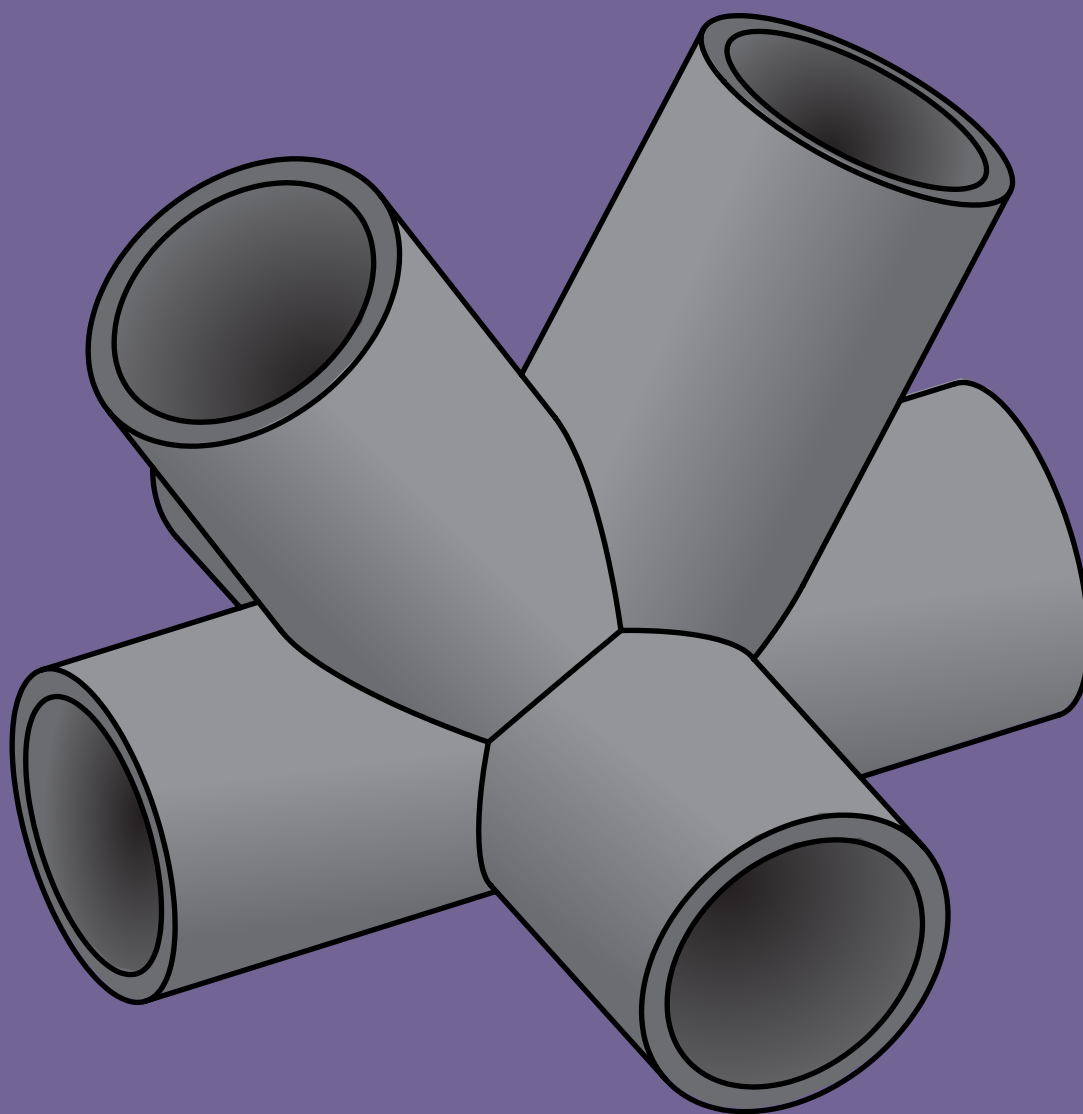




Design Guide 24

Hollow Structural Section Connections

Second Edition



**Smarter.
Stronger.
Steel.**



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Hollow Structural Section Connections

Second Edition

Jeffrey A. Packer, PEng, PhD, DSc
Kimberley Olson, PE

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Printed in the United States of America

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Acknowledgments

The authors extend their appreciation to Professor Jason McCormick, University of Michigan, for his involvement in the initial phases of this Design Guide project and his contributions to Chapter 4. The assistance and oversight of the following AISC staff members and reviewers are gratefully acknowledged:

Eric Bolin
Michael Desch
Carol Drucker
Cindi Duncan
Matthew Eatherton
Jim Fisher

Brad Fletcher
Steve Herlache
Steve Herth
Jack Kenney
Larry Kruth
Conner Maines

Margaret Matthew
Larry Muir
Tom Schlafly
Don Sherman
Victor Shneur

Preface

The first edition of this Design Guide, by Jeffrey Packer, Donald Sherman, and Maura Lecce, was published by AISC in 2010 and conformed to the 2005 AISC *Specification for Structural Steel Buildings*, as well as the 13th (2005) edition of the AISC *Steel Construction Manual*. This 2nd edition of the Design Guide has been written in accordance with the 2022 AISC *Specification* and the 16th (2023) edition of the AISC *Manual*. While the Design Guide is still directed at connections to, or between, hollow structural section (HSS) members under predominantly static loading, the content has been considerably expanded and updated, from nine chapters and 146 pages to ten chapters and 344 pages. The presentation of this Design Guide is particularly focused on connection limit states, contains an extensive review of background literature, and supports the application of AISC *Specification* Chapters J and K and the AISC *Manual* to HSS connections. A total of 33 design examples are given, in both load and resistance factor design (LRFD) and allowable strength design (ASD) methods.

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Chapter 1

Introduction

Nature provides many examples of tubular sections, from the hollow bones of birds to reeds, to a naturally occurring hollow structural section (HSS) still used in building construction in parts of the world today—bamboo. However, the Forth Bridge (Figure 1-1), a cantilever structure completed in 1890 with circular steel members formed from rolled steel plates and riveted together on site, is generally attributed to be the first notable tubular steel structure. The construction of offshore structures for oil and gas platforms then produced significant advances in design and fabrication technology for steel tubes. Later still, the era of manufactured HSS, produced for structural purposes, really commenced in England in 1952 with the development of the rectangular hollow section by Stewarts and Lloyds. The product soon proliferated worldwide, and its use has progressively expanded. Traditionally employed in exposed structures for aesthetic reasons, HSS are now selected for routine structural applications because of their many other attributes. HSS can be found in atria, pavilions, transportation hubs, arenas, exhibition halls, convention centers, schools, hospitals, big-box stores, low- to medium-rise buildings, industrial structures, towers, masts, sign supports, pedestrian bridges, amusement

rides, canopies, and conveyors. Some contemporary international examples are shown in Figures 1-2 through 1-6.

1.1 SCOPE

Demonstrated in this Design Guide are design methods for a wide range of HSS welded and bolted connections, in structures subject to predominantly static loading. Design procedures, where such are covered, conform to the American Institute of Steel Construction (AISC) *Specification for Structural Steel Buildings* (AISC, 2022c) and the AISC 16th Ed. *Steel Construction Manual* (AISC, 2023b), hereafter referred to as the AISC *Specification* and AISC *Manual*, respectively. The 2022 AISC *Specification* and 16th Ed. AISC *Manual* supersede all previous editions. The connection types covered in this Design Guide exceed those described in the preceding AISC documents. In such cases, other contemporary, international-consensus design recommendations for HSS connections (from Section 1.1.1), other U.S. or national standards, or published research are utilized. The Design Guide deals with connections to unfilled HSS, connections that are generally unreinforced and connections



Fig. 1-1. Forth railway bridge, Edinburgh, Scotland.

that have symmetry perpendicular to the connection plane (i.e., elements positioned on the chord centerline). All design examples are presented in both load and resistance factor design (LRFD) and allowable strength design (ASD) formats. Design examples for reinforced HSS connections are beyond the scope of this Design Guide. Connections subjected to seismic loading that need to conform to the AISC *Seismic Provisions for Structural Steel Buildings* (AISC,

2022b) are not covered, but the suitability of some materials for seismically loaded structures is discussed in this chapter.

1.1.1 Prominent Design Resources for HSS Connections

Research on HSS connections has been actively pursued since the 1960s, and periodic research summaries are available (e.g. Wardenier, 1982; Giddings and Wardenier, 1986).

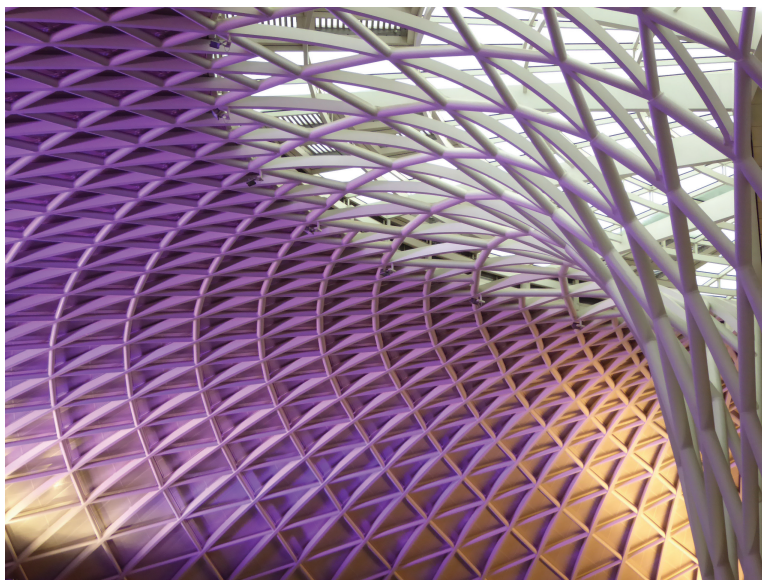


Fig. 1-2. Roof of King's Cross railway station, London, England.



Fig. 1-3. Pritzker Pavilion trellis, Millennium Park, Chicago, Illinois.

The world's first HSS connections manual was produced in Canada by Stelco (1971) followed by a larger second edition (Stelco, 1981), which was the first English-language HSS design guide to be expressed in a limit states format. The Canadian Institute of Steel Construction (CISC) then assumed the mantle in North America with two HSS design guides (Packer and Henderson, 1992, 1997). The publication of the AISC *Specification for the Design of Steel Hollow Structural Sections* (AISC, 1997a), which was later amalgamated into the AISC *Specification*, represented a new landmark in the United States. In coordination with the AISC

Specification, AISC and the Steel Tube Institute (STI)—a North American trade association formed in 1930—published the first U.S. HSS connections manual (AISC, 1997b).

The International Institute of Welding (IIW) Subcommittee XV-E on “Tubular Structures” has, since 1964, played a major role in assessing and assimilating HSS connection design knowledge into specification format. To date, three editions of design recommendations for statically loaded, welded, HSS connections have been published by IIW (1981, 1989, 2012). It should be noted that the second edition (1989) is the basis for most of the provisions in the



Fig. 1-4. Warren truss pedestrian bridge, Ontario Place, Toronto, Canada.



Fig. 1-5. Opryland Hotel, Nashville, Tennessee.

AISC *Specification* and is similarly the basis for most of the provisions published in Eurocode 3 (CEN, 2005) and in the CISC design guide (Packer and Henderson, 1997), the latter being still widely used throughout North America. The third edition (IIW, 2012) incorporated recent research findings, introduced the concept of a deformation limit as a connection ultimate limit state, and was then adopted as an international model code (ISO, 2013).

From 1962, the Comité International pour le Développement et l'Étude de la Construction Tubulaire (CIDECT), which is an association of international HSS producers with the aim of collectively developing the market for manufactured tubing, has sponsored and coordinated much worldwide research on HSS. Their work has been summarized in ten design guides and two illustrated books, the latter dealing with HSS architecture (Eekhout, 2011) and structural design (Wardenier et al., 2010). CIDECT Design Guide No. 1 dealing with round HSS connections (Wardenier et al., 1991, 2008a), No. 3 dealing with rectangular HSS connections (Packer et al., 1992, 2009), and No. 9 dealing with column connections (Kurobane et al., 2004) are particularly relevant to the topics of this Design Guide and are valuable resources. The first editions of CIDECT Design Guides 1 (Wardenier et al., 1991) and 3 (Packer et al., 1992) were in accord with the IIW second edition (1989) recommendations. The second editions of CIDECT Design Guides 1 (Wardenier et al., 2008a) and 3 (Packer et al., 2009), IIW (2012), ISO (2013) and the CIDECT book (Wardenier et al., 2010), are all harmonized.

Although beyond the scope of this Design Guide, it might be noted that fatigue design of welded HSS connections is well covered by CIDECT Design Guide No. 8 (Zhao et al., 2001). This CIDECT Design Guide was developed from IIW

(2000) recommendations, which were also adopted as an international model code (ISO, 2008). In addition to welding procedures, the American Welding Society AWS D1.1/D1.1M code (AWS, 2020) supplies design guidance on the fatigue strength of hollow section connections.

STI is another technical resource in North America for HSS and steel tube products. STI offers resources for HSS design, including HSS Design Manuals (four volumes) (STI, 2015, 2016a, 2016b, 2016c), a capability tool showing HSS section availability, HSS Connex software, and HSS limit state tables.

1.2 PRINCIPLES OF HSS CONNECTION DESIGN

A large proportion of structures involving HSS have exposed members, and it is a frequent architectural choice to have unreinforced connections (i.e., without stiffeners). Furthermore, reinforcement is usually a difficult and expensive option with HSS so, whenever possible, design should avoid such an outcome. Although this is not usually feasible with connections governed by seismic loading considerations, it is generally achievable with statically loaded connections, provided the members themselves are selected in an astute manner. Wide-flange sections have a natural “stiffener” (the web) at the center of the flange. The mass of wide-flange members tends to be concentrated at the flanges, and wide-flange sections are available in greater thicknesses than HSS. By contrast, HSS have the web elements at the edges of the flange and the walls tend to be relatively thin, resulting in a softer, more-flexible connection. The method of force transfer is thus inherently different (see AISC *Specification* Commentary Figure C-K1.3) and more demanding for achieving unstiffened connections to HSS (Figure 1-7).



Fig. 1-6. Melrose Arch Galleria, Johannesburg, South Africa.

