr>
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</table>

#### Table F-16 Weld measurements for the south weld element of specimen T-0.50-34

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<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3SW</td>
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<td>0.13</td>
</tr>
<tr>
<td>2SW</td>
<td>0.20</td>
<td>0.18</td>
<td>0.13</td>
</tr>
<tr>
<td>1S</td>
<td>0.20</td>
<td>0.18</td>
<td>0.13</td>
</tr>
<tr>
<td>2SE</td>
<td>0.20</td>
<td>0.18</td>
<td>0.13</td>
</tr>
<tr>
<td>3SE</td>
<td>0.18</td>
<td>0.16</td>
<td>0.11</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.19</strong></td>
<td><strong>0.17</strong></td>
<td><strong>0.12</strong></td>
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</table>
### Table F-17 Weld measurements for the east weld element of specimen T-0.50-23

<table>
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<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SE</td>
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<td>0.19</td>
<td>0.13</td>
</tr>
<tr>
<td>6SE</td>
<td>0.20</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td>7E</td>
<td>0.19</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td>6NE</td>
<td>0.19</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td>5NE</td>
<td>0.20</td>
<td>0.19</td>
<td>0.13</td>
</tr>
<tr>
<td><strong>Average</strong></td>
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<td><strong>0.20</strong></td>
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</table>

### Table F-18 Weld measurements for the west weld element of specimen T-0.50-23

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</thead>
<tbody>
<tr>
<td>5SW</td>
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<td>0.17</td>
<td>0.12</td>
</tr>
<tr>
<td>6SW</td>
<td>0.20</td>
<td>0.18</td>
<td>0.13</td>
</tr>
<tr>
<td>7W</td>
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<td>0.14</td>
</tr>
<tr>
<td>6NW</td>
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<td>0.20</td>
<td>0.13</td>
</tr>
<tr>
<td>5NW</td>
<td>0.16</td>
<td>0.19</td>
<td>0.11</td>
</tr>
<tr>
<td><strong>Average</strong></td>
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<td><strong>0.19</strong></td>
<td><strong>0.12</strong></td>
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### Table F-19 Weld measurements for the north weld element of specimen T-0.50-23

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<tbody>
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<td>0.13</td>
</tr>
<tr>
<td>2NW</td>
<td>0.20</td>
<td>0.18</td>
<td>0.13</td>
</tr>
<tr>
<td>1N</td>
<td>0.20</td>
<td>0.19</td>
<td>0.13</td>
</tr>
<tr>
<td>2NE</td>
<td>0.20</td>
<td>0.17</td>
<td>0.12</td>
</tr>
<tr>
<td>3NE</td>
<td>0.19</td>
<td>0.16</td>
<td>0.11</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.20</strong></td>
<td><strong>0.18</strong></td>
<td><strong>0.12</strong></td>
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### Table F-20 Weld measurements for the south weld element of specimen T-0.50-23

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</thead>
<tbody>
<tr>
<td>3SW</td>
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<td>0.11</td>
</tr>
<tr>
<td>2SW</td>
<td>0.18</td>
<td>0.19</td>
<td>0.13</td>
</tr>
<tr>
<td>1S</td>
<td>0.18</td>
<td>0.16</td>
<td>0.11</td>
</tr>
<tr>
<td>2SE</td>
<td>0.20</td>
<td>0.12</td>
<td>0.08</td>
</tr>
<tr>
<td>3SE</td>
<td>0.20</td>
<td>0.16</td>
<td>0.11</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.19</strong></td>
<td><strong>0.16</strong></td>
<td><strong>0.11</strong></td>
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</table>
### Table F-21 Weld measurements for the east weld element of specimen T-0.50-17

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<tbody>
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<tr>
<td>6SE</td>
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<td>0.45</td>
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</tr>
<tr>
<td>7E</td>
<td>0.43</td>
<td>0.47</td>
<td>0.31</td>
</tr>
<tr>
<td>6NE</td>
<td>0.41</td>
<td>0.47</td>
<td>0.29</td>
</tr>
<tr>
<td>5NE</td>
<td>0.39</td>
<td>0.43</td>
<td>0.28</td>
</tr>
<tr>
<td><strong>Average</strong></td>
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### Table F-22 Weld measurements for the west weld element of specimen T-0.50-17

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</thead>
<tbody>
<tr>
<td>5SW</td>
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<td>0.26</td>
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<tr>
<td>6SW</td>
<td>0.47</td>
<td>0.39</td>
<td>0.28</td>
</tr>
<tr>
<td>7W</td>
<td>0.39</td>
<td>0.43</td>
<td>0.28</td>
</tr>
<tr>
<td>6NW</td>
<td>0.41</td>
<td>0.39</td>
<td>0.28</td>
</tr>
<tr>
<td>5NW</td>
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<td>0.35</td>
<td>0.25</td>
</tr>
<tr>
<td><strong>Average</strong></td>
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<td><strong>0.39</strong></td>
<td><strong>0.27</strong></td>
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### Table F-23 Weld measurements for the north weld element of specimen T-0.50-17

