


conference preview

WELDING OF HOLLOW STRUCTURAL SECTIONS

BY JEFFREY A. PACKER, P.ENG., PH.D., D.SC.,
AND MATTHEW R. MCFADDEN

 **WHILE WELDING** might seem like a standardized process across the board, there are definite logistical differences between welding W-shapes and hollow structural sections (HSS). Recognizing these differences and the inherent challenges and characteristics of welding HSS can result in a much more efficient operation.

The first thing to consider is that unlike open sections, where welding is typically possible from both sides of an element, welding of HSS is only possible from one side, thus requiring larger weld sizes. Second, the main HSS member face to which a branch is welded is generally much more flexible than its wide flange counterpart; the two webs of the main member, which act as stiffeners, are at the outside of the connection rather than in the middle, as would be the case with a W-shape web. This increased flexibility of the connecting face tends to cause an uneven load distribution in the welded joint.

Member Selection

The ideal size or type of HSS to be used depends on the type of connection. For truss-type connections, the branch width-to-chord width ratio (β) should be relatively high (say 0.7 to 0.8), but still preferably enable the branch to sit on the “flat” of the main member if it is a square/rectangular HSS. An exception to this recommendation is the use of connections in Vierendeel frames, where matched-width HSS ($\beta \approx 1.0$) are typically necessary in order to achieve full moment capacity. In addition, the branch thickness-to-chord thickness ratio (β) should be relatively low. Something less than unity is preferable, with a value of 0.5 being a good target. These conditions will generate a truss-type connection with a high static strength as well as high fatigue resistance. As member selection is intimately tied to connection capacity, and most HSS connections are required to be unreinforced, it is clear that checking the connection capacity is the responsibility of the structural engineer.

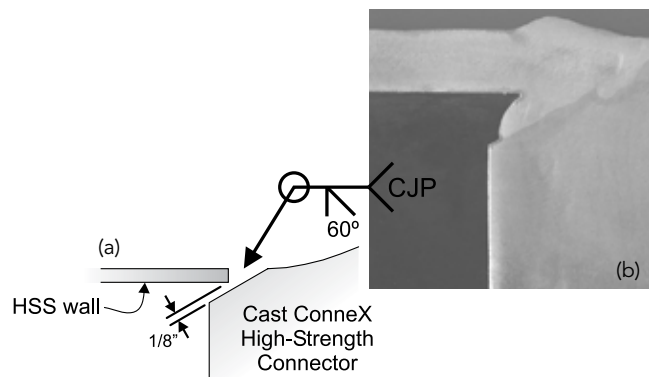
Weld Types

Three basic types of welds account for practically all structural weld joints, and HSS are no exception: complete-joint-penetration (CJP) groove welds, partial-joint-penetration (PJP) groove welds and fillet welds. But again, HSS welding comes with its own challenges that need to be taken into consideration with each of these three welding types.

An overview of welded HSS connections from several different perspectives.

Complete-joint-penetration groove welds (from one side and without backing) are extremely expensive, require specially qualified welders and should almost never be specified for HSS connections. One exception that comes to mind is for a round HSS welded to a proprietary special-purpose steel casting—e.g., the High-Strength Connector by Cast ConneX Corp., used with diagonal HSS braces in seismic load-resisting braced frames, shown in Figure 1. In this case the tapered nose of the casting inserted into the HSS essentially serves as backing.

Partial-joint-penetration groove welds are an option for HSS connections, especially if fillet weld sizes become large (leg sizes greater than ½ in.) and the branch member is reasonably thick. Prequalified joint details for PJP welds to HSS, particularly for the longitudinal welds in “matched box” connections as in Figure 2, are given in AWS D1.1, *Structural Welding Code—Steel* (2010).



▲ Fig. 1: An illustrated (a) and macro-etched (b) CJP joint between round HSS and a Cast ConneX High-Strength Connector.

► Fig. 2: A macro-etched PJP groove weld in a matched-width ($\beta \approx 1.0$) HSS connection.



Fillet welds, being the least expensive and easiest weld type, are the preferred and most common weld type for HSS connections. The design of fillet welds in structural steel buildings in the U.S. is governed by ANSI/AISC 360-10 Table J2.5 and is based on the limit state of shear failure of the weld using a matching (or under-matching) filler metal. For a simple 90° T-joint the LRFD resistance of a single weld is given by:

$$\Phi R_n = \Phi F_{nw} A_{we} = (0.75)(0.60 F_{EXX})(D/\sqrt{2})(\text{weld length}),$$

where D = weld leg size.

The design of fillet welds in Canada is governed by Clause 13.13.2.2 in *Design of Steel Structures*, CSA S16-09, and although different coefficients are used, an identical resistance is obtained. Both AISC and CSA allow an enhancement to the nominal strength of the weld metal of $1.0 + 0.50 \sin 1.5\theta$ for welds loaded at an angle of θ degrees to the weld longitudinal axis, and include some further provisions for weld groups. ANSI/AISC 360-10, however, limits the $\sin\theta$ enhancement factor to only weld groups where all elements are in a line or are parallel, also referred to as linear weld groups. Thus, the apparent inapplicability of this factor to HSS T-, Y- and K-connections is pointed out in AISC *Steel Design Guide No. 24, Hollow Structural Section Connections*. The CSA standard, on the other hand, does not rule out the applicability of the $\sin\theta$ factor for HSS connections, leading to a much greater resistance for a fillet weld group in an HSS connection, and hence much smaller weld sizes (see Table 1). The prior edition, CAN/CSA S16-01, included a check for shearing of the base metal at the edge of a fillet weld along the fusion face (see Figure 3), which frequently governed and thus resulted in generally larger weld sizes at that time.