To maximize the strength and stiffness of HSS connections, the through member—the chord in a truss-type connection or the column in a beam-to-column or bracing-to-column connection—should be relatively thick. This concept is often contrary to a designer’s natural inclination, especially for compression members, to select a larger-size thin-walled section in order to maximize the radius of gyration (and hence the compressive capacity). For the compression chord of a truss, the ideal width-to thickness ratio of a square HSS would usually be in the range of 15 to 25, while the ideal diameter-to-thickness ratio of a round HSS would usually be in the range of 15 to 30. A corollary of this through-member thickness rule is that the branch member should be relatively thin. Hence the adage “weld thin to thick” still holds. An ideal branch-to-chord thickness ratio will usually be around 1:2. These principles for maximizing the strength of statically loaded HSS-to-HSS connections also apply to fatigue-critical HSS-to-HSS connections.

If the branch is relatively thin in an HSS-to-HSS connection, then it implies that the outside dimensions will be relatively large in order to provide sufficient cross-sectional area to resist the branch force. Fillet welding is the less-expensive welding procedure; therefore, branches should still be sized to suit this welding technique whenever possible. For square and rectangular HSS chord members, fillet welding of the branches is possible if they land on the “flat” of the chord member. Because the outside corner radius of most square/rectangular HSS is typically around twice the wall thickness, t , this means that the branch width should be slightly less than the HSS width minus $4t$. The section property data for square and rectangular HSS produced by electric resistance welding (ERW) are based upon an outside corner radius of

two times the design wall thickness. The section property data for large-size square and rectangular HSS produced by submerged arc welding (SAW) are based upon an outside corner radius of three times the design wall thickness. These production methods are discussed further in Section 1.3.2, with relevant manufacturing standards in Section 1.4.

One should always bear in mind that HSS member selection will determine connection capacity and stiffness and that optimization for minimum weight does not result in minimum cost. In truss constructions, some other rules of thumb include:

- The number of connections should be minimized (hence favoring Warren trusses) to reduce fabrication costs.
- The number of different member sizes should be kept low to encourage repetition.
- The chord member should be kept continuous through all connections, and gapped K-connections should be preferred to those with overlapped branches for economy in fabrication. (The terminology for Warren truss K-connections, gapped versus overlapped, is defined in Section 9.2).
- If overlapped K-connections are chosen, perhaps as a means of increasing the strength beyond that of a gapped connection, fabrication is easiest with 100% overlapping branches in which one branch sits entirely on another branch.

From the foregoing, it is apparent that HSS designers must understand, and be influenced by, connection design and fabrication criteria at the structural modeling and member selection stages, and not endeavor to resolve connection issues after the members have been chosen. This is especially pertinent if connection design is delegated, per Section 3.2.3 of the *AISC Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2022a), hereafter referred to as the *AISC Code of Standard Practice*.

1.3 MANUFACTURING OF HOLLOW SECTIONS

1.3.1 Terminology and Designations

Around the world, manufactured hollow steel sections have become known by a number of acronyms, such as structural hollow sections (SHS), rectangular hollow sections (RHS), square hollow sections (SHS), circular hollow sections (CHS), and elliptical hollow sections (EHS). In the United States, the term structural tubing (TS) was used until 1997 when the Canadian term hollow structural section (HSS) was adopted across North America (AISC, 1997a, 1997b). Thus, from 1997 the designation on drawings and for procurement changed from TS to HSS. The designation used to identify square and rectangular HSS is, for example:

HSS8×4×¼

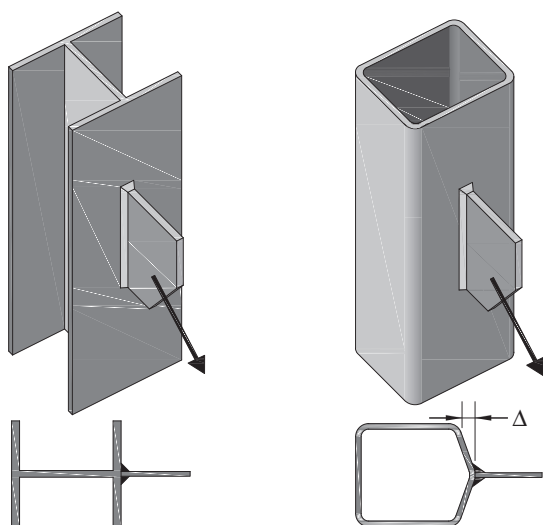


Fig. 1-7. Effect of out-of-plane loading on the flange element for wide-flange versus HSS members.

In this designation, the whole numbers are the long side, followed by the short side, followed by the nominal thickness as a fraction, all in inches. Decimal numbers are used for the outside diameter and nominal thickness in the designation of round HSS, with both expressed to three decimal places. For example:

HSS6.000×0.375

Pipe is sometimes used in lieu of round HSS and the designation for pipes, up to and including NPS (nominal pipe size) 12, takes the following form: Pipe, nominal diameter (in.), and weight class, which can be Standard (Std.), Extra Strong (x-Strong), or Double Extra Strong (xx-Strong). Pipes with wall thicknesses that do not correspond to the foregoing weight classes are designated by pipe, outside diameter (in.), and wall thickness (in.), with both dimensions expressed to three decimal places. Examples of pipe designations are:

Pipe 8 x-Strong

Pipe 5.563×0.500

The dimensions and geometric properties of most HSS produced to ASTM A500/A500M (ASTM, 2021b) are included in AISC *Manual* Tables 1-11 (for rectangular HSS), 1-12 (for square HSS), and 1-13 (for round HSS). Dimensions and geometric properties for pipe are given in AISC *Manual* Table 1-14. These properties in the AISC *Manual* are also available as an Excel file from the AISC website under “Shapes Database.” Dimensional tolerances are also included in AISC *Manual* Tables 1-27 (for rectangular and square HSS) and 1-28 (for round HSS and pipe). HSS are produced to specifications other than ASTM A500/A500M, including ASTM A53/A53M (ASTM, 2022b), ASTM A501/501M (ASTM, 2021f), ASTM A847/847M (ASTM, 2021d), ASTM A1065/1065M (ASTM, 2018a), ASTM A1085/1085M (ASTM, 2015), ASTM A1110/A1110M (ASTM, 2021c), ASTM A1112/A1112M (ASTM, 2018b), ASTM A618/A618M (ASTM, 2021g), and CSA G40 (CSA, 2018a), which are discussed more fully in Section 1.4. Section properties for HSS products conforming to ASTM A847/847M follow ASTM A500/A500M, and ASTM A1085/A1085M section properties are also available from the AISC website. It is important that the correct mechanical and geometric properties are used for design, depending upon the product chosen, and in particular the correct “design thickness.” The AISC website provides nominal availability, by producer, for any particular HSS size.

1.3.2 Production Methods

The principal methods used to make hollow steel sections are (1) cold-formed from round with ERW, (2) cold-formed directly to square/rectangular with ERW, (3) cold-formed directly to square/rectangular with SAW, (4) hot-formed or hot-finished, and (5) fabricated box sections from plates.

Cold-Formed from Round with ERW

In this continuous-forming process, a flat strip is progressively bent around its longitudinal axis by passing the strip through a series of rolls to form an open-seam round. The open-seam round is joined into a closed round section by passage through a high-frequency induction heating coil that fuses the open tips together, without the addition of any filler metal, producing a continuous longitudinal seam weld. The stages of this process are illustrated in Figure 1-8. The weld bead (flash) is then removed from the outside of the tube, the weld seam is cooled, and the tube is then run through additional sizing and shaping rolls to finish the product as a round, square, or rectangular HSS. Most manufacturers can automatically cut the finished HSS to prescribed lengths at the mill to suit customer orders. The weld flash on the inside of the HSS is normally left in place, but some manufacturers can remove this during the production process by prior arrangement. HSS are not made with telescoping tubes in mind, but this is one application that requires removal of the inner weld flash. Cold forming from round with ERW is the most common production process in North America, with HSS produced to ASTM A53/A53M, ASTM A500/A500M, ASTM A847/847M, ASTM A1085/A1085M, ASTM A1110/A1110M, and CSA G40.

Cold-Formed Directly to Square/Rectangular with ERW

In this continuous-forming process, driven dies progressively shape the flat strip directly into an open-seam square or rectangular shape. The open-seam shaped section is then joined

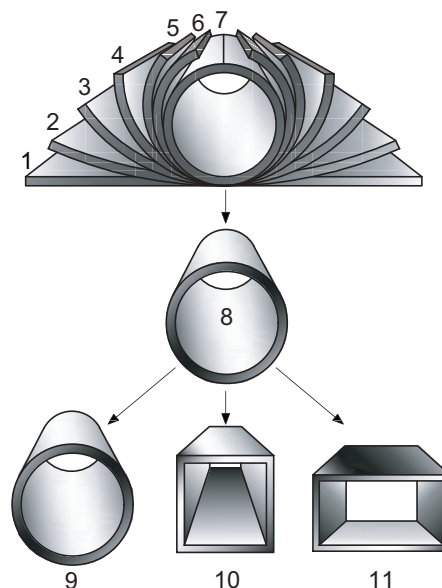


Fig. 1-8. Production steps for HSS cold-formed from round with ERW.

into a closed square or rectangular HSS by passage through a high-frequency induction heating coil that fuses the open tips together, without the addition of any filler metal, producing a continuous longitudinal seam weld with the flash removed on the outside. The stages of this process are illustrated in Figure 1-9. After cooling of the weld seam, the HSS is driven through a series of sizing and shaping rolls to qualify the final dimensions. Large amounts of cold working are performed on the corners, but considerably less on the flats, by this forming process. Typical residual stresses and tensile and compressive behavior for this product are described by Sun and Packer (2014a). Cold forming directly to square/rectangular with ERW is a less-common production process in North America, and HSS can be produced in accordance with ASTM A500/A500M, ASTM A847/A847M, ASTM A1085/A1085M, and ASTM A1112/A1112M. High yield strength HSS (up to 110 ksi) is available by this production method.

Cold-Formed Directly to Square/Rectangular with SAW

With this process, two identical pieces of flat strip are brake-formed to make two identical U-shaped sections, which represent the two halves of an eventual square or rectangular HSS. These half-sections are then joined together with two longitudinal SAW and the addition of weld metal, with the aid of continuous backing bars tack welded to each leg on the inside of one half-section. The seam welds are generally partial-joint-penetration (PJP) groove welds, with fusion through at least 80% of the wall thickness. A protruding crown to the seam weld may be left on the outside of the HSS, but this is not to exceed $\frac{1}{8}$ in. in height above the flat face. Cold forming directly to square or rectangular with SAW is a unique process, suited to large-size sections, with HSS being produced in conformance with ASTM A1065/A1065M.

Hot-Formed or Hot-Finished

This process involves making various shapes of finished hollow sections, commencing with a round shape produced by cold-forming with ERW or by hot-forming seamless tubes. However, a unique feature of this complete process involves

heating (or reheating) the closed tube to above the upper critical transformation temperature to achieve full normalizing and finishing to the final shape in this condition. This produces a fine-grain metallurgy and good toughness with very low residual stresses and uniform mechanical properties around the cross section. Round, square, rectangular, and elliptical HSS are produced in this manner, with the square and rectangular HSS having outside corner radii that are much smaller than their cold-formed counterparts. Structural tubing is not manufactured in North America by this process, but it is available in Europe to the standard EN 10210 (CEN, 2006, 2019). Certain grades comply with ASTM A501/A501M or ASTM A618/A618M, which are the U.S. standards to which such HSS are imported to North America. These hollow sections might be specified for special situations where the metallurgical properties are desired, elliptical shapes are required (with an aspect ratio of 2:1 available), or very thick-walled round HSS are needed. This production method is not to be confused with HSS manufactured by cold-forming then subjected to heat treatment at 850°F (which provides partial relief of residual stresses), as available with ASTM A1085/A1085M Supplement S1 (ASTM, 2015) or CSA G40 Class H (CSA, 2018a) and discussed further in Section 1.4.2.

Fabricated Box Sections from Plates

Fabricated box sections are not HSS, the latter being defined as being produced in accordance with a pipe or tubing product specification by the AISC *Specification* Glossary. Box sections are formed by welding four plates together at the corners, using either fillet, PJP, or complete-joint-penetration (CJP) groove welds. A common application is in large-size columns. More discussion about the welding criteria for such corner joints in built-up hollow sections is given in AISC Design Guide 21 (Miller, 2017). Box sections are included in this list of hollow section types because the limit states for HSS connections can also be applied to box section connections. AISC *Specification* Chapter K is therefore titled “Additional Requirements for HSS and Box Section Connections,” with the scope of application applying to box sections of uniform wall thickness (like HSS). Importantly,

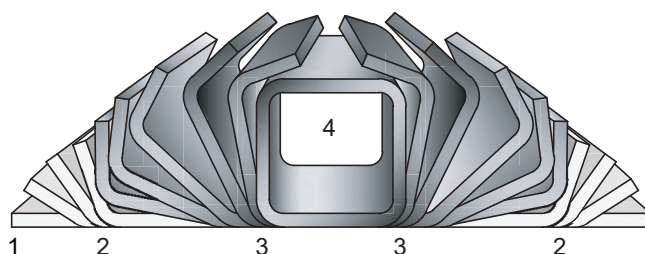


Fig. 1-9. Production steps for HSS cold-formed directly to square/rectangular with ERW.

the seam welds between box-section elements (the corner joints) are required to be CJP groove welds in the connection region—if not elsewhere along the member length—so that the chord or column (through) member integrity is not compromised in the connection region.

1.4 MANUFACTURING STANDARDS AND PROPERTIES

This section reviews the important characteristics of various production standards for HSS that may be found in North America. Production standards all cite permissible tolerances on geometric properties such as length, straightness, wall flatness, squareness, and twist, but the features that are discussed in the following section are those that affect material selection and engineering design, with particular focus on the mechanical and cross-sectional properties. A broader commentary beyond North American production specifications is available by Packer et al. (2010a).

1.4.1 ASTM A500/A500M

This is the most common manufacturing standard, with HSS cold-formed either from round with ERW or directly to square or rectangular with ERW. Round, square, and rectangular shapes are produced with peripheries up to 88 in. and nominal wall thicknesses up to 1 in. A lower tolerance of -10% is permitted on wall thickness, with no tolerance on mass, weight, or cross-sectional area, resulting in HSS being produced routinely undersized because manufacturers interpret tolerances as allowances. The AISC *Specification* accounts for this practice by designating a design wall thickness of 0.93 times the nominal wall thickness for structural design in the United States. (This value is above 0.90 due to a safety/reliability calibration that considered all statistical distributions for participating geometric and mechanical properties). For square and rectangular HSS, the only limitation on the outside corner radius is a maximum of three times the specified wall thickness. All section properties for this product in the AISC *Manual*—and available online—are calculated using this design wall thickness, as well as a presumed average outside corner radius of twice the design wall thickness. [In Canada, a design wall thickness of 0.90 times the nominal wall thickness is specified for structural design with ASTM A500/A500M (CSA, 2019; CISC, 2021)].