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</thead>
<tbody>
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<td>3NW</td>
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<td>0.29</td>
</tr>
<tr>
<td>2NW</td>
<td>0.39</td>
<td>0.45</td>
<td>0.28</td>
</tr>
<tr>
<td>1N</td>
<td>0.39</td>
<td>0.47</td>
<td>0.28</td>
</tr>
<tr>
<td>2NE</td>
<td>0.39</td>
<td>0.47</td>
<td>0.28</td>
</tr>
<tr>
<td>3NE</td>
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<td>0.47</td>
<td>0.29</td>
</tr>
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### Table F-24 Weld measurements for the south weld element of specimen T-0.50-17

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</thead>
<tbody>
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<td>0.29</td>
</tr>
<tr>
<td>2SW</td>
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<td>0.39</td>
<td>0.28</td>
</tr>
<tr>
<td>1S</td>
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<td>0.45</td>
<td>0.43</td>
<td>0.31</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.45</strong></td>
<td><strong>0.43</strong></td>
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### Appendix F.1: External Weld Measurements

#### Table F-25 Weld measurements for the east weld element of specimen T-0.75-34

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</thead>
<tbody>
<tr>
<td>5SE</td>
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<td>0.10</td>
</tr>
<tr>
<td>6SE</td>
<td>0.12</td>
<td>0.10</td>
<td>0.09</td>
</tr>
<tr>
<td>7E</td>
<td>0.11</td>
<td>0.10</td>
<td>0.09</td>
</tr>
<tr>
<td>6NE</td>
<td>0.14</td>
<td>0.12</td>
<td>0.10</td>
</tr>
<tr>
<td>5NE</td>
<td>0.13</td>
<td>0.11</td>
<td>0.09</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.12</strong></td>
<td><strong>0.10</strong></td>
<td><strong>0.09</strong></td>
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#### Table F-26 Weld measurements for the west weld element of specimen T-0.75-34

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</thead>
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<tr>
<td>6SW</td>
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<td>0.09</td>
</tr>
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<td>7W</td>
<td>0.14</td>
<td>0.15</td>
<td>0.09</td>
</tr>
<tr>
<td>6NW</td>
<td>0.12</td>
<td>0.16</td>
<td>0.09</td>
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<td>0.12</td>
<td>0.13</td>
<td>0.07</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.12</strong></td>
<td><strong>0.14</strong></td>
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#### Table F-27 Weld measurements for the north weld element of specimen T-0.75-34

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<th>Horizontal Leg</th>
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<tr>
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<td>0.08</td>
</tr>
<tr>
<td>2NW</td>
<td>0.12</td>
<td>0.14</td>
<td>0.10</td>
</tr>
<tr>
<td>1N</td>
<td>0.14</td>
<td>0.12</td>
<td>0.09</td>
</tr>
<tr>
<td>2NE</td>
<td>0.12</td>
<td>0.11</td>
<td>0.08</td>
</tr>
<tr>
<td>3NE</td>
<td>0.16</td>
<td>0.11</td>
<td>0.09</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.13</strong></td>
<td><strong>0.12</strong></td>
<td><strong>0.09</strong></td>
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</tbody>
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#### Table F-28 Weld measurements for the south weld element of specimen T-0.75-34

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</tr>
</thead>
<tbody>
<tr>
<td>3SW</td>
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<td>0.14</td>
<td>0.09</td>
</tr>
<tr>
<td>2SW</td>
<td>0.15</td>
<td>0.13</td>
<td>0.10</td>
</tr>
<tr>
<td>1S</td>
<td>0.12</td>
<td>0.14</td>
<td>0.09</td>
</tr>
<tr>
<td>2SE</td>
<td>0.13</td>
<td>0.13</td>
<td>0.09</td>
</tr>
<tr>
<td>3SE</td>
<td>0.12</td>
<td>0.14</td>
<td>0.09</td>
</tr>
<tr>
<td><strong>Average</strong></td>
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<td><strong>0.13</strong></td>
<td><strong>0.09</strong></td>
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</tbody>
</table>
### Table F-29 Weld measurements for the east weld element of specimen T-0.75-23

<table>
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<th>Theoretical Throat</th>
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</thead>
<tbody>
<tr>
<td>5SE</td>
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<td>0.24</td>
<td>0.14</td>
</tr>
<tr>
<td>6SE</td>
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<td>0.26</td>
<td>0.16</td>
</tr>
<tr>
<td>7E</td>
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<td>0.26</td>
<td>0.15</td>
</tr>
<tr>
<td>6NE</td>
<td>0.20</td>
<td>0.26</td>
<td>0.14</td>
</tr>
<tr>
<td>5NE</td>
<td>0.16</td>
<td>0.24</td>
<td>0.12</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.18</strong></td>
<td><strong>0.25</strong></td>
<td><strong>0.14</strong></td>
</tr>
</tbody>
</table>

### Table F-30 Weld measurements for the west weld element of specimen T-0.75-23

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SW</td>
<td>0.26</td>
<td>0.20</td>
<td>0.12</td>
</tr>
<tr>
<td>6SW</td>
<td>0.26</td>
<td>0.24</td>
<td>0.15</td>
</tr>
<tr>
<td>7W</td>
<td>0.24</td>
<td>0.26</td>
<td>0.12</td>
</tr>
<tr>
<td>6NW</td>
<td>0.24</td>
<td>0.24</td>
<td>0.15</td>
</tr>
<tr>
<td>5NW</td>
<td>0.24</td>
<td>0.24</td>
<td>0.12</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.24</strong></td>
<td><strong>0.23</strong></td>
<td><strong>0.13</strong></td>
</tr>
</tbody>
</table>