Weld Design

Weld design for HSS-to-HSS connections can follow either of these two design philosophies:

1. The weld may be proportioned so that it develops the yield strength of the connected branch wall at all locations around the branch.
2. The weld may be proportioned to resist the applied branch forces, with adjustments for uneven stress distributions along the length of the weld.

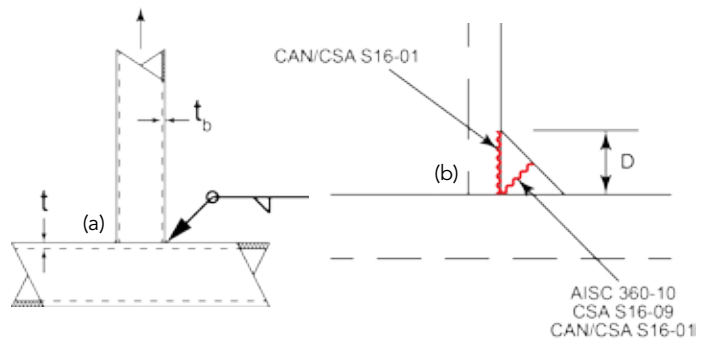
In terms of the first method, this will represent an upper limit on the weld size, and hence a conservative design procedure. For example, consider the simple 90° HSS-to-

HSS T-connection under branch axial tension load in Figure 3, with sections manufactured to ASTM A500 Grade C and fillet welded with E70 electrodes. In this case, all welds are oriented transversely (at 90°) to the applied load, forming a non-linear weld group, and one can consider that the HSS branch wall yield resistance, *per unit length*, is given by:

$$(\Phi = 0.9)F_y t_b = 45 t_b \text{ kips/in.},$$

where t_b is the branch wall thickness.

Note that various codes/specifications have different fillet weld effective throat size requirements to develop this branch wall resistance (see Table 1).



▲ Fig. 3: A 90° HSS T-connection under branch axial tension (a) and detail of the fillet weld showing assumed failure planes (b).
▼ Table 1: Comparison of fillet weld effective throats theoretically necessary to develop the yield resistance of the connected branch member wall, based on branch wall thickness t_b .

Specification or Code	Weld Effective Throat
ANSI/AISC 360-10 Table J2.5	$1.43t_b$
AWS D1.1/D1.1M: 2010 Clause 2.25.1.3 and Fig. 3.2	$1.07t_b$
CSA S16-09 Clause 13.13.2.2	$0.95t_b$
CAN/CSA S16-01 Clause 13.13.2.2	$1.14t_b$
CEN (2005)* or IIW (2009)	$1.10t_b$

*Eurocode 3: *Design of Steel Structures—Part 1-8: Design of Joints*, EN1993-1-8:2005(E), European Committee for Standardization, Brussels, Belgium.

Jeffrey A. Packer, P.Eng., Ph.D., D.Sc., is the Bahen/Tanenbaum Professor of Civil Engineering, and **Matthew R. McFadden** is a research assistant in the Department of Civil Engineering, both at the University of Toronto.



Packer



McFadden

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The second method, essentially a “fit for purpose” approach, involves taking weld effective lengths into account because HSS welded joints typically have highly varying load distributions around their perimeter. For joints with relatively low branch forces, the use of weld effective lengths may lead to smaller weld sizes and result in a more economical weld design. The same effective weld size should still be maintained all around the attached branch, with the entire branch perimeter welded. (An exception to the latter may apply to the “hidden weld” in HSS-to-HSS overlapped connections.) Some HSS weld effective lengths were introduced into AWS D1.1 in the 1990s, based on prior experimental research, then AISC 360 adopted these in 2005 and further expanded the coverage in Section K4 of AISC 360-10. *Static Design Procedure for Welded Hollow Section Joints—Recommendations*, 3rd edition, published in 2009 by the International Institute of Welding (IIW), Paris, specifically acknowledges the effective length concept for weld design, but like all other steel design specifications/codes (except AISC 360 and AWS D1.1) does not prescribe any effective lengths.

An AISC-sponsored experimental research project is currently being performed by the authors on weld-critical HSS-to-HSS T-connections, under branch in-plane bending and on weld-critical HSS-to-HSS overlapped K-connections within complete trusses; this is being done in order to validate or further improve the HSS weld effective rules added to Section K4 of AISC 360-10. The heightened interest in welding of HSS is also reflected in the recent formation of an AWS Tubular Task Group, at the instigation of the American Association of State Highway and Transportation Officials (AASHTO), primarily to address tubular bridge construction. **MSC**

This article provides a preview of some of what the authors will present in Session N13 at NASCC: The Steel Conference, April 18-20 in Dallas. Learn more about The Steel Conference at www.aisc.org/nascc.