Although there are theoretically three grades of ASTM A500/A500M (B, C, and D), only Grades B and C are readily available, with these two grades having different minimum yield and ultimate strengths, as shown in Table 1-1. (Note that prior to 2021, Grades B and C had different yield and ultimate strengths depending on the section shape, but the mechanical properties for round, square, and rectangular HSS have now been harmonized.) Manufacturers in North America now commonly dual-certify their ASTM A500/A500M product to both Grades B and C, thus producing

to the higher grade and working to a “one product fits all” approach. Although this dual certification has implications when accounting for member expected overstrength in seismic design, specifying Grade C is now the rational and preferred choice for all ASTM A500/A500M material.

The ASTM A500/A500M standard has no provision for a Charpy V-notch (CVN) toughness rating, limiting its applicability in some dynamic loading situations. In fact, Note 1 in this standard specifically stipulates that, “Products manufactured to this specification may not be suitable for those applications such as dynamically loaded elements in welded structures, etc., where low-temperature notch-toughness properties, which are typically measured by Charpy Impact testing, may be important.” ASTM A500/A500M is listed as a permitted material for seismic force-resisting systems by the AISC *Seismic Provisions* and will often pass the requirements for CVN, if tested.

1.4.2 ASTM A1085/A1085M

This is a relatively new higher-performance HSS introduced in 2013, cold-formed either from round with ERW or directly to square or rectangular with ERW. As with ASTM A500/A500M, round, square, and rectangular shapes are produced with peripheries up to 88 in. Specified nominal wall thicknesses up to $\frac{7}{8}$ in. are covered. A tolerance of -5% is permitted on wall thickness, with a further tolerance on mass (and therefore, effectively the cross-sectional area) of -3.5% . These stringent tolerances ensure structural reliability comparable to competing hot-formed open sections; hence, AISC *Specification* Section B4.2 permits the design wall thickness to be equal to the nominal wall thickness. For square and rectangular HSS, a minimum outside corner radius has been introduced (as well as the same maximum as ASTM A500/A500M HSS), as follows: for $t \leq 0.400$ in., $1.6t \leq$ corner radius $\leq 3.0t$; for $t > 0.400$ in., $1.8t \leq$ corner radius $\leq 3.0t$. These lower limits on corner radii assist with alleviating some concerns associated with very tight cold-worked corners. All section properties for this product (available online from the AISC website) are calculated using a design wall thickness equal to the full nominal thickness, as well as a presumed average outside corner radius of twice the wall thickness.

Only a single grade (A) is available; therefore, the grade need not be specified. The single design yield strength of 50 ksi meets or exceeds the design yield strengths of all grades in ASTM A500/A500M (see Table 1-1). An upper limit of 70 ksi on the as-delivered yield strength is also specified in ASTM A1085/A1085M, which renders this material more suitable for energy-dissipative seismic designs where steel probable overstrength needs to be restricted. The ratio of the expected yield stress to the specified minimum yield stress, R_y , is different for various HSS products and grades that are permitted in seismic force-resisting systems by AISC

Seismic Provisions Table A3.2: R_y is 1.6 for ASTM A53/A53M, 1.4 for ASTM A500/A500M Grade B and ASTM A501/A501M, 1.3 for ASTM A500/A500M Grade C, but is reduced to 1.25 for ASTM A1085/A1085M.

ASTM A1085/A1085M also imposes a minimum CVN toughness requirement of 25 ft-lb at 40°F (34 J at 4°C), based on full-size longitudinal test specimens. This moderate toughness rating meets the American Association of State Highway and Transportation Officials (AASHTO, 2020) CVN criteria for fracture critical elements in Zone 2, making ASTM A1085/A1085M much more suitable for bridge applications (as well as cranes, sign supports, poles, roll-over protection structures, bollards, amusement rides, blast- and impact-resistant structures, and buildings governed by seismic loadings). As noted previously, the R_y value for ASTM A1085/A1085M is the lowest available for HSS products for use in seismic force-resisting systems. Like tensile test samples, Charpy impact test samples are taken from the finished HSS product, away from the weld seam, in the longitudinal direction and in the flat region in the case of square and rectangular HSS. For cold-formed HSS, this location produces the most favorable CVN reading (Kosteski et al., 2005; Sun and Packer, 2014b). If an increased toughness level is desired in the test location, ASTM A1085/A1085M provides a supplementary requirement S2 clause that enables an alternate CVN impact value and test temperature to be specified.

Another feature of ASTM A1085/A1085M is the ability to specify heat treatment of the finished cold-formed HSS (via supplementary requirement S1) at a temperature of 850°F (450°C) or higher, followed by cooling in air. This is equivalent to the Class H finishing option of the Canadian G40 standard (CSA, 2018a). The advantage of this heat treatment is a reduction in the level of residual stress throughout the entire cross section. When columns of intermediate overall slenderness are subjected to compression, a member with such heat treatment will have a delayed onset of first yielding (i.e., a higher proportional limit on the stress-strain curve) and hence a higher compressive strength relative to its untreated counterpart. In Canada (and Europe) one can take advantage of this generally higher column compressive strength by using a more-favorable “column curve” (CSA, 2019), but this option does not exist in the United States. The overall reduction in residual stresses arising from heat treatment will also decrease the effects of severe cold-working in the corners of square and rectangular HSS. This facilitates welding in the corner regions of thick-walled HSS, reduces the incidence of strain aging, and renders square and rectangular HSS more amenable to hot-dip galvanizing (Section 1.6.1).

1.4.3 ASTM A1065/A1065M

This is another relatively new standard for HSS introduced in 2009, cold-formed using a brake press directly to square

or rectangular with SAW. It is intended for large shapes, with peripheries up to 200 in. and nominal wall thicknesses up to 1 in. A tolerance of only -0.01 in. is permitted on wall thickness, which translates into a permissible undersizing tolerance of -1.0% to -3.2% across the thickness range. AISC *Specification* Section B4.2, therefore, permits the design wall thickness to be taken equal to the nominal wall thickness (Table 1-1). An outer corner radius of at least three times the wall thickness is specified; hence, calculated section properties are based on an outside corner radius of $3t$. The standard manufacturing method produces a seam weld of 80% penetration through the material thickness, which may be a concern in some connections. If 100% through-thickness welding is required, the customer may request this from the manufacturer.

Although a number of possible ASTM steel plate grades may be used as source material, this standard provides one minimum yield strength of 50 ksi HSS, termed Grade 50 (Table 1-1), although a variant with enhanced atmospheric corrosion resistance, Grade 50W, is also possible. The F_y/F_u ratio of Grade 50 (0.83) places this product outside the limit of validity (0.80) of some connection design recommendations in this Design Guide, but Grade 50W has $F_y/F_u = 0.71 < 0.80$. There is no automatic provision for a CVN toughness rating and the cautionary “Note 1” regarding applicability to dynamic loading in ASTM A500/A500M is reproduced in ASTM A1065/A1065M. However, supplementary requirement S1 enables the purchaser to specify a minimum CVN for the plate material.

1.4.4 ASTM A847/A847M

This is a cold-formed welded or seamless HSS in round, square, and rectangular shapes, with peripheries in welded sizes up to 88 in. and nominal wall thicknesses up to 1 in. The geometric tolerances are replicated from ASTM A500/A500M (see Section 1.4.1), resulting in a similar design wall thickness of 0.93 times the nominal wall thickness, therefore ASTM A500/A500M section properties from AISC—from the *Manual* or online—can be used for design. However, only one strength grade, 50 ksi minimum yield, is provided (Table 1-1). The feature of this product is that it provides a chemical composition giving improved atmospheric corrosion resistance over ASTM A500/A500M, to an index of 6.0 or higher, as described in Guide G101 (ASTM, 2020a). One of the main applications of this “weathering steel” HSS is in exposed pedestrian bridges, which are not normally subject to cyclic loading design criteria. Nevertheless, ASTM A847/A847M provides a supplementary requirement S1 clause that enables specification of a CVN impact value and test temperature. Appropriate welding procedures must be used in conjunction with this type of steel.

1.4.5 ASTM A53/A53M

This is actually a welded or seamless pipe production standard, intended for mechanical and pressure applications. Pipe sizes are properly designated using the NPS, which is the nominal pipe diameter with a scheduled wall thickness. The product needs to satisfy a hydrostatic test and is only produced in circular shapes, but Grade B, with a low yield strength of only 35 ksi, is authorized for use in structural applications by AISC *Specification* Table A3.1 and also for use in seismic force-resisting systems by AISC *Seismic Provisions* Table A3.1. For seismic applications, designers need to note the high material overstrength factor ($R_y = 1.6$). There is no CVN provision, but, being a pressure pipe standard, all ERW seam welds are subject to nondestructive testing (NDT). A tolerance of -12.5% is permitted on wall thickness, but with a controlling mass (and therefore cross-sectional area) tolerance of -10% . This area tolerance results in a thickness tolerance that is similar to ASTM A500/A500M. The AISC *Specification* thus assigns a design wall thickness of 0.93 times the nominal wall thickness (Table 1-1). Section properties can be obtained from the AISC *Manual*.

In many parts of the United States, ASTM A53/A53M Grade B pipe is used as a substitute for round HSS. Round HSS conforming to ASTM A500/A500M are most commonly available in standard pipe sizes (outside diameters of 3.5, 4, 4.5, 5, 5.563, 6.625, 8.625, 10.75, and 12.75 in.) and offer a yield strength premium over ASTM A53/A53M. Users should be aware that ASTM A53/A53M typically comes with a lacquer coating that needs to be removed from the pipe to accommodate welding and painting.

1.4.6 ASTM A501/A501M

Although HSS conforming to this standard is not routinely made in the United States, this specification (which dates back to 1964) has been maintained to primarily facilitate the importation of hot-formed or hot-finished hollow sections from other jurisdictions (e.g., Europe). The size range covers square and rectangular HSS with sides up to 16 in. and thicknesses up to 1 in., and round HSS diameters up to 48 in. and thicknesses up to 4 in. are within the scope. ASTM A501/A501M is an approved material in AISC *Specification* Table A3.1 and also for seismic force-resisting systems in AISC *Seismic Provisions* Table A3.1. A tolerance of -10% is permitted on wall thickness, with a further tolerance on mass (and therefore, effectively the cross-sectional area) of -3.5% . The latter tolerance is very tight and the same as ASTM A1085/A1085M, but AISC *Specification* Section B4.2 mandates a design wall thickness equal to 0.93 times the nominal wall thickness. For square and rectangular HSS, a maximum outside corner radius of twice the nominal wall thickness is stipulated. Because available sections are likely

to be “hard metric” sizes, section property tables are best obtained from the manufacturer.

Three grades (A, B, and C) are listed, with the highest strength being Grade C. The yield and tensile strengths of all grades are thickness-dependent but, for common wall thicknesses up to 1 in., Grade C has a minimum yield strength of 50 ksi (Table 1-1) and is preferable. Grade EN 10210: S355NH (CEN 2006, 2019) is very similar to ASTM A501/A501M Grade C, although a careful comparison of properties and composition is warranted for due diligence. All ASTM A501/A501M grades provide a CVN toughness of at least 20 ft-lb at 0°F (27 J at -18°C). The prime advantage of this product is that the manufacturing process delivers excellent fine-grain material properties, but hot-formed HSS is more expensive than its cold-formed counterpart. Hot-formed HSS are typically specified in Europe for demanding dynamic loading applications and where brittle fracture is a major concern, such as in vehicular and railway bridges, amusement rides, and cranes.

1.4.7 ASTM A618/A618M

This is another standard for hot-formed or hot-finished hollow sections, with seven grades from I to VII. The size range covers square and rectangular HSS with sides up to 16 in. and thicknesses up to 1 in., whereas round HSS diameters up to 48 in. and thicknesses up to $2\frac{1}{2}$ in. are within the scope. ASTM A618/A618M is an approved material in AISC *Specification* Table A3.1 for Grades I, II, and III, which all have a specified yield strength of 50 ksi up to $\frac{3}{4}$ in. thickness. Up to this thickness, the yield strengths of Grade IV (67 ksi), Grade V (90 ksi), Grade VI (100 ksi), and Grade VII (129 ksi) match the high-strength grades available in EN 10210 (CEN 2006, 2019). Such high-strength HSS present interesting options for designers, but they are well beyond the current scope for welded HSS-to-HSS K-connections in AISC *Specification* Chapter K, which limits the applicability of design guidance to nominal yield strengths up to 52 ksi.

A tolerance of -10% is permitted on wall thickness, with a further tolerance on mass (and therefore, effectively the cross-sectional area) of -3.5% . The latter tolerance is very tight and the same as ASTM A1085/A1085M and ASTM A501/A501M, but AISC *Specification* Section B4.2 mandates a design wall thickness equal to 0.93 times the nominal wall thickness. For square and rectangular HSS, a maximum outside corner radius of twice the nominal wall thickness is stipulated. As with ASTM A501/A501M, available sections are likely to be “hard metric” sizes, so section property tables are best obtained from the manufacturer. ASTM A618/A618M Grades III through VII also provide a CVN toughness of at least 20 ft-lb at 0°F (27 J at -18°C), which corresponds to the notch toughness regularly supplied in Europe.