### Table F-31 Weld measurements for the north weld element of specimen T-0.75-23

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3NW</td>
<td>0.20</td>
<td>0.22</td>
<td>0.12</td>
</tr>
<tr>
<td>2NW</td>
<td>0.22</td>
<td>0.24</td>
<td>0.16</td>
</tr>
<tr>
<td>1N</td>
<td>0.22</td>
<td>0.26</td>
<td>0.16</td>
</tr>
<tr>
<td>2NE</td>
<td>0.22</td>
<td>0.24</td>
<td>0.16</td>
</tr>
<tr>
<td>3NE</td>
<td>0.24</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.22</strong></td>
<td><strong>0.23</strong></td>
<td><strong>0.15</strong></td>
</tr>
</tbody>
</table>

### Table F-32 Weld measurements for the south weld element of specimen T-0.75-23

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3SW</td>
<td>0.18</td>
<td>0.20</td>
<td>0.12</td>
</tr>
<tr>
<td>2SW</td>
<td>0.20</td>
<td>0.18</td>
<td>0.12</td>
</tr>
<tr>
<td>1S</td>
<td>0.20</td>
<td>0.22</td>
<td>0.14</td>
</tr>
<tr>
<td>2SE</td>
<td>0.22</td>
<td>0.20</td>
<td>0.16</td>
</tr>
<tr>
<td>3SE</td>
<td>0.22</td>
<td>0.20</td>
<td>0.14</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.20</strong></td>
<td><strong>0.20</strong></td>
<td><strong>0.13</strong></td>
</tr>
</tbody>
</table>
### Table F-33: Weld measurements for the east weld element of specimen T-0.75-17

<table>
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<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SE</td>
<td>0.28</td>
<td>0.22</td>
<td>0.16</td>
</tr>
<tr>
<td>6SE</td>
<td>0.35</td>
<td>0.28</td>
<td>0.16</td>
</tr>
<tr>
<td>7E</td>
<td>0.35</td>
<td>0.24</td>
<td>0.14</td>
</tr>
<tr>
<td>6NE</td>
<td>0.35</td>
<td>0.24</td>
<td>0.12</td>
</tr>
<tr>
<td>5NE</td>
<td>0.24</td>
<td>0.24</td>
<td>0.16</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.31</strong></td>
<td><strong>0.24</strong></td>
<td><strong>0.15</strong></td>
</tr>
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</table>

### Table F-34: Weld measurements for the west weld element of specimen T-0.75-17

<table>
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<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SW</td>
<td>0.31</td>
<td>0.24</td>
<td>0.16</td>
</tr>
<tr>
<td>6SW</td>
<td>0.31</td>
<td>0.28</td>
<td>0.22</td>
</tr>
<tr>
<td>7W</td>
<td>0.33</td>
<td>0.28</td>
<td>0.22</td>
</tr>
<tr>
<td>6NW</td>
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<td>0.30</td>
<td>0.20</td>
</tr>
<tr>
<td>5NW</td>
<td>0.24</td>
<td>0.31</td>
<td>0.14</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.31</strong></td>
<td><strong>0.28</strong></td>
<td><strong>0.19</strong></td>
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</table>

### Table F-35: Weld measurements for the north weld element of specimen T-0.75-17

<table>
<thead>
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<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3NW</td>
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<td>0.28</td>
<td>0.24</td>
</tr>
<tr>
<td>2NW</td>
<td>0.26</td>
<td>0.28</td>
<td>0.24</td>
</tr>
<tr>
<td>1N</td>
<td>0.26</td>
<td>0.33</td>
<td>0.24</td>
</tr>
<tr>
<td>2NE</td>
<td>0.26</td>
<td>0.35</td>
<td>0.24</td>
</tr>
<tr>
<td>3NE</td>
<td>0.26</td>
<td>0.31</td>
<td>0.22</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.26</strong></td>
<td><strong>0.31</strong></td>
<td><strong>0.23</strong></td>
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</table>

### Table F-36: Weld measurements for the south weld element of specimen T-0.75-17

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3SW</td>
<td>0.26</td>
<td>0.26</td>
<td>0.24</td>
</tr>
<tr>
<td>2SW</td>
<td>0.28</td>
<td>0.31</td>
<td>0.22</td>
</tr>
<tr>
<td>1S</td>
<td>0.28</td>
<td>0.31</td>
<td>0.22</td>
</tr>
<tr>
<td>2SE</td>
<td>0.30</td>
<td>0.31</td>
<td>0.22</td>
</tr>
<tr>
<td>3SE</td>
<td>0.28</td>
<td>0.28</td>
<td>0.20</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.28</strong></td>
<td><strong>0.30</strong></td>
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</table>
### Table F-37 Weld measurements for the east weld element of specimen T-1.00-34

<table>
<thead>
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<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SE</td>
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<td>0.27</td>
</tr>
<tr>
<td>6SE</td>
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<td>-</td>
<td>0.17</td>
</tr>
<tr>
<td>7E</td>
<td>-</td>
<td>-</td>
<td>0.17</td>
</tr>
<tr>
<td>6NE</td>
<td>-</td>
<td>-</td>
<td>0.17</td>
</tr>
<tr>
<td>5NE</td>
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<td>-</td>
<td>0.17</td>
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<tr>
<td><strong>Average</strong></td>
<td>-</td>
<td>-</td>
<td><strong>0.19</strong></td>
</tr>
</tbody>
</table>