Table 1-1. North American Manufacturing Standards for HSS and Pipe with Mechanical Properties and Design Wall Thickness of Common Grades

Product	Specification	Grade	Minimum Yield Stress, F_y , ksi (MPa)	Minimum Ultimate Tensile Stress, F_u , ksi (MPa)	Design Wall Thickness, t
Cold-formed HSS	ASTM A500/A500M	B	46 (317)	58 (400)	0.93 nominal
	ASTM A500/A500M	C	50 (345)	62 (427)	
Cold-formed HSS	ASTM A1085/A1085M	A	50 (345)	65 (448)	1.00 nominal
Cold-formed HSS	ASTM A1065/A1065M	50	50 (345)	60 (413)	1.00 nominal
		50W	50 (345)	70 (482)	
Cold-formed HSS	ASTM A847/A847M	–	50 (345)	70 (482)	0.93 nominal
Pipe	ASTM A53/A53M	B	35 (241)	60 (413)	0.93 nominal
Hot-formed HSS	ASTM A501/A501M	C	50 (345)	70 (482)	0.93 nominal
Hot-formed HSS	ASTM A618/A618M	III	50 (345)	65 (448)	0.93 nominal
Cold-formed HSS	ASTM A1110/A1110M	A	52 (358)	65 (448)	0.93 nominal
		B	57 (393)	70 (482)	
Cold-formed HSS	ASTM A1112/A1112M	Class 70	70 (482)	80 (551)	0.93 nominal
		Class 80	80 (551)	90 (620)	
		Class 90	90 (620)	100 (689)	
		Class 100	100 (689)	110 (758)	
		Class 110	110 (758)	118 (813)	
Pipe	API 5L	PSL2X52	52 (358)	66 (455)	0.93 nominal
Cold-formed and cold-formed stress-relieved HSS	CAN/CSA-G40.20/G40.21	350W	51 (350)	65 (450)	1.00 nominal

1.4.8 ASTM A1110/A1110M

This standard recently introduced in 2018 offers two grades (A and B) of cold-formed, higher-strength HSS in rounds and shapes, with both grades having a Charpy impact toughness rating. Grade A has a minimum yield stress of 52 ksi, with a CVN of 25 ft-lb at -20°F (34 J at -29°C); Grade B has a minimum yield stress of 57 ksi, with a CVN of 25 ft-lb at -40°F (34 J at -40°C). The F_y/F_u ratios of Grades A and B are 0.80 and 0.81, respectively. Requests for alternate CVN test temperatures can also be made to the manufacturer. Geometric tolerances are similar to ASTM A500/A500M, such that a design thickness of 0.93 times the nominal wall thickness would be assigned (Table 1-1), and ASTM A500/A500M section properties can be used. At present, production to this standard is limited in the United States to one manufacturer.

1.4.9 ASTM A1112/A1112M

This is the newest (2018) production standard, which is reflected by the ASTM number. The feature of ASTM A1112/A1112M is that it offers five grades of high-strength

HSS: Class 70, Class 80, Class 90, Class 100, and Class 110, where the class corresponds to the minimum yield stress in ksi (Table 1-1). The product is cold-formed directly to square or rectangular with ERW and is again currently limited in the United States to one manufacturer. The F_y/F_u ratios are high, as is typical for high-strength steels, ranging from 0.875 to 0.93. This implies that connection design—among other factors that require ductility—requires special consideration and may be beyond the scope of the AISC *Specification* (see Section 1.4.7). There is no CVN toughness rating and the cautionary “Note 1” regarding applicability to dynamic loading in ASTM A500/A500M is reproduced in ASTM A1112/A1112M. A wall thickness tolerance of -5% (as per ASTM A1085/A1085M), but without the tight mass tolerance of ASTM A1085/A1085M, means that a design wall thickness of 0.93 times the nominal wall thickness would be assignable (Table 1-1), and the section properties of ASTM A500/A500M can be used.

1.4.10 API 5L

Round tubes are sometimes required in long-span roofs, large-diameter columns, and bridges that are beyond the

size range of conventional HSS. In such cases, designers have often sourced pipe to the American Petroleum Institute (API) specification, API 5L (API, 2018). This covers pipe for pipeline transportation systems and has a multitude of tube grades and sizes. There are two product specification levels—PSL1 and PSL2. Level PSL2 has additional mandatory requirements for chemical composition, notch toughness, and strength properties, and additional NDT. PSL2 Grade X52, with a minimum yield strength of 52 ksi, is probably the most common choice for structural purposes. Wall thickness tolerances (down to -10%) vary with thickness, and tight mass tolerances apply for certain end conditions. This is not an approved material in AISC *Specification* Table A3.1 and is subject to approval by the Engineer of Record; therefore, a design wall thickness of 0.93 times the nominal thickness would be a prudent choice (Table 1-1). With a diameter range up to 84 in., this higher-quality pipe product addresses a frequent need for large diameter and thick-walled tubes.

1.4.11 CAN/CSA G40.20/G40.21

This is another production standard in North America for round, square, and rectangular HSS cold-formed from round with ERW that governs part of the overall HSS stock made in Canada (CSA, 2018a). There is one predominant grade

(Grade 350W) with a minimum yield strength of 350 MPa (51 ksi), applicable to all HSS. The same wall thickness and mass tolerances as ASTM A1085/A1085M apply (as ASTM A1085/A1085M was modeled on this CSA standard); hence, a design wall thickness equal to the nominal thickness would apply (Table 1-1), and the section properties given for ASTM A1085/A1085M can be used (available online from the AISC website) or obtained in SI units from the CISC Handbook (CISC, 2021).

This standard enables one to specify a CVN toughness requirement by stipulating Grade 350WT and one of five toughness categories. These range from Category 1 requiring 27 J at 0°C (20 ft-lb at 32°F), Category 2 requiring 27 J at -20°C (20 ft-lb at -4°F), Category 3 requiring 27 J at -30°C (20 ft-lb at -22°F), to Category 4 requiring 27 J at -45°C (20 ft-lb at -49°F), or even a Category 5 “to be specified by purchaser.” The cold-formed HSS end product (Class C) can be heat-treated to form an alternative Class H product by heating to 450°C (850°F) or higher, followed by cooling in air. This is directly equivalent to the supplementary requirement S1 of ASTM A1085/A1085M, which has the effects discussed in Section 1.4.2.

Pictorial applications of hollow sections utilizing some of the foregoing HSS and tube manufacturing standards are shown in Figures 1-10 through 1-13.

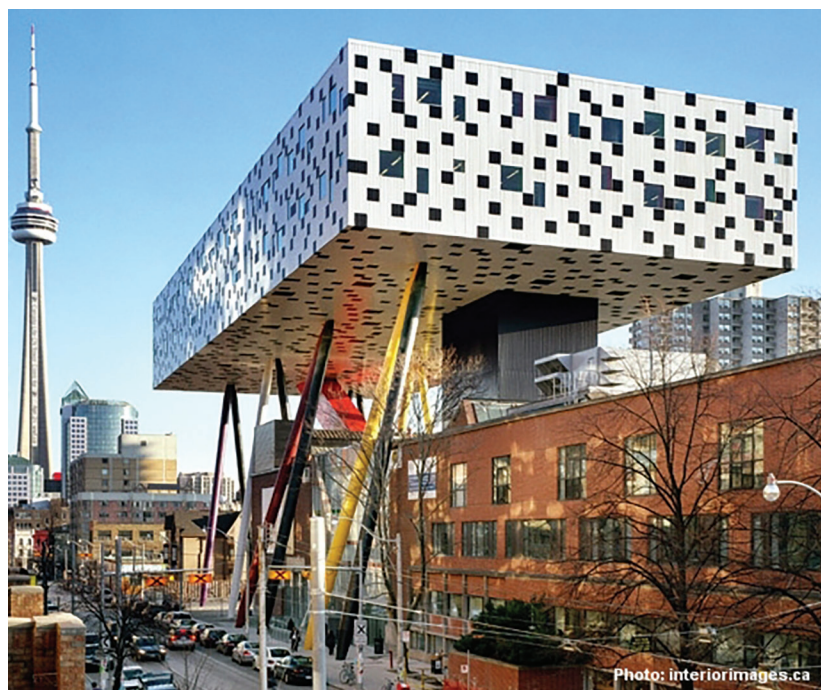


Fig. 1-10. Examples of large-size tubular columns to API specification (photos courtesy of Interior Images and Cast Connex Corporation).



(a) Rough surface finish with hot-formed HSS



(b) Smooth surface finish with cold-formed HSS

Fig. 1-11. Typical surface finishes associated with hot-formed (ASTM A501/A501M, ASTM A618/A618M) and cold-formed (ASTM A500/A500M, ASTM A1085/A1085M) HSS.

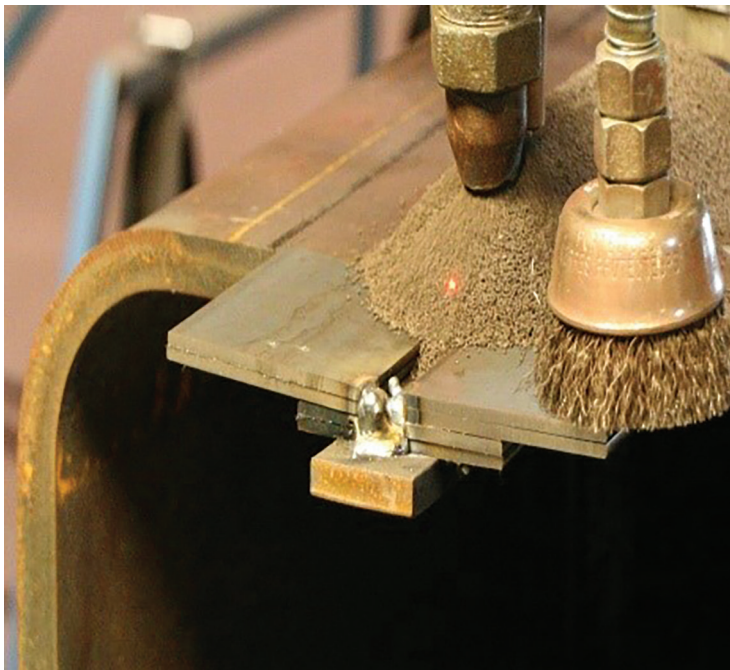


Fig. 1-12. ASTM A1065/A1065M Type 1 HSS (SAW C-shapes, with backing bars to the short sides, left in place) (photos courtesy of Valmont Industries).

1.5 ADVANTAGES OF HSS

1.5.1 AECS

Any exposed HSS members have the appeal of looking “modern,” but even higher visual finishing standards can be assured by specifically designating an AISC category for architecturally exposed structural steel (AECS) in the contract documents. The AISC *Code of Standard Practice* Section 10 has categories AECS 1 through 4, plus a further AECS C category for customized requirements (AISC, 2022a). HSS is frequently the chosen section when AECS is invoked.

1.5.2 Torsion

Hollow sections, especially round HSS, have the most effective cross section for resisting torsional loads because the material is uniformly distributed about the longitudinal axis and the shape is closed. A comparison between open steel sections and HSS of approximately the same mass shows that the torsional constant of HSS is about 200 times that of open sections (Wardenier et al., 2010).

1.5.3 Compression

HSS are very efficient compression members due to the favorable weak-axis radius of gyration, which often controls the available compressive strength. It is not reliable to make accurate cost comparisons between HSS and other shapes because prices vary with time, application, and geographic location. However, it is instructive to perform column design comparisons between ASTM A992/A992M (ASTM, 2022c) wide-flange shapes and competing HSS, selecting sections on the basis of minimum weight in all cases.

Comparisons are illustrated for two cases of axially loaded compression members—one with a moderate effective length ($KL = 16$ ft) and load (Table 1-2A), and a longer compression member ($KL = 32$ ft) with a higher load (Table 1-2B). For the required compressive strengths chosen, the LRFD load is 1.5 times the ASD load, and because this is the calibration load ratio in the AISC *Specification*, the same sections are determined in LRFD and ASD. Sections are selected for a particular length and available compressive strength using AISC *Manual* Table 4-1a for ASTM A992/A992M wide-flange shapes, AISC *Manual* Table 4-4



Fig. 1-13. ASTM A847/A847M weathering steel HSS (photo courtesy of The Wild Center, New York).

Table 1-2A. Comparison of W-Shape and HSS Compression Members—Moderate Length and Load

Length and Required Strength	$KL = 16 \text{ ft}$				
	$P_u = 240 \text{ kips (LRFD)}$		$P_a = 160 \text{ kips (ASD)}$		
Member Type and Grade	W-Shape ASTM A992/A992M	HSS ASTM A500/A500M Grade C		HSS ASTM A1085/A1085M	
		Round	Square	Round	Square
Section	W8×35	HSS9.625×0.250	HSS8×8×¼	HSS9.625×0.250	HSS8×8×¼
Available strength, kips	241 (LRFD) 160 (ASD)	242 (LRFD) 161 (ASD)	243 (LRFD) 162 (ASD)	259 (LRFD) 173 (ASD)	260 (LRFD) 173 (ASD)
Weight ¹ , lb/ft	35 (100%)	25.1 (72%)	25.8 (74%)	25.1 (72%)	25.8 (74%)
Surface area ¹ , ft ² /ft	3.89 (100%)	2.52 (65%)	2.60 (67%)	2.52 (65%)	2.60 (67%)

¹ The number in parentheses represents the percentage of nominal weight or surface area relative to the wide-flange option.

Table 1-2B. Comparison of W-Shape and HSS Compression Members—Longer Length and Higher Load

Length and Required Strength	$KL = 30 \text{ ft}$				
	$P_u = 750 \text{ kips (LRFD)}$		$P_a = 500 \text{ kips (ASD)}$		
Member Type and Grade	W-Shape ASTM A992/A992M	HSS ASTM A500/A500M Grade C		HSS ASTM A1085/A1085M	
		Round	Square	Round	Square
Section	W14×120	HSS18.000×0.500	HSS14×14×½	HSS16.000×0.500	HSS14×14×½
Available strength, kips	807 (LRFD) 537 (ASD)	900 (LRFD) 599 (ASD)	809 (LRFD) 538 (ASD)	798 (LRFD) 531 (ASD)	867 (LRFD) 577 (ASD)
Weight ¹ , lb/ft	120 (100%)	93.5 (78%)	89.7 (75%)	82.9 (69%)	89.7 (75%)
Surface area ¹ , ft ² /ft	7.07 (100%)	4.71 (67%)	4.53 (64%)	4.19 (59%)	4.53 (64%)

¹ The number in parentheses represents the percentage of nominal weight or surface area relative to the wide-flange option.

for ASTM A500/A500M Grade C square HSS, AISC *Manual* Table 4-5 for ASTM A500/A500M Grade C round HSS, and the AISC website for ASTM A1085/A1085M HSS. It should be noted that all five member types and grades have the same specified minimum yield stress of 50 ksi.

It can be seen in Tables 1-2A and 1-2B that the weights of the HSS column options range from 69% to 78% of the alternative wide-flange section. This decrease in compression member weight can also be applied to trusses, where approximately half of the members are loaded in compression. Lighter structural steelwork produces further cost benefits—foundation size can be reduced, transportation costs to site are reduced, and crane lifts are simpler. Also, there is a clear environmental benefit to using less material.

1.5.4 Surface Area

The surface areas shown in Tables 1-2A and 1-2B are from AISC Design Guide 19 (Ruddy et al., 2003) for wide-flange sections, calculated by πD (with D in ft) for round HSS, and from either AISC Design Guide 19 or AISC *Manual* Table 1-12 for square HSS. These tables show that the outside surface areas of HSS compression members are

significantly less than their wide-flange counterparts. This is particularly influential for decreasing the costs of painting or surface-coating fire-protection materials.

1.5.5 Concrete Filling

Filling with a nonshrink concrete or grout is a frequent use of the internal void, which enables the member to be designed as a composite section. The concrete filling increases the wall width-to-thickness limits to enable thinner HSS cross sections to be classed as nonslender, and the nominal compression strength prescribed by AISC *Specification* Chapter I is usually increased (Zhao et al., 2019). Moreover, the steel of the HSS acts as “left-in-place formwork” for the concrete, which is another construction efficiency. Load transfer between the HSS and concrete needs to be considered in accordance with AISC *Specification* Section I6.

A common method for filling reasonable-size columns is to pump the concrete or grout through a spigot at the bottom as shown in Figure 1-14. In addition to concrete infill, reinforcing bars or steel fibers can also be included. The concrete infill (and reinforcing bars, if included) add to the flexural stiffness of the member, resulting in a stiffer structural

frame. Another structural advantage of concrete filling is that the strength of some connections, particularly where a compression force is acting normal to the face of an HSS member, may be increased.

Concrete filling of HSS columns contributes not only to an increase in member capacity, but it also improves the fire resistance (the time duration for which a fire can be tolerated). Extensive test programs have shown that reinforced concrete-filled HSS, without any external fire protection, can attain a fire life of over 2 hr, depending on the ratio of steel-to-concrete in the cross section, the percentage of reinforcement, and the applied load (Twilt et al., 1994). An empirical fire design method that is valid over a limited parameter range is given by Kodur and MacKinnon (2000) and AISC Design Guide 19. This shows that a 2 hr fire rating can even be achieved by some concrete-filled HSS columns without reinforcing bars or steel fibers. During an extreme fire, moisture in the concrete is converted to steam, which must be allowed to escape to prevent bursting of the HSS. Thus, concrete-filled HSS without complete external fire protection (which can subject the concrete infill to heat) must be provided with steam vent holes. AISC Design Guide 19 recommends that two ½-in.-diameter holes should be placed opposite each other in the HSS walls, at both the top and bottom of the column and clear of the floor slab. Concrete filling

is sometimes specified as an invisible fire-protection measure, after the structural steel design has been completed. It is clearly beneficial to make such a decision early in the design process to take advantage of composite action.

1.5.6 Resistance to Fluid Flow

Bare hollow sections, and in particular round HSS, offer lower resistance to the flow of fluids (such as air and water) than shapes with flat elements (such as W-shapes). Tubular members are thus very popular in offshore structures, towers, monopoles, and masts. Wind forces on exposed steel members are proportional to a drag (or shape) coefficient that varies considerably with the orientation of the section (unless circular) relative to the direction of the wind and with the Reynolds number. Conservatively, a reduction in the wind force acting on the projected area of an exposed HSS (or a framework composed of HSS) can be taken—relative to a section or shape comprised of flat elements—of (AISC, 1997a, 1997c):

1. 0.67 for round HSS.
2. $0.4 + 0.6B/H \geq 0.67$ for rectangular HSS with outside corner radii $\geq 0.05B$ with the wind force is acting on the short side, B . (In this instance H is considered the long side). Given that the outside corner radius is assumed to be



Fig. 1-14. Filling a round HSS by pumping concrete from the bottom of the column.

Table 1-3. Typical Contemporary Cold-Bending Radii (WhiteFab Inc.)

Member	Bending Axis	B/H	B/t	Minimum Outside Bend Radius, R_{min}	R_{min}/B	R_{min}/H
HSS20×12×½	Easy way	1.67	43	35'-0"	21	35
HSS16×12×½	Easy way	1.33	34	30'-0"	22.5	30
HSS14×6×¾	Hard way	0.43	17	15'-0"	30	12.9
HSS12×8×¾	Hard way	0.67	23	15'-0"	22.5	15
HSS10×6×⅝	Hard way	0.60	21	15'-0"	30	18
HSS8×6×¼	Hard way	0.75	26	12'-0"	24	18
HSS6×4×¼	Hard way	0.67	17	10'-0"	30	20

Note: B = dimension of HSS at the outside bend radius; t = design wall thickness

twice the design wall thickness, t , for HSS section properties, this implies a liberal application limit of $B/t \leq 40$ for the short side.

3. 1.0 for rectangular HSS under other conditions and for all square HSS.

1.5.7 Cold Bending or Hot Bending

Curved steel hollow sections are widely used in the construction industry. The possible curvature of these members

is limited by the tolerable magnitude of distortion of the cross section during the bending process. Wall distortion is more of a concern for cold bending (the most common process) relative to hot bending, with the latter process enabling smaller bending radii. A lower bend radius is achievable with thicker-walled HSS without causing wall distortions, and furthermore, round HSS are easier to curve (Figure 1-15) than square or rectangular HSS. Bending a rectangular HSS about the strong axis (or “the hard way”), although requiring more force, makes a rectangular HSS less susceptible



Fig. 1-15. Stocky, round HSS, tightly curved by three-roll cold bending.

to deformation problems, relative to the same rectangular HSS bent about the weak axis (or “the easy way”). Benders are typically a specialty subcontractor of the fabricator and consider their process both a science and an art, and therefore, capabilities need to be confirmed with individual facilities. Some typical minimum cold-bending radii for rectangular HSS are given in Table 1-3. A list of AISC-member bender-rollers across North America is obtainable from www.aisc.org/benders. Additional information on the design of curved members can be found in AISC Design Guide 33, *Curved Member Design* (Dowswell, 2018a).

1.5.8 Connecting to Steel Castings

Steel castings are increasingly specified for nodes, as connectors to the ends of members (even as catalog items), for designated functions in seismic force-resisting systems, and in AECS applications. Castings present the ability to tailor-make 3D components and locate steel where it is particularly desired, to resist high forces efficiently, to provide enhanced fatigue resistance, to act as sacrificial fuses in earthquake-resistant structures, and to deliver aesthetic surface finishes. HSS are the natural and almost universal member choice for connecting to steel castings (Figure 1-16).

AISC *Specification* Section A3.2 requires that steel castings and forgings conform to an ASTM standard intended for structural applications and provide strength, ductility, weldability, and toughness to be fit-for-purpose. The commonly specified grades for structural applications include ASTM A216/A216M Grades WCA, WCB, or WCC (ASTM, 2021h); ASTM A352/A352M Grades LCA, LCB, or LCC

(ASTM, 2021i); and ASTM A958/A958M Grade SC8620 Class 80/50 (ASTM, 2017). Advice on welding HSS to castings is given in AISC Design Guide 21 (Miller, 2017).

1.6 OTHER CONSIDERATIONS

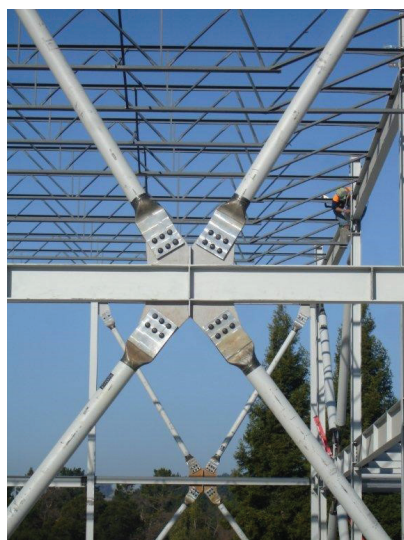
1.6.1 Galvanizing and Metalizing

AISC *Specification* Section B3.13 notes that corrosion of steel components must be addressed where corrosion could impair the strength or serviceability of a structure. Painting, galvanizing, and metalizing are popular coating systems used to inhibit corrosion.

With hot-dip galvanizing, the steelwork is immersed in a bath of molten zinc (with up to 2% of other elements) at a temperature around 825° to 870°F (440° to 465°C). The average galvanizing kettle is 40 ft long, although oversized pieces can sometimes be dipped in stages. The zinc temperature and immersion time do influence the thickness of the zinc coating obtained, but the most critical factor is the steel chemical composition and in particular the silicon (Si) content. The American Galvanizers Association (AGA, 2006) specifically recommends the following limits for good galvanized coatings:

1. $C < 0.25\%$, $P < 0.04\%$, and $Mn < 1.35\%$
2. $Si < 0.04\%$ or $0.15\% < Si < 0.22\%$

The silicon ranges specifically avoid $0.04\% < Si < 0.15\%$, a zone of high reactivity termed the “Sandelin effect” in which thicker-than-average zinc coatings will likely be deposited. ASTM A500/A500M and ASTM A1085/A1085M heat



(a) Braced frame during erection



(b) Casting with tapered nose fitted into HSS



(c) Casting-to-HSS fabrication

Fig. 1-16. Cast Connex High Strength Connectors™ welded to round HSS diagonals (photos courtesy of Cast Connex Corporation).

compositions generally meet these carbon (C), phosphorous (P), and manganese (Mn) limits, but there is no control on Si with ASTM A500/A500M. ASTM A1085/A1085M has a $\text{Si} \leq 0.04\%$ restriction, which makes it a preferable product for galvanizing. A low silicon content gives a lower carbon equivalent (CE), according to some CE formulas, which has been shown to lower the susceptibility of steel to liquid metal embrittlement (BCSA/GA, 2005).

The temperature of a galvanizing bath is very similar to that specified for ASTM A1085/A1085M Supplementary Requirement S1 for heat treatment (see Section 1.4.2), so stress relief will occur. This should be borne in mind if hot-dip galvanizing cold-bent HSS members (see Section 1.5.7), for example, as pieces may distort. With galvanizing, however, the stress relief is very rapid, and it occurs in the presence of a liquid metal that often includes certain additives that exacerbate the situation. The heavily cold-worked corners of square and rectangular HSS have a very high level of residual stress, and all conditions may exist to induce liquid metal embrittlement of the corners (Packer et al., 2010a). One of the contributing causes of liquid metal embrittlement is the metallic content of the galvanizing bath itself; small quantities of certain elements—in particular, tin and bismuth—need to be well-controlled (Kania et al., 2020). Thick-walled square and rectangular HSS with an open end to the member are most at risk of corner cracking during hot-dip galvanizing (Sun and Packer, 2019).

Although the outside corner radius of ASTM A1065/A1065M is specified to be three times the wall thickness, $3t$, or greater, this is the *upper* corner radius limit for square and rectangular HSS to ASTM A500/A500M and ASTM A1085/A1085M. In fact, the outside corner radius of ASTM A500/A500M and ASTM A1085/A1085M averages $2t$ in practice, and this geometry is used for determination of their section properties. The galvanizing standard ASTM A143/A143M (ASTM, 2020b) states that a cold-bending radius of three times the section thickness will ordinarily ensure satisfactory properties in the final product. Thus, all square and rectangular ASTM A500/A500M and ASTM A1085/A1085M HSS produced in North America are potentially prone to embrittlement and corner cracking during hot-dip galvanizing. Kinstler (2005) pointed out that the amount of cold work, as measured by the bending radius, is the most important single factor to consider when there is concern for brittle-type failure of steel galvanized after cold working.

ASTM A143/A143M notes that, for corner bends with a radius smaller than their recommendation of $3t$, the HSS should be given proper thermal treatment prior to galvanizing. ASTM A143/A143M states that for heavy cold deformation exemplified by cold rolling, subcritical annealing at temperatures from 1,200° to 1300°F (650° to 705°C) should be employed. ASTM A1085/A1085M Supplementary Requirement S1 provides heat treatment of 850°F (450°C)

or higher, so this could be specified for square and rectangular HSS that is destined for hot-dip galvanizing. Cracking in HSS corners during hot-dip galvanizing typically starts from the inside, and grinding the inside corners at open ends of a member has been found to be effective in improving crack resistance. This procedure helps to remove folds and other surface roughness that tend to act as stress raisers and crack initiation sites. It also probably removes some of the hardest and most brittle material at these locations (Poag and Zerovoudis, 2003). Trial dipping of HSS member lengths, with a particular galvanizer, is also always an option.

A visual inspection of the corners of square and rectangular HSS at the member ends is prudent after galvanizing. However, one must bear in mind that small “invisible” cracks may be present but masked by filling with liquid zinc. Such cracks may become a problem during the service life of the structure, particularly if fatigue loading occurs. Round HSS, which do not have zones of severe cold forming like rectangular HSS, do not have any concerns with cracking during hot-dip galvanizing. Multi-sided cold-formed hollow sections, such as used for high mast illumination poles (HMIPs), on the other hand, have been found to occasionally crack during hot-dip galvanizing (Nasouri et al., 2019).

For hot-dip galvanizing, holes to allow for filling, venting, and drainage are necessary (Figure 1-17), and adequate sizing minimizes differential thermal stresses experienced by the structural component during galvanizing. It is not an option to leave HSS sealed during hot-dip galvanizing because there is a high risk of bursting the HSS. Consequently, the inside surfaces of the HSS will also be galvanized. Vent holes with a minimum diameter of $\frac{1}{2}$ in. and drain holes with a diameter of 1 in. are recommended. It has been found that the presence of vent and drainage holes in HSS trusses, such as at the ends of branch members (see Figure 1-17), does not have a significant effect on the HSS connection resistance (Serrano et al., 2019).

Metalizing is another form of coating structural steel for corrosion control, providing protection for significantly longer times than just paint, but is more expensive than galvanizing. In this process, zinc or aluminum wire is continuously melted and propelled by compressed air and is sprayed onto the steelwork forming both a barrier and a galvanic coating. The steel remains relatively cool (around 250° to 300°F) so there is virtually no risk of heat distortion or liquid metal embrittlement.