### Table F-38 Weld measurements for the west weld element of specimen T-1.00-34

<table>
<thead>
<tr>
<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SW</td>
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</tr>
<tr>
<td>6SW</td>
<td>-</td>
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</tr>
<tr>
<td>7W</td>
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<tr>
<td>6NW</td>
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<tr>
<td>5NW</td>
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<td>-</td>
<td>0.17</td>
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<td>-</td>
<td>-</td>
<td><strong>0.17</strong></td>
</tr>
</tbody>
</table>

### Table F-39 Weld measurements for the north weld element of specimen T-1.00-34

<table>
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<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3NW</td>
<td>0.13</td>
<td>0.26</td>
<td>0.12</td>
</tr>
<tr>
<td>2NW</td>
<td>0.17</td>
<td>0.28</td>
<td>0.15</td>
</tr>
<tr>
<td>1N</td>
<td>0.16</td>
<td>0.28</td>
<td>0.14</td>
</tr>
<tr>
<td>2NE</td>
<td>0.18</td>
<td>0.28</td>
<td>0.14</td>
</tr>
<tr>
<td>3NE</td>
<td>0.16</td>
<td>0.26</td>
<td>0.12</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.16</strong></td>
<td><strong>0.27</strong></td>
<td><strong>0.13</strong></td>
</tr>
</tbody>
</table>

### Table F-40 Weld measurements for the south weld element of specimen T-1.00-34

<table>
<thead>
<tr>
<th>Location</th>
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<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3SW</td>
<td>0.15</td>
<td>0.20</td>
<td>0.12</td>
</tr>
<tr>
<td>2SW</td>
<td>0.16</td>
<td>0.22</td>
<td>0.13</td>
</tr>
<tr>
<td>1S</td>
<td>0.19</td>
<td>0.22</td>
<td>0.14</td>
</tr>
<tr>
<td>2SE</td>
<td>0.18</td>
<td>0.24</td>
<td>0.14</td>
</tr>
<tr>
<td>3SE</td>
<td>0.18</td>
<td>0.24</td>
<td>0.11</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.17</strong></td>
<td><strong>0.22</strong></td>
<td><strong>0.13</strong></td>
</tr>
</tbody>
</table>
## Appendix F.1: External Weld Measurements

### Table F-41 Weld measurements for the east weld element of specimen T-1.00-23

<table>
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<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SE</td>
<td>-</td>
<td>-</td>
<td>0.43</td>
</tr>
<tr>
<td>6SE</td>
<td>-</td>
<td>-</td>
<td>0.43</td>
</tr>
<tr>
<td>7E</td>
<td>-</td>
<td>-</td>
<td>0.43</td>
</tr>
<tr>
<td>6NE</td>
<td>-</td>
<td>-</td>
<td>0.43</td>
</tr>
<tr>
<td>5NE</td>
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<td>0.43</td>
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<td>-</td>
<td>-</td>
<td><strong>0.43</strong></td>
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</table>

### Table F-42 Weld measurements for the west weld element of specimen T-1.00-23

<table>
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<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
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<td>5SW</td>
<td>-</td>
<td>-</td>
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</tr>
<tr>
<td>6SW</td>
<td>-</td>
<td>-</td>
<td>0.43</td>
</tr>
<tr>
<td>7W</td>
<td>-</td>
<td>-</td>
<td>0.43</td>
</tr>
<tr>
<td>6NW</td>
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</tr>
<tr>
<td>5NW</td>
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<td>-</td>
<td>0.40</td>
</tr>
<tr>
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<td>-</td>
<td>-</td>
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</table>

### Table F-43 Weld measurements for the north weld element of specimen T-1.00-23

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<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3NW</td>
<td>0.23</td>
<td>0.25</td>
<td>0.16</td>
</tr>
<tr>
<td>2NW</td>
<td>0.26</td>
<td>0.26</td>
<td>0.20</td>
</tr>
<tr>
<td>1N</td>
<td>0.24</td>
<td>0.26</td>
<td>0.20</td>
</tr>
<tr>
<td>2NE</td>
<td>0.26</td>
<td>0.28</td>
<td>0.19</td>
</tr>
<tr>
<td>3NE</td>
<td>0.25</td>
<td>0.24</td>
<td>0.19</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.25</strong></td>
<td><strong>0.25</strong></td>
<td><strong>0.19</strong></td>
</tr>
</tbody>
</table>

### Table F-44 Weld measurements for the south weld element of specimen T-1.00-23

<table>
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<tr>
<th>Location</th>
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<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3SW</td>
<td>0.26</td>
<td>0.26</td>
<td>0.19</td>
</tr>
<tr>
<td>2SW</td>
<td>0.24</td>
<td>0.28</td>
<td>0.19</td>
</tr>
<tr>
<td>1S</td>
<td>0.24</td>
<td>0.25</td>
<td>0.20</td>
</tr>
<tr>
<td>2SE</td>
<td>0.24</td>
<td>0.24</td>
<td>0.19</td>
</tr>
<tr>
<td>3SE</td>
<td>0.24</td>
<td>0.28</td>
<td>0.19</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.24</strong></td>
<td><strong>0.26</strong></td>
<td><strong>0.19</strong></td>
</tr>
</tbody>
</table>
### Table F-45 Weld measurements for the east weld element of specimen T-1.00-17