1.6.2 Notch Toughness

ASTM A53/A53M, ASTM A500/A500M, ASTM A847/A847M, ASTM A1065/A1065M, and ASTM A1112/A1112M have no toughness requirements, although ASTM A1065/A1065M includes a supplementary requirement S1 in which the purchaser can request CVN test results for the parent material. ASTM A500/A500M, ASTM A847/A847M,

ASTM A1065/A1065M, and ASTM A1112/A1112M even bear a warning note that the product may not be suitable for dynamic load applications. The inherent notch toughness of such North American HSS can be very low (Kosteski et al., 2005). Heat treatment at a temperature in the range of 850° to 900°F (450° to 480°C), such as provided by ASTM A1085/A1085M supplementary requirement S1 or CSA G40 Class H, does not affect the metallurgical properties to the extent of influencing the toughness. It has been shown by laboratory testing that such heat treatment does not provide any improvement in the CVN toughness of cold-formed HSS (Kosteski et al., 2005; Sun and Packer, 2014b).

Hot-formed HSS ASTM A501/A501M (all grades) and ASTM A618/A618M (Grades III to VII) include a CVN requirement for full-size longitudinal test specimens of 20 ft-lb at 0°F (27 J at -18°C). The cold-formed ASTM A1085/A1085M standard includes a CVN requirement, again based on full-size longitudinal test specimens, of 25 ft-lb at 40°F (34 J at 4°C), although more stringent requirements can be specified using supplementary requirement S2. Cold-formed ASTM A1110/A1110M provides a CVN minimum of 25 ft-lb (34 J) at either -20°F (-29°C) or -40°F (-40°C), based on full-size specimens. Sun and Packer (2014b) give a discussion of typical variations in CVN values around the cross section of cold-formed HSS, as well as the use of subsize coupons and through-thickness versus surface V-notches.

1.6.3 Internal Corrosion and Freezing

AISC *Specification* Commentary Section B3.13 discusses the topic of internal corrosion in HSS. In summary:

- Corrosion will not occur in an enclosed building and is a consideration only in HSS exposed to weather.
- Internal corrosion will not occur in a sealed HSS because corrosion will stop once the oxygen or moisture necessary for oxidation is used up.
- Weep (drain) holes are recommended at locations where water cannot flow into the HSS by gravity yet allow water to exit if there is any possibility of water accumulating inside an exposed HSS.
- Internal protection (e.g., by galvanizing) is required only in open HSS where a change in air volume by ventilation or direct flow of water is possible, or when the HSS is subjected to a temperature gradient that would cause condensation.

Care should be taken to avoid water accumulating in an HSS during or after construction because the expansion caused by possible freezing can create pressure that is sufficient to burst the HSS (Figure 1-18). If water accumulation in a column or post cannot be avoided, then galvanizing is recommended, and a drain hole of at least ½ in. diameter near the base plate should be provided.



Fig. 1-17. HSS truss with drain/vent holes after hot-dip galvanizing (photo courtesy of American Galvanizers Association).

1.6.4 Fire Protection

Construction materials gradually lose strength at elevated temperatures, and unprotected HSS have an inherent fire resistance of only 15 to 30 min. The temperature at which a steel member reaches its ultimate limit state depends on the cross section and the load level. It should be noted that building codes do provide some cases in which structural steel can be left unprotected against fire, but when HSS are required to have extended fire resistance times, additional measures have to be considered to delay the rise in steel temperature.

AISC *Specification* Section B3.12 introduces structural design for fire conditions, which is more fully covered in AISC *Specification* Appendix 4. During a “design-basis fire,” a building must maintain both its load-bearing function and any other performance requirements stipulated for its type of occupancy, according to the applicable building code. Two methods of design for fire conditions are provided: (1) by analysis and (2) by qualification testing. Design by analysis is a relatively new holistic performance-based technique, whereas design by qualification testing involves using prescriptive approved assemblies with a fire rating time based on certified tests already conducted to ASTM E119 (ASTM, 2022d). If performance-based fire engineering is to be employed, it should be recognized that cold-worked steel loses strength and stiffness at high temperatures more quickly than hot-rolled steel and that the strength increase due to cold-working may not be available post-fire.

Where fire protection is required, it can be active or passive. Active protection typically involves a water system,

such as provided by sprinklers, spray, or mist. However, HSS columns offer the possibility of being cooled by filling with water and thereby serving a dual purpose (Twilt et al., 1994; Wardenier et al., 2010). Cooling is achieved by water absorbing the heat from the HSS, circulating the water, and then vaporizing water. Such systems have been used on rare occasions in the United States (e.g., in the 64-story U.S. Steel Tower in Pittsburgh, Pennsylvania) because they are quite sophisticated and need a thorough design with proper hydraulic installations. Two different systems are possible—permanently water-filled columns (with consideration of internal corrosion and freezing) or columns that are filled only when a fire breaks out (which requires a fire detection system and rapid water filling).

Passive protection methods include:

- Direct applied systems, such as spray-applied fire-resistive materials (SFRM), using fibrous or cementitious protection, and intumescent coatings (paints and mastics that expand and blanket the HSS during a fire).
- Membrane systems, which include mineral fiberboard, gypsum wallboard, and rated ceilings.
- Concrete systems, utilizing concrete or masonry encasing as a barrier, or concrete-filling of the HSS. The latter process is discussed in Section 1.5.5.

More details regarding fire protection can be obtained from AISC Design Guide 19 (Ruddy et al., 2003).



(a) Longitudinal cracking in a cold-formed corner with low ductility

(b) Ice forming on the outside from water seeping out through crack

(c) Typical repair procedure with an angle and a drain hole

Fig. 1-18. Cracking of HSS due to internal ice formation.

Chapter 2

Limit States for HSS Connections

This chapter presents the limit states (or failure modes) that govern the design of welded HSS connections. These are discussed, and their manifestation in the AISC *Specification* is displayed. Examples of welded connection nominal strength formulas are derived; appropriate resistance factors are justified; and, where necessary, limitations on the validity are pointed out. The limit states for bolted HSS connections are the same as for all bolted connections, and these are covered in Chapter 4. Features of the connectors (welds and fasteners) themselves that are special to HSS applications are reviewed in Chapter 3 (welds) and Chapter 4 (mechanical fasteners). A physical understanding of the limit states for HSS connections allows the user of this Design Guide to understand HSS connection behavior and extrapolate “engineering judgment” to other connection cases that are beyond the scope of this Design Guide.

2.1 EVOLUTION OF HSS CONNECTION DESIGN IN THE AISC SPECIFICATION

The design of HSS connections was addressed explicitly in the AISC *Specification* for the first time in the 2005 edition, with the addition of a new Chapter K (Design of HSS and Box Member Connections). This was presented in three sections (K1: Concentrated Forces on HSS; K2: HSS-to-HSS Truss Connections; and K3: HSS-to-HSS Moment Connections) with descriptive text, sets of limit states for a variety of connection types, and certain geometric parameter ranges in which each limit state needed to be checked. Furthermore, the entire collection of limits of validity, each of which might be applicable to one or more limit states, was given as a restriction on the use of the nominal strength formulas for a whole connection type. The design methods were thus very prescriptive, the connection checks were simplified in number to suit hand calculations, and designs were constrained to a parameter range that was perceived to produce efficient connections.

The first edition of this Design Guide (Packer et al., 2010b) conformed to the 2005 AISC *Specification* (AISC, 2005a) and presented a tabulated version of these design provisions. This tabulated presentation was then introduced into the 2010 AISC *Specification* (AISC, 2010), complete with pictorial representations of the many connection types. Only very minor changes were made to the technical content of Sections K1, K2, and K3, but the scope of

Chapter K was expanded with a new Section K4: Welds of Plates and Branches to Rectangular HSS. The first edition of this Design Guide was thus still applicable to the 2010 AISC *Specification*.

After 2010, AISC considered that Chapter K was an anomaly in the *Specification* because it represented prescribed solutions for connection types rather than elaborating a set of possible limit states with their corresponding available strengths, as done elsewhere in the AISC *Specification* (e.g., bolted connections). Also, with such a tabulated prescriptive design presentation, many limit states and formulas were repeated identically or in a rearranged form. Moreover, certain limit states, such as those that were analytically derived and had no bounds on their use (e.g., yield-line solutions) were constrained by a package of limits of validity applied to a whole connection group. Thus, for the 2016 AISC *Specification*, a connection limit state was described generically just once in Chapter J (Design of Connections) for all connections, and a reduced-size Chapter K (Additional Requirements for HSS and Box-Section Connections) covered extra requirements that could not be deduced from Chapter J. Unlike the two previous versions of the AISC *Specification*, the 2016 AISC *Specification* did not usually point out where particular limit states would not govern (to avoid redundant checks). It is hence generally implicit that all plausible limit states need to be considered. This entails longer manual checking procedures but is not a concern for computer-programmed checking. It is important to note that design is still permitted to be performed outside the parameter limits of applicability (when they are stipulated in Chapter K). In such cases, the AISC *Specification* points out that design “by rational analysis” may be used. In the 2022 AISC *Specification*, modest changes have been made to Chapter K to improve the clarity.

This Design Guide identifies information from the 2022 AISC *Specification* Chapters J and K and presents it in a collated form, applied to types of HSS connections to facilitate and aid design. Relative to the first edition of this Design Guide, the 2022 AISC *Specification* has necessitated relatively few technical changes. Some advances in the 2022 *Specification* relative to the 2016 edition include the definition of some common parameters in Section K1, the addition of weld effective lengths for round-to-round HSS connections in Section K5, and revisions to the strength of single-sided fillet welds to rectangular HSS in Section J2.4.

2.2 STRENGTH VS. DEFORMATION LIMIT STATES

AISC *Specification* Section K1 notes that connection strengths in Chapter K, plus the applicable sections of Chapter J, are based on strength limit states only. Some of these “strength limit states” are based on connection plastic deformation patterns (discussed in Section 2.3) but still represent collapse mechanisms. Because there is no serviceability-load-level check performed for connection deformations, it is assumed that ultimate limit state strength-based formulas will result in an acceptable connection displacement at the service-load level. For hollow section connections, a local deformation of 1% of the hollow section width or diameter ($0.01B$ or $0.01D$), normal to the HSS connecting face, has often been considered a reasonable limit under service loads (IIW, 1989; Wardenier et al., 2010). This is relatively arbitrary but is very similar to the permitted manufacturing tolerances (which include allowances for concavity and convexity) for larger-size round and square ASTM A500/A500M, ASTM A847/A847M, and ASTM A1085/A1085M HSS.

Some HSS connections under specific load actions have been found to give service-load deformations exceeding the 1% nominal value if the connection limit state was defined solely on the basis of ultimate strength. For example, Figure 2-1(a) illustrates three of the EHS T- or X- (cross-) connection configurations that have been tested experimentally (Haque and Packer, 2012). (EHS behavior represents a combination of round and rectangular HSS behaviors, as may be expected.) The branch-to-chord width ratio, β , of these connections ranges from 0.5 (Type 1), producing a very flexible connection, to 1.0 (Types 2 and 3), which are much stiffer. The extreme chord deformations experienced by the Type 1 connection at ultimate load under branch axial compression

are shown in Figure 2-1(b), which illustrates a transverse bisection of the T-connection.

The Type 1 connection has a chord width of 220 mm (8.66 in.), so a tolerable service-load chord displacement would be approximately 2.2 mm (0.09 in.) Figure 2-2(a) shows the load-displacement response of three Type 1 ($\beta = 0.5$) connections under branch axial compression (T- and X-connections with the branch at 90° and 45° to the chord). If an LRFD-to-ASD (or factored-to-unfactored) load ratio of 1.5 is assumed, one can see that by dividing the absolute ultimate (peak) load by 1.5, one obtains a notional service load with a corresponding connection displacement that may considerably exceed 2.2 mm (0.087 in.). Thus, caution needs to be exercised when interpreting the “ultimate limit state strength” that is to be used for HSS connection design. Figure 2-2(b), on the other hand, illustrates the load-displacement responses of three Type 3 ($\beta = 1.0$) connections, with a chord width of 110 mm (4.33 in.), for which a tolerable service-load chord displacement would be approximately 1.1 mm (0.04 in.) These stiff connections, unlike those in Figure 2-2(a), all have a distinct peak ultimate load, and the corresponding service-load deformations are all well below the notional 1% value.

The concept of an ultimate deformation limit, at which the connection is deemed to have failed, has been a long-standing proposition for HSS connections, which are known to be very flexible. Ultimate deformation limits have been proposed by Mouty (1976, 1977), Yura et al. (1980, 1981), Korol and Mirza (1982), and Lu et al. (1994). In the case of Lu et al., an ultimate deformation of 3% of the main member width ($0.03B$ or $0.03D$), normal to the HSS connecting face, was proposed. The suitability of this ultimate deformation limit for a variety of HSS connections was demonstrated by Lu et al. and Zhao (1996). An extensive discussion of HSS deformation limits is given by Koteski et al. (2003). The

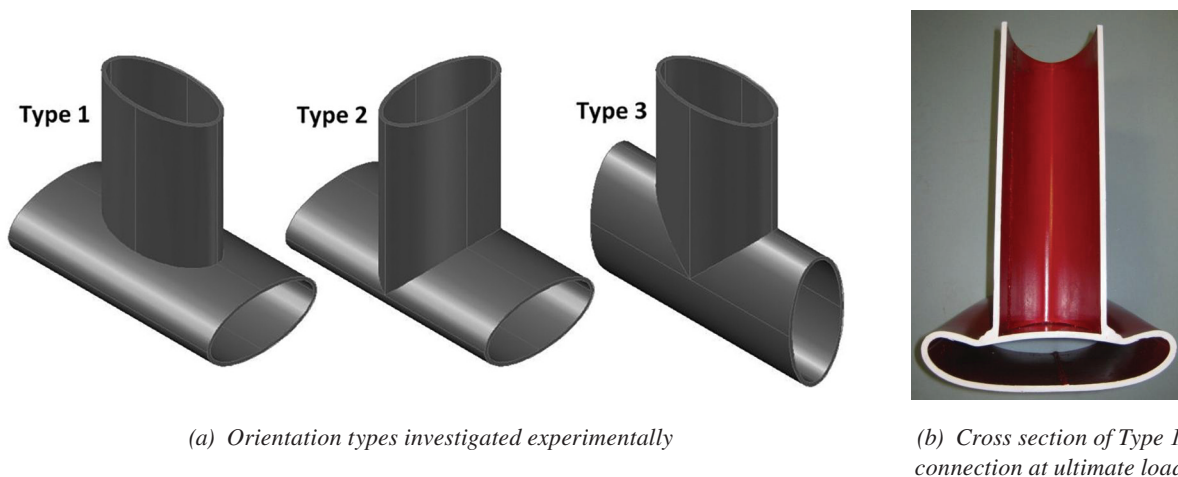


Fig. 2-1. EHS T-connections with member aspect ratios of 2:1.

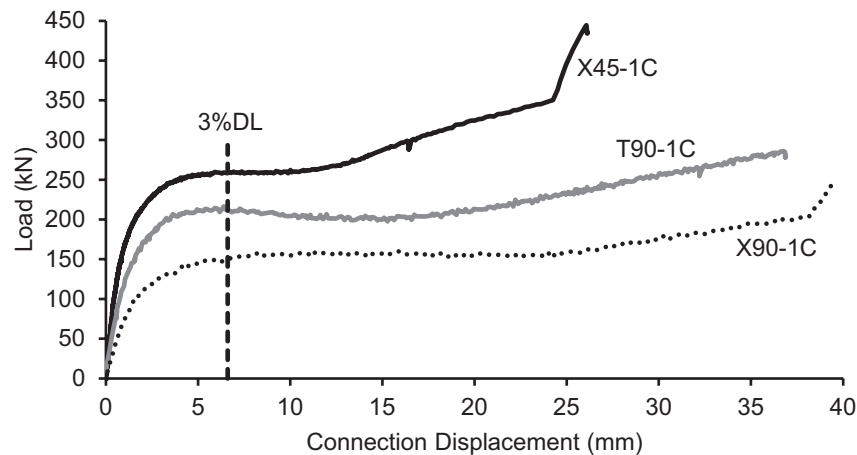
load corresponding to a connection deformation of 3% has now been adopted by IIW as another ultimate limit state condition, for the interpretation of experimental or numerical tests where a peak load does not occur prior to 3% deformation and is incorporated into recent design recommendations by IIW (2012), ISO (2013), and CIDECT (Wardenier et al., 2008a; Packer et al., 2009; Wardenier et al., 2010). This 3% ultimate deformation limit is shown in Figure 2-2 as 3%DL. In the case of stiff connections [Figure 2-2(b)], it is considered irrelevant because a pronounced peak load occurs at a displacement prior to the 3%DL. However, in the case of flexible connections [Figure 2-2(a)], it can be seen to provide an appropriate control on service-load deformations.

A deformation limit is not incorporated into the HSS connection design procedures in the *AISC Specification*. However, it should be pointed out that highly flexible HSS connections include a limit state of main member

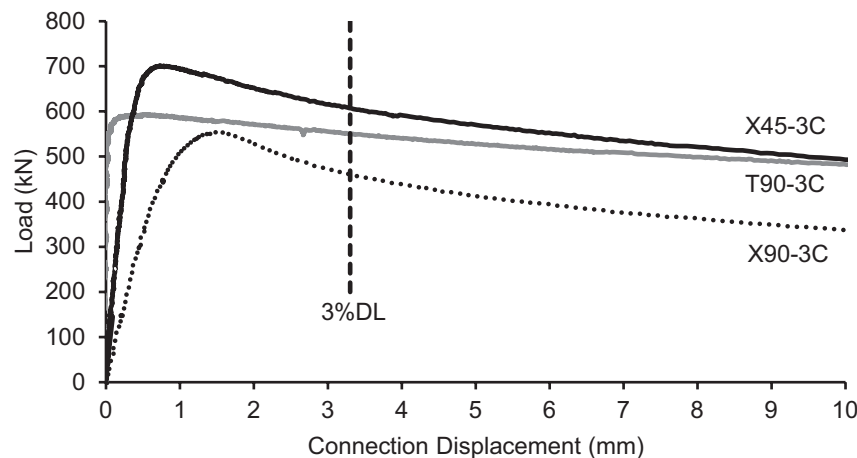
“plastification” that is often determined by yield-line analysis (Section 2.3), and this serves as a means of controlling connection deformations at both the ultimate and serviceability load levels. *AISC Specification* Commentary to Chapter K notes that if connection deformations are a concern, due to serviceability or stability, the IIW (2012) or CIDECT (Wardenier et al., 2008a; Packer et al., 2009) recommendations could be used. Connection flexibility is not normally an issue with triangulated truss assemblies, but it may need to be considered with moment connections and frames.

2.3 LIMIT STATE OF WALL PLASTIFICATION

AISC Specification Section J10.10 requires that the limit state of flexure be considered, in accordance with Section J4.5, when there is a force component applied normal to the plane of a plate element.



(a) Load vs. chord face displacement for Type 1 ($\beta = 0.5$)



(b) Load vs. chord face displacement for Type 3 ($\beta = 1.0$)

Fig. 2-2. EHS T-connection behavior under branch axial compression.

2.3.1 Yield-Line Theory

The yield-line method of analysis is a classic tool used for this purpose, particularly for branches applying a load to the flat connecting face of a rectangular HSS. Yield-line theory is an application of the Principle of Virtual Work in which the external work done by an applied load on a connection is equated to, in this case, the internal energy dissipated within a flexural mechanism. Being an upper bound method of analysis, all possible collapse mechanisms should be considered, and the lowest of these upper bound solutions would govern. Rigid-plastic behavior is normally assumed, with all internal energy dissipated by rotation of discrete yield lines. Many different yield-line mechanisms can normally be postulated for a particular connection and loading, but any yield-line pattern must be kinematically admissible and compatible with the type of loading. Rectilinear and curvilinear patterns are possible, but all are idealizations of the real deformed shape.

Due to the continuity of the HSS chord (or column) member and its closed shape, the HSS connecting face does not act as a simply supported flat plate and it is effectively constrained or clamped at its “supports.” Thus, at deflections greater than the thickness, considerable in-plane membrane forces develop in the connecting face, particularly in the direction parallel to the chord member axis. These membrane forces raise the connection “plastic collapse load” to a level well above that expected of a simply supported, rigid-plastic plate. Consequently, the consideration of many alternate yield-line mechanisms (such as the consideration of yield fans) is usually not worthwhile because the connection yield load is, in reality, a lower bound on the real connection capacity. Furthermore, simple yield-line solutions for HSS connections, which lend themselves to design application, tend to make further conservative assumptions:

- Rounded corners are neglected for the chord, and a box shape is assumed. This increases the width of the “flat,” decreases the branch-to-chord width ratio, and leads to a lower predicted connection strength.
- The plastic hinges (yield lines) occurring in the chord side walls are assumed to be located at the external dimensions of the chord member, rather than at the mid-thickness of the wall. This increases the width of the chord flat, decreases the branch-to-chord width ratio, and leads to a lower predicted connection strength.
- The size of any welds (which are usually fillets) around the branch are neglected. This underestimates the footprint of the branch, decreases the branch-to-chord width ratio, and leads to a lower predicted connection strength.
- A uniform yield stress is used, associated with the measuring position in the flat, which neglects the higher yield stress associated with zones of greater cold working. This also leads to a lower predicted connection strength.

Many research papers present analyses of test results using so-called effective dimensions, which consider the geometric effects of the second and third bullet points, by considering the measured wall thickness and weld sizes, and predicted connection strengths are usually more accurate. For these reasons, the chord face flexural capacity is considerably greater than that estimated by simple, first-order, rigid-plastic, rectilinear yield-line theory with specified HSS dimensions. In recognition of this, AISC *Manual* Part 9 adopts a resistance factor of $\phi = 1.00$ for LRFD and $\Omega = 1.50$ for ASD (in keeping with the 2010 version of AISC *Specification* Chapter K) for connection available strengths determined by such yield-line mechanisms. Formulas from valid yield-line mechanisms, being analytical solutions and not empirically determined from tests, are not subject to limits of applicability. An example of a rectilinear yield-line pattern, drawn onto a laboratory-tested longitudinal branch plate-to-HSS connection under branch axial tension, is shown in Figure 2-3 (Kosteski and Packer, 2003a).

2.3.2 Rectangular HSS Chord T-, Y-, and Cross-Connections

Yield-line theory was first applied to a rectangular-to-rectangular HSS 90° T-connection, with the branch under axial loading, by Jubb and Redwood (1966). Moreover, the flexural strength of the connecting face of rectangular HSS T-, Y-, and cross-connections in general, with the branch under axial loading, in-plane bending, or out-of-plane bending, lends itself to determination by yield-line analysis. AISC *Manual* Part 9 Design of Connecting Elements considers “Plate Elements Subjected to Out-of-Plane Loads” and provides appropriate yield-line mechanisms for a plate (or HSS connecting face) under transverse load, in-plane moment, and out-of-plane moment. For these three branch loading conditions, solutions for the HSS connection nominal strength are given as AISC *Manual* Equation 9-44 for out-of-plane transverse loads, AISC *Manual* Equation 9-46 for in-plane moments, and AISC *Manual* Equation 9-48 for out-of-plane moments. These yield-line solutions use generic dimensions given in AISC *Manual* Figure 9-5, and they are even generalized for a branch potentially offset laterally from the chord center line. For the typical case of the branch centered on the chord axis, the HSS connection nominal strengths (P_n or M_n) can be determined for common connection types. For each, the available connection strength (ϕP_n or ϕM_n for LRFD; P_n/Ω or M_n/Ω for ASD) can be obtained using $\phi = 1.00$ and $\Omega = 1.50$.

HSS Branch under Axial Loading, Remote from Chord End

The yield-line pattern given in AISC *Manual* Figure 9-5(a) represents this loading case. The square or rectangular HSS branch 90° T- and cross-connection nominal strength,

under branch axial loading, is given by AISC *Manual* Equation 9-44 which can be rewritten as:

$$P_n = F_y t^2 \left(\frac{2\eta}{1-\beta} + \frac{4}{\sqrt{1-\beta}} \right) \quad (2-1)$$

where F_y is the chord yield stress, t is the chord face design thickness, β is the ratio of branch-to-chord width transverse to the chord (out of the plane of the connection), and η is the ratio of the length of contact of the branch with the chord (in the plane of the connection) to the chord width.

If the branch was inclined to the chord at an angle θ , the nominal strength, P_n , can be replaced by the normal component $P_n \sin \theta$ and η also becomes $H_b/B \sin \theta$, where H_b is the height of the rectangular HSS branch (measured in the plane of the connection) and B is the width of the rectangular HSS chord (measured transverse or normal to the plane of the connection). It is also known that a plastic hinge moment of resistance is reduced by normal stress, so the connection yield load could be expected to be reduced by axial and bending stresses acting in the chord connecting face. If the net normal stress is compressive, a further reduction in plastic capacity would be generated by a $P-\delta$ effect, but if the net normal stress is tensile, it would lessen the negative effect of the normal stress on the plastic moment of resistance. This complex influence of chord normal stress on connection capacity has been studied by a number of researchers (e.g., Cao et al., 1998a, 1998b; Voth and Packer, 2012b; Kuhn et

al., 2019) and is accounted for in the AISC *Specification* by the “chord stress effect” reduction factor, Q_f . This term has a value of 1.0 if the HSS connecting surface is in tension and a value less than or equal to unity [as given by AISC *Specification* Section K1.3(c)] if the chord connecting surface is in compression. The square or rectangular HSS T-, Y-, and cross-connection nominal strength under branch axial loading can hence be written as:

$$P_n \sin \theta = F_y t^2 \left(\frac{2\eta}{1-\beta} + \frac{4}{\sqrt{1-\beta}} \right) Q_f \quad (2-2)$$

Examination of Equation 2-2 will reveal that the connection nominal strength, P_n , tends to infinity as the width ratio, β , tends toward unity. In practice, this limit state is not likely to govern the design of connections with $\beta > 0.85$.

Longitudinal Plate Branch under Axial Loading, Remote from Chord End

Equation 2-2 can be reinterpreted for a longitudinal plate T-, Y-, and cross-connection under branch plate axial load (as shown in Figure 2-3), with β defined as t_p/B and η defined as l_b/B , where l_b is the plate contact length along the HSS connecting face, to give Equation 2-3. For such connections, with very low β , Q_f is redefined as per AISC *Specification* Equation K1-3 because Q_f (having been derived empirically from limited HSS-to-HSS K-connection experiments with

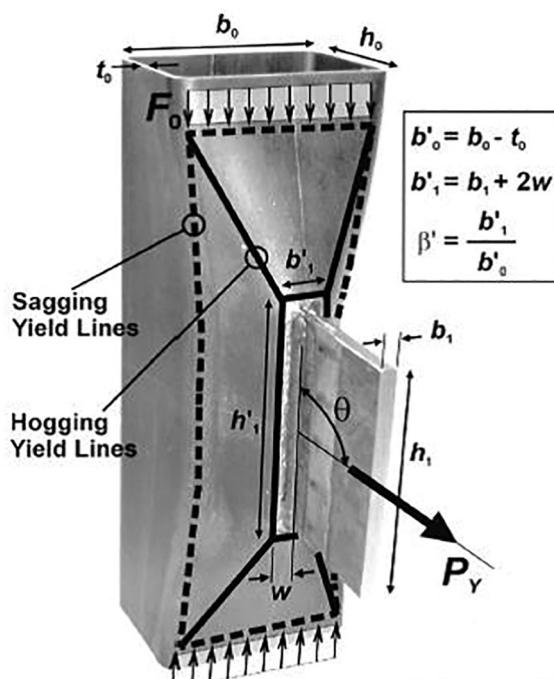


Fig. 2-3. Application of yield-line method applied to a rectangular HSS T-connection.

$\beta \geq 0.35$) was not originally intended for $\beta < 0.35$ (Packer et al., 1992).

$$R_n \sin \theta = F_y t^2 \left[\frac{2 \left(\frac{l_b}{B} \right)}{1 - \frac{t_p}{B}} + \frac{4}{\sqrt{1 - \frac{t_p}{B}}} \right] Q_f \quad (2-3)$$

HSS Branch or Longitudinal Plate Branch under Axial Loading, Near Chord End

The yield-line solutions represented by Equations 2-1, 2-2, and 2-3 presume that the chord is continuous for a considerable distance on either side of the branch, and thus the yield-line pattern is symmetrical about the center of the connection. If, however, the branch is close to a chord open end [Figure 2-4(a)] an asymmetrical failure mechanism is possible, as shown in Figure 2-4(b), for which the connection nominal strength by yield-line analysis is given by (Fan and Packer, 2017a, 2017b):

$$P_n \sin \theta = F_y t^2 \left[\frac{2\eta}{1-\beta} + \frac{2}{\sqrt{1-\beta}} + \frac{2l_{end}}{B(1-\beta)} \right] Q_f \quad (2-4)$$

where l_{end} is the end distance, measured from the branch toe to the chord open end. Equation 2-2, represented by the yield-line pattern of Figure 2-4(c), will govern over Equation 2-4 when:

$$l_{end} > B\sqrt{1-\beta} \quad (2-5)$$

Equation 2-5 is included as a limit of applicability in AISC *Specification* Section K1.4, as Equation K1-7, for rectangular HSS connections. Because Equation 2-5 is derived from a comparison of yield-line solutions, it can only be expected

to be valid in the parameter range for which yield-line mechanisms are appropriate (i.e., when $\beta \leq 0.85$) and potentially control. Recent research (Bu and Packer, 2020) has shown that the full connection strength can be achieved for $l_{end} \geq B$ for multiple failure modes covering a wide range of β and B/t values. If a branch or longitudinal plate is close to a chord open end, and the end distance is inadequate, the chord end can be capped as an alternative to taking a reduced connection nominal strength. A connection strength given by Equation 2-2, or higher, will then be achieved, provided the cap plate is welded on all sides (Fan and Packer, 2017a, 2017b). There is, therefore, no minimum end distance requirement in the case of a chord-end cap plate.