<table>
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<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
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<tr>
<td>6SE</td>
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<tr>
<td>6NE</td>
<td>-</td>
<td>-</td>
<td>0.39</td>
</tr>
<tr>
<td>5NE</td>
<td>-</td>
<td>-</td>
<td>0.39</td>
</tr>
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</tbody>
</table>

### Table F-46 Weld measurements for the west weld element of specimen T-1.00-17

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<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>5SW</td>
<td>-</td>
<td>-</td>
<td>0.39</td>
</tr>
<tr>
<td>6SW</td>
<td>-</td>
<td>-</td>
<td>0.39</td>
</tr>
<tr>
<td>7W</td>
<td>-</td>
<td>-</td>
<td>0.39</td>
</tr>
<tr>
<td>6NW</td>
<td>-</td>
<td>-</td>
<td>0.39</td>
</tr>
<tr>
<td>5NW</td>
<td>-</td>
<td>-</td>
<td>0.39</td>
</tr>
<tr>
<td>Average</td>
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<td>-</td>
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</tr>
</tbody>
</table>

### Table F-47 Weld measurements for the north weld element of specimen T-1.00-17

<table>
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<th>Location</th>
<th>Vertical Leg</th>
<th>Horizontal Leg</th>
<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>3NW</td>
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<td>0.37</td>
<td>0.22</td>
</tr>
<tr>
<td>2NW</td>
<td>0.35</td>
<td>0.35</td>
<td>0.25</td>
</tr>
<tr>
<td>1N</td>
<td>0.33</td>
<td>0.37</td>
<td>0.24</td>
</tr>
<tr>
<td>2NE</td>
<td>0.33</td>
<td>0.37</td>
<td>0.24</td>
</tr>
<tr>
<td>3NE</td>
<td>0.33</td>
<td>0.37</td>
<td>0.24</td>
</tr>
<tr>
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<td>0.23</td>
</tr>
</tbody>
</table>

### Table F-48 Weld measurements for the south weld element of specimen T-1.00-17

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<th>Theoretical Throat</th>
</tr>
</thead>
<tbody>
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<td>0.30</td>
<td>0.35</td>
<td>0.24</td>
</tr>
<tr>
<td>2SW</td>
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<td>0.26</td>
</tr>
<tr>
<td>1S</td>
<td>0.33</td>
<td>0.35</td>
<td>0.25</td>
</tr>
<tr>
<td>2SE</td>
<td>0.35</td>
<td>0.35</td>
<td>0.24</td>
</tr>
<tr>
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</tr>
<tr>
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</table>
Appendix F.2: Macroetch Examinations

F.2 Macroetch Examinations

Figure F-1 Macroetch examination for specimen T-0.25-34
Appendix F.2: Macroetch Examinations

Figure F-2 Macroetch examination for specimen T-0.25-23
Appendix F.2: Macroetch Examinations

Figure F-3 Macroetch examination for specimen T-0.25-17
Appendix F.2: Macroetch Examinations

Figure F-4 Macroetch examination of north and south weld elements for specimen T-0.50-34
Appendix F.2: Macroetch Examinations

Figure F-5 Macroetch examination of east and west weld elements for specimen T-0.50-34
Appendix F.2: Macroetch Examinations

Figure F-6 Macroetch examination of north and south weld elements for specimen T-0.50-23
Appendix F.2: Macroetch Examinations

Figure F-7 Macroetch examination of east and west weld elements for specimen T-0.50-23
Appendix F.2: Macroetch Examinations

Figure F-8 Macroetch examination of north and south weld elements for specimen T-0.50-17

(a) North weld element, Section #5 (compression side)

(b) North weld element, Section #6 (compression side)

(c) South weld element, Section #1 (tension side)

(d) South weld element, Section #2 (tension side)

$t_w = 0.257''$
$L_V = 0.410''$
$L_H = 0.474''$

$t_w = 0.237''$
$L_V = 0.431''$
$L_H = 0.468''$

$t_w = 0.270''$
$L_V = 0.433''$
$L_H = 0.419''$

$t_w = 0.271''$
$L_V = 0.454''$
$L_H = 0.421''$
Appendix F.2: Macroetch Examinations

Figure F-9 Macroetch examination of east and west weld elements for specimen T-0.50-17
Appendix F.2: Macroetch Examinations

Figure F-10 Macroetch examination of north and south weld elements for specimen T-0.75-34

(a) North weld element, Section #5 (compression side)

(b) North weld element, Section #6 (compression side)

(c) South weld element, Section #1 (tension side)

(d) South weld element, Section #2 (tension side)

\[ t_w = 0.112" \]
\[ L_V = 0.208" \]
\[ L_H = 0.189" \]

\[ t_w = 0.106" \]
\[ L_V = 0.170" \]
\[ L_H = 0.183" \]

\[ t_w = 0.104" \]
\[ L_V = 0.144" \]
\[ L_H = 0.131" \]

\[ t_w = 0.111" \]
\[ L_V = 0.152" \]
\[ L_H = 0.146" \]
Appendix F.2: Macroetch Examinations

Figure F-11 Macroetch examination of east and west weld elements for specimen T-0.75-34

- (a) East weld element, Section #7
  \[ t_w = 0.084" \]
  \[ L_V = 0.157" \]
  \[ L_H = 0.213" \]

- (b) East weld element, Section #8
  \[ t_w = 0.084" \]
  \[ L_V = 0.157" \]
  \[ L_H = 0.213" \]

- (c) West weld element, Section #3
  \[ t_w = 0.128" \]
  \[ L_V = 0.229" \]
  \[ L_H = 0.174" \]

- (d) West weld element, Section #4
  \[ t_w = 0.134" \]
  \[ L_V = 0.177" \]
  \[ L_H = 0.190" \]
Appendix F.2: Macroetch Examinations

Figure F-12 Macroetch examination of north and south weld elements for specimen T-0.75-23
Appendix F.2: Macroetch Examinations

Figure F-13 Macroetch examination of east and west weld elements for specimen T-0.75-23
Appendix F.2: Macroetch Examinations

(a) North weld element, Section #5 (compression side)

(b) North weld element, Section #6 (compression side)

(c) South weld element, Section #1 (tension side)

(d) South weld element, Section #2 (tension side)

Figure F-14 Macroetch examination of north and south weld elements for specimen T-0.75-17
Figure F-15 Macroetch examination of east and west weld elements for specimen T-0.75-17

(a) East weld element, Section #7

(b) East weld element, Section #8

(c) West weld element, Section #3

(d) West weld element, Section #4
Appendix F.2: Macroetch Examinations

Figure F-16 Macroetch examination of north and south weld elements for specimen T-1.00-34
Appendix F.2: Macroetch Examinations

(a) East weld element, Section #7

(b) East weld element, Section #8

(c) West weld element, Section #3

(d) West weld element, Section #4

Figure F-17 Macroetch examination of east and west weld elements for specimen T-1.00-34
Appendix F.2: Macroetch Examinations

(a) North weld element, Section #5 (compression side)

(b) North weld element, Section #6 (compression side)

(c) South weld element, Section #1 (tension side)

(d) South weld element, Section #2 (tension side)

Figure F-18 Macroetch examination of north and south weld elements for specimen T-1.00-23
Appendix F.2: Macroetch Examinations

Figure F-19 Macroetch examination of east and west weld elements for specimen T-1.00-23
Appendix F.2: Macroetch Examinations

(a) North weld element, Section #5 (compression side)

(b) North weld element, Section #6 (compression side)

(c) South weld element, Section #1 (tension side)

(d) South weld element, Section #2 (tension side)

Figure F-20 Macroetch examination of north and south weld elements for specimen T-1.00-17
Figure F-21 Macroetch examination of east and west weld elements for specimen T-1.00-17
F.3 All-Weld-Metal Tensile Coupon Test Data Forms
PROJECT: Effective Weld Properties of RHS-to-RHS Moment T-Connections

OPERATOR: John MacDonald
DATE: Aug 14 2012

SUPERVISOR: J.A. Packer
TIME: 15:00 - 15:35

SPECIMEN NO: Weld TC-11

GAUGE LENGTH: 7.9/10
WIDTH W:
AFTER RUPTURE: 2.5/8
MACHINED WIDTH:
0.5025
0.5015
AVERAGE/MIN:
0.5025
THICKNESS:

AVERAGE/MIN:
0.5015
SECTION AREA: 0.1980

TEST DATA:

YIELD LOAD $T_y$: $T_{st}$:
STATIC YIELD LOAD $T_{st}$: $T_{u}$:
ULTIMATE LOAD $T_u$: 1722
MAX. STRAIN AT RUPTURE: 282.76
YOUNG'S MODULUS: MPa
STRAIN HARDENING MODULUS: MPa
YIELD STRESS $\sigma_y$: MPa
STATIC YIELD STRESS $\sigma_{st}$: MPa
ULTIMATE STRESS $\sigma_u$: MPa

EQUIPMENT: MTS Testing Machine

CONTROL:
RANGE 1 STRAIN/STROKE/LOAD RATE: 0.00016 in/s
2 STRAIN/STROKE/LOAD RATE: 0.00031 in/s

SUPPLEMENTARY DATA: Small porosity defect area on the fatigue coupon. Failure occurred at a location away from the defect.
## Project: Effective Weld Properties of RHS-to-RHS Moment T-Connections

**Operator:** John MacDonald  
**Date:** Aug 14 2012  
**Supervisor:** J.A. Parker  
**Time:** 15:40 - 16:10

### Tensile Coupon Test Data

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>UT Gauge Length</th>
<th>Width W</th>
<th>Thickness T</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.963 in.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Test Data

- **Yield Load** \( T_y \)  
- **Static Yield Load** \( T_{st1} \)  
- **Ultimate Load** \( T_u \)  
- **Max. Strain at Rupture** 280 με  
- **Young's Modulus** \( E \)  
- **Strain Hardening Modulus** \( E_{sh} \)  
- **Yield Stress** \( σ_y \)  
- **Static Yield Stress** \( σ_{st} \)  
- **Ultimate Stress** \( σ_u \)  

### Equipment

- *MTS Testing Machine*

### Control

- **Range 1**  
  - Strain/Stroke/Load Rate: 0.0001 in/s  
- **Range 2**  
  - Strain/Stroke/Load Rate: 0.0001 in/s

### Supplementary Data:

-
Appendix F.4: All-Weld-Metal Mechanical Properties

### F.4 All-Weld-Metal Mechanical Properties

**Results for All-Weld Metal Tensile Coupons**

- \( F_{y, \text{avg}} = 75.9 \text{ ksi} \)
- \( E_{\text{avg}} = 30277 \text{ ksi} \)
- \( F_{EXX, \text{avg}} = 88.1 \text{ ksi} \)
- \( \varepsilon_{u, \text{avg}} = 285010 \text{ (\( \mu \varepsilon \))} \)

**Coupon \([i]\)**

- \( F_{y, [i]} = 76.2 \text{ ksi} \)
- \( E_{[i]} = 29559 \text{ ksi} \)
- \( F_{EXX, [i]} = 90 \text{ ksi} \)
- \( \varepsilon_{u, [i]} = 291858 \text{ (\( \mu \varepsilon \))} \)

**Coupon \([ii]\)**

- **Coupon \([iii]\)**
Results for All-Weld Metal Tensile Coupons

Results for All-Weld Metal Tensile Coupons

- Coupon [ii]
  - $F_y, [ii] = 75.7$ ksi
  - $E_{[ii]} = 31629$ ksi
  - $F_{EXX, [ii]} = 86.9$ ksi
  - $\varepsilon_{u, [ii]} = 282764$ ($\mu$ε)

- Coupon [iii]
  - $F_y, [iii] = 75.8$ ksi
  - $E_{[iii]} = 29643$ ksi
  - $F_{EXX, [iii]} = 87.5$ ksi
  - $\varepsilon_{u, [iii]} = 280407$ ($\mu$ε)
APPENDIX G  Test Photographs
Appendix G: Test Photographs

(a) East elevation view of failed test specimen
(b) Weld rupture at southeast corner
(c) Weld rupture at southwest corner

Figure G-1 Failed test specimen: T-0.25-34
Appendix G: Test Photographs

(a) East elevation view of failed test specimen

(b) Weld rupture at southeast corner

(c) Weld rupture at southwest corner

Figure G-2 Failed test specimen: T-0.25-23
Appendix G: Test Photographs

(a) East elevation view of failed test specimen

(b) Weld rupture at southeast corner

(c) Weld rupture at southwest corner

Figure G-3 Failed test specimen: T-0.25-17
Appendix G: Test Photographs

(a) East elevation view of failed test specimen
(b) Chord face tearing at southeast corner
(c) Chord face tearing at southwest corner

Figure G-4 Failed test specimen: T-0.50-34
Appendix G: Test Photographs

(a) West elevation view of failed test specimen
(b) Weld rupture at southeast corner
(c) Weld rupture at southwest corner

Figure G-5 Failed test specimen: T-0.50-23
Appendix G: Test Photographs

(a) West elevation view of failed test specimen
(b) Chord face tearing at southeast corner
(c) Chord face tearing at southwest corner

Figure G-6 Failed test specimen: T-0.50-17
Appendix G: Test Photographs

Figure G-7 Failed test specimen: T-0.75-34
Appendix G: Test Photographs

(a) East elevation view of failed test specimen

(b) Weld rupture at southeast corner

(c) Weld rupture at southwest corner

Figure G-8 Failed test specimen: T-0.75-23
Appendix G: Test Photographs

Figure G-9 Failed test specimen: T-0.75-17

(a) East elevation view of failed test specimen

(b) Weld rupture at southeast corner

(c) Weld rupture at southwest corner
Appendix G: Test Photographs

(a) East elevation view of failed test specimen

(b) Weld rupture at southeast corner

(c) Weld rupture along west face

Figure G-10 Failed test specimen: T-1.00-34
Appendix G: Test Photographs

(a) West elevation view of failed test specimen
(b) Weld rupture at southeast corner
(c) Weld rupture at southwest corner

Figure G-11 Failed test specimen: T-1.00-23
Appendix G: Test Photographs

Figure G-12 Failed test specimen: T-1.00-17

(a) East elevation view of failed test specimen

(b) Weld rupture at southeast corner

(c) Weld rupture at southwest corner
APPENDIX H  Moment-Strain Plots
Appendix H: Moment-Strain Plots

Figure H-1 Moment-strain relationship for T-0.25-34: Transverse face, tension side

Figure H-2 Moment-strain relationship for T-0.25-34: Transverse face, compression side
Appendix H: Moment-Strain Plots

Figure H-3 Moment-strain relationship for T-0.25-34: Longitudinal face

Figure H-4 Moment-strain relationship for T-0.25-34: Upper branch (constant stress region)
Appendix H: Moment-Strain Plots

Figure H-5 Moment-strain relationship for T-0.25-23: Transverse face, tension side

Figure H-6 Moment-strain relationship for T-0.25-23: Transverse face, compression side
Appendix H: Moment-Strain Plots

Figure H-7 Moment-strain relationship for T-0.25-23: Longitudinal face

Figure H-8 Moment-strain relationship for T-0.25-23: Upper branch (constant stress region)
Appendix H: Moment-Strain Plots

Figure H-9 Moment-strain relationship for T-0.25-17: Transverse face, tension side

Figure H-10 Moment-strain relationship for T-0.25-17: Transverse face, compression side
Appendix H: Moment-Strain Plots

Figure H-11 Moment-strain relationship for T-0.25-17: Longitudinal face

Figure H-12 Moment-strain relationship for T-0.25-17: Upper branch (constant stress region)
Appendix H: Moment-Strain Plots

Figure H-13 Moment-strain relationship for T-0.50-34: Transverse face, tension side

Figure H-14 Moment-strain relationship for T-0.50-34: Transverse face, compression side
Figure H-15 Moment-strain relationship for T-0.50-34: Longitudinal face
Appendix H: Moment-Strain Plots

Figure H-16 Moment-strain relationship for T-0.50-23: Transverse face, tension side

Figure H-17 Moment-strain relationship for T-0.50-23: Transverse face, compression side
Figure H-18 Moment-strain relationship for T-0.50-23: Longitudinal face

Figure H-19 Moment-strain relationship for T-0.50-23: Upper branch (constant stress region)
Figure H-20 Moment-strain relationship for T-0.50-17: Transverse face, tension side

Figure H-21 Moment-strain relationship for T-0.50-17: Transverse face, compression side
Figure H-22 Moment-strain relationship for T-0.50-17: Longitudinal face

Figure H-23 Moment-strain relationship for T-0.50-17: Upper branch (constant stress region)
Figure H-24 Moment-strain relationship for T-0.75-34: Transverse face, tension side

Figure H-25 Moment-strain relationship for T-0.75-34: Transverse face, compression side
Figure H-26 Moment-strain relationship for T-0.75-34: Longitudinal face

Figure H-27 Moment-strain relationship for T-0.75-34: Upper branch (constant stress region)
Figure H-28 Moment-strain relationship for T-0.75-23: Transverse face, tension side

Figure H-29 Moment-strain relationship for T-0.75-23: Transverse face, compression side
Appendix H: Moment-Strain Plots

Figure H-30 Moment-strain relationship for T-0.75-23: Longitudinal face

Figure H-31 Moment-strain relationship for T-0.75-23: Upper branch (constant stress region)
Appendix H: Moment-Strain Plots

Figure H-32 Moment-strain relationship for T-0.75-17: Transverse face, tension side

Figure H-33 Moment-strain relationship for T-0.75-17: Transverse face, compression side
Appendix H: Moment-Strain Plots

Figure H-34 Moment-strain relationship for T-0.75-17: Longitudinal face

Figure H-35 Moment-strain relationship for T-0.75-17: Upper branch (constant stress region)
Figure H-36 Moment-strain relationship for T-1.00-34: Transverse face, tension side

Figure H-37 Moment-strain relationship for T-1.00-34: Transverse face, compression side
Appendix H: Moment-Strain Plots

Figure H-38 Moment-strain relationship for T-1.00-34: Longitudinal face

Figure H-39 Moment-strain relationship for T-1.00-34: Upper branch (constant stress region)
Appendix H: Moment-Strain Plots

Figure H-40 Moment-strain relationship for T-1.00-23: Transverse face, tension side

Figure H-41 Moment-strain relationship for T-1.00-23: Transverse face, compression side
Appendix H: Moment-Strain Plots

Figure H-42 Moment-strain relationship for T-1.00-23: Longitudinal face

Figure H-43 Moment-strain relationship for T-1.00-23: Upper branch
Appendix H: Moment-Strain Plots

Figure H-44 Moment-strain relationship for T-1.00-17: Transverse face, tension side

Figure H-45 Moment-strain relationship for T-1.00-17: Transverse face, compression side
Appendix H: Moment-Strain Plots

Figure H-46 Moment-strain relationship for T-1.00-17: Longitudinal face

Figure H-47 Moment-strain relationship for T-1.00-17: Upper branch (constant stress region)
APPENDIX I  Branch Deflection and Chord Deformation Profiles
I.1 Branch Deflection Profiles

Figure I-1 Branch deflection profile for T-0.25-34

Figure I-2 Branch deflection profile for T-0.25-23
Appendix I.1: Branch Deflection Profiles

Figure I-3 Branch deflection profile for T-0.25-17

Figure I-4 Branch deflection profile for T-0.50-23
Appendix I.1: Branch Deflection Profiles

Figure I-5 Branch deflection profile for T-0.50-17

Figure I-6 Branch deflection profile for T-0.75-34
Figure I-7 Branch deflection profile for T-0.75-23

Figure I-8 Branch deflection profile for T-0.75-17
Appendix I.1: Branch Deflection Profiles

Figure I-9 Branch deflection profile for T-1.00-34

Figure I-10 Branch deflection profile for T-1.00-23
Figure I-11 Branch deflection profile for T-1.00-17
I.2 Chord Deformation Profiles

Figure I-12 Chord face deformation profile for T-0.25-34

Figure I-13 Chord face deformation profile for T-0.25-23
Appendix I.2: Chord Deformation Profiles

Figure I-14 Chord face deformation profile for T-0.25-17

Figure I-15 Chord face deformation profile for T-0.50-23
Appendix I.2: Chord Deformation Profiles

Figure I-16 Chord face deformation profile for T-0.50-17

Figure I-17 Chord face deformation profile for T-0.75-34
Appendix I.2: Chord Deformation Profiles

Figure I-18 Chord face deformation profile for T-0.75-23

Figure I-19 Chord face deformation profile for T-0.75-17
Appendix I.2: Chord Deformation Profiles

Figure I-20 Chord sidewall deformation profile for T-1.00-34

Figure I-21 Chord sidewall deformation profile for T-1.00-23
Appendix I.2: Chord Deformation Profiles

Figure I-22 Chord sidewall deformation profile for T-1.00-17