HSS Branch or Longitudinal Plate under In-Plane Bending, Remote from Chord End

The yield-line pattern given in AISC *Manual* Figure 9-5(b) can be used for this loading case, providing $\beta \leq 0.85$. The square or rectangular HSS branch (or longitudinal plate branch) 90° T- and cross-connection nominal flexural strength, under branch in-plane bending, is given by AISC *Manual* Equation 9-46. This can be rewritten as:

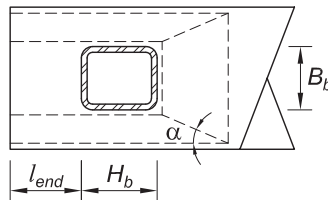
$$M_{n-ip} = F_y t^2 H_b \left(\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{1-\beta} \right) Q_f \quad (2-6)$$

HSS Square Branch under In-Plane Bending, Near Chord End

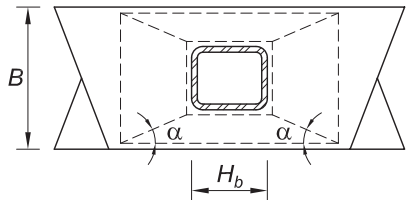
The yield-line patterns shown in Figure 2-5 are possible flexural mechanisms for this loading situation, for $\beta \leq 0.85$. Assuming square branches for simplification, the 90° T- and cross-connection nominal flexural strength, for $l_{end} \leq 0.5B$, will be given by Equation 2-7, corresponding to Figure 2-5(a). The end distance at which the maximum



(a) Experiments with small end distance



(b) Mechanism for Equation 2-4



(c) Mechanism for Equation 2-2

Fig. 2-4. Effect of axially loaded branch close to the end of an open HSS chord.

connection strength, represented by Equation 2-6, is achieved depends on the β value (as with Equation 2-5). Conservatively, this can be taken as $l_{end} \geq 1.5B$.

$$M_{n-ip} = F_y t^2 H_b \left(\frac{1-0.5\beta}{2\eta} + \frac{1}{\sqrt{1-\beta}} + \frac{\eta + \frac{l_{end}}{B}}{1-\beta} \right) Q_f \quad (2-7)$$

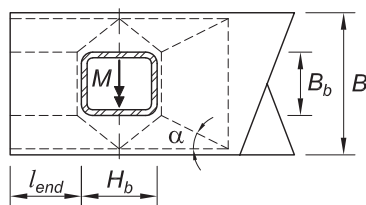
HSS Branch or Transverse Plate under Out-of-Plane Bending, Remote from Chord End

The yield-line pattern given in AISC *Manual* Figure 9-5(c) can be used for this loading case, providing $\beta \leq 0.85$. Note that for a transverse plate branch, the applied bending moment is out of the plane of the connection, but branch rotation takes place in the plane of the plate. (A transverse plate has almost zero flexural strength under an applied bending moment that is in the plane of the connection. Similarly, a longitudinal plate has almost zero flexural strength under an applied bending moment that is out of the plane of the connection.) The 90° T- and cross-connection nominal flexural strength, under branch out-of-plane bending, is given by AISC *Manual* Equation 9-48. For the usual case of the branch centered on the axis of the chord this can be rewritten as:

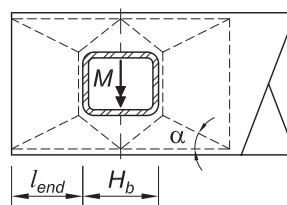
$$M_{n-op} = F_y t^2 \left[\frac{0.5H_b(1+\beta)}{1-\beta} + \sqrt{\frac{2BB_b(1+\beta)}{1-\beta}} \right] Q_f \quad (2-8)$$

HSS Branch under Combined Axial Load, In-Plane Bending, and Out-of-Plane Bending, Remote from Chord End

For T- and cross-connections, with branch(es) under combined axial load, in-plane bending, and out-of-plane bending, or any combination of these load effects, a linear interaction is recommended using Equations 2-2, 2-6, and 2-8 for the nominal connection strengths. This interaction is expressed in terms of available strengths in AISC *Specification* Table K4.2 (Equation K4-9).



(a) Controlling mechanism for $l_{end} \leq 0.5B$



(b) Controlling mechanism for $0.5B < l_{end} < 1.5B$

Fig. 2-5. Possible yield-line patterns for square branches under in-plane bending.

2.3.3 Rectangular HSS Chord K-Connections

The principal failure mode for gapped (but not overlapped) K- and N-connections, with small to medium β ratios, is again chord wall plastification. In the case of gapped K-connections, the actions of the compression branch and tension branch develop a “push-pull” mechanism on the chord connecting face, usually resulting in large deformations as shown in Figure 2-6. Yield-line analysis has also been applied to gapped K-connections (Packer et al., 1980, 1982) but the solutions are not directly suited for design and have been used instead to inform semi-empirical design expressions. The nominal connection strength, expressed as an axial force in either branch, is given by Equation K3-7 in AISC *Specification* Table K3.2. Because this connection capacity is fitted to a broad range of experimental tests, it covers the entire β range but is subject to the limits of applicability in Table K3.2A, which bound the range of parameters validated. The connection available strength is obtained using ϕ and Ω factors (AISC *Specification* Table K3.2) appropriate for the calibration to ultimate strengths from experiments.

It is instructive to note how this connection limit state, which governs the design of most gapped K-connections, can be negated by connection reinforcement. Figure 2-7 shows the chord face deformations associated with a typical gapped K-connection and the same connection with the chord filled with concrete. The latter prevents the push-pull mechanism developing, causing the compression branch to bear on the concrete and the tension branch to act independently. For the tension branch, only two possible limit states then apply: chord shear yielding (punching) and branch local yielding due to uneven load distribution (AISC *Specification* Table K3.2).

2.3.4 Round HSS Chord Connections

With rectangular HSS connections, the chord connecting face acts like a flat plate under transverse load, supported by two remote webs, and the connecting face acts relatively independent of the other three sides. However, if the main member is a round HSS, the chord behaves like a closed ring

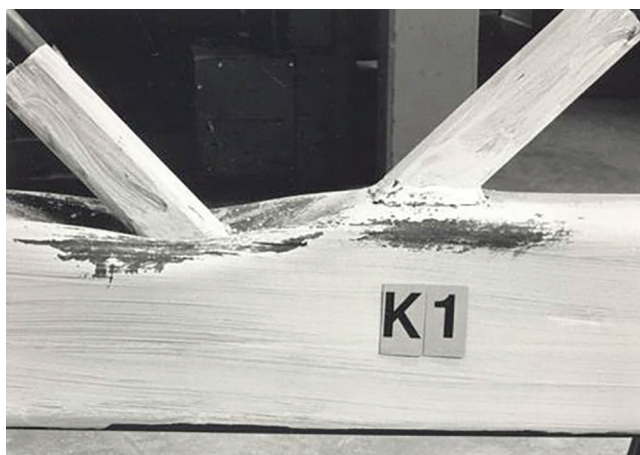
under transverse load and chord plastification results in distortion of the entire chord cross section. Examples of round HSS chord connections experiencing severe plastification are shown in Figure 2-8.

Yield-line analysis has been applied to welded HSS connections with a round chord since the 1980s (Makino et al., 1989), but the results are ill-suited for design. Instead, a ring model for chord plastification, originally developed by Togo (1967), has been a popular basis for round HSS connection formulas. This model assumes—for example, in a round-to-round cross-connection—that most of the loading is transferred at the branch “saddles” (the lowest contact points of the branch intersection length) where the connection is

stiffest, and the branch stresses are highest [Figure 2-9(a)]. The branch force normal to the chord is then converted into two line-loads at the saddles, with an effective length along the chord. A plastic collapse mechanism is then assumed with hinges having this effective length and located (for a round-to-round cross-connection) at the four saddle positions and the two chord-mid-depth locations [Figure 2-9(b)]. It can then be shown (Wardenier et al., 2010) that the cross-connection nominal strength takes the form of Equation 2-9, where C_1 and C_2 are constants to be determined by best-fits to experimental and numerical tests. This is the form of AISC *Specification* Equation K2-1a (Table K2.1) and Equation K3-3 (Table K3.1). Further adjustments to this model



Fig. 2-6. Longitudinal cross section through an HSS K-connection at ultimate load.



(a) K-connection with unfilled chord



(b) K-connection with concrete-filled chord

Fig. 2-7. Failure modes for gapped K-connections.

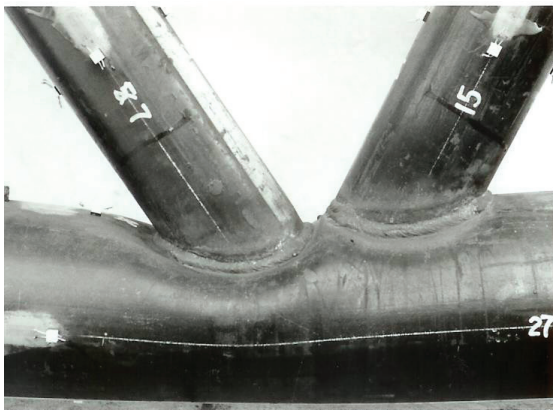
formula are then needed to suit other round HSS connection types, as is evident in AISC *Specification* Chapter K. With Tables K2.1, K3.1, and K4.1 all containing equations for chord plastification that are empirically fit to test results, each of these tables has limits of applicability that represent the range of parameters over which the nominal strength formulas have been validated. The connection available strengths are obtained using ϕ and Ω factors appropriate for the calibration to ultimate strengths in tests.

$$P_n \sin \theta = F_y t^2 \left[\frac{C_1}{(1 - C_2 \beta)} \right] Q_f \quad (2-9)$$

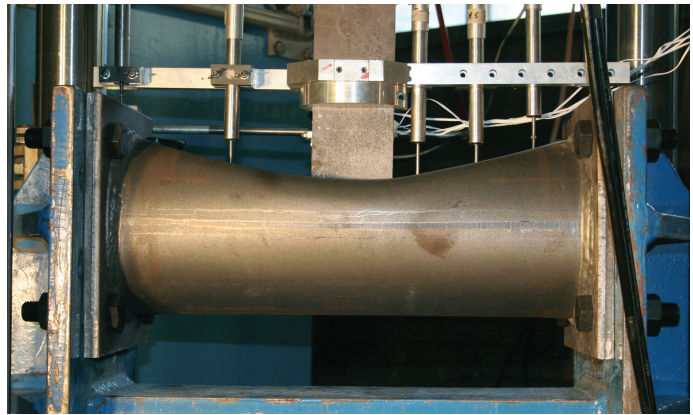
2.4 LIMIT STATE OF SHEAR YIELDING (PUNCHING SHEAR)

AISC *Specification* Section J10.10 requires that the limit state of shear be considered, in accordance with Section J4.2, when there is a force component applied normal to the plane of a plate element.

This failure mode, often termed simply “punching shear,” may govern in various HSS connections, particularly for medium to high branch-to-chord width ratios, β . With this failure mechanism, a patch of chord material pulls out (or punches in) around the footprint of a branch member at the toe of the weld to the chord. Such a failure can occur under either a tension branch or a compression branch member,

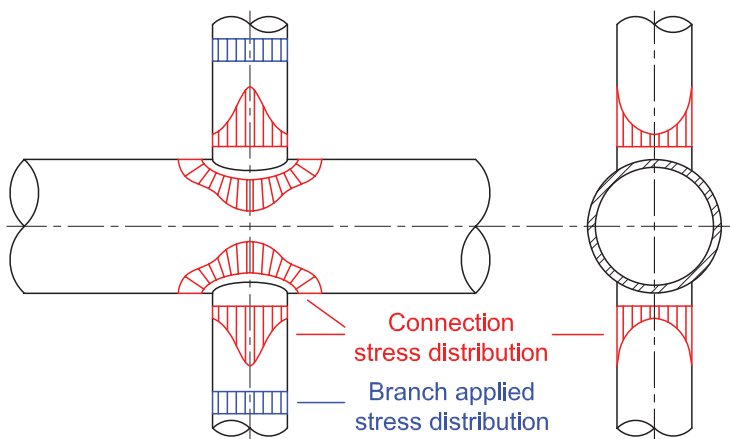


(a) Balanced gapped K-connection at ultimate load

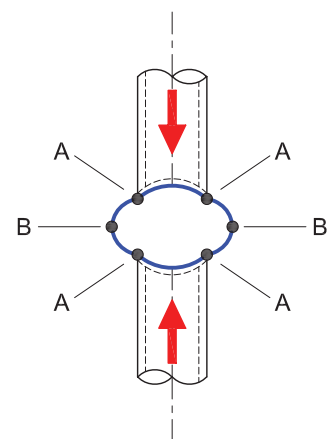


(b) Longitudinal plate connection at ultimate load

Fig. 2-8. Chord plastification for round HSS connections.



(a) Stress distribution in branches and chord



(b) Plastic hinges assumed in “ring model”

Fig. 2-9. Cross-connection under branch axial loading.

providing the branch is physically capable of shearing through the chord wall [i.e., the branch outside width (or diameter) is less than the inside width (or diameter) of the chord]. Shear failure strength is typically calculated in steel design codes based on either the ultimate shear stress ($F_u/\sqrt{3} \approx 0.6F_u$) associated with shear rupture, with a corresponding low resistance factor ($\phi = 0.75$ in AISC *Specification* Eq. J4-4), or the shear yield stress ($F_y/\sqrt{3} \approx 0.6F_y$) with a resistance factor close to unity ($\phi = 1.00$ is used in AISC *Specification* Equation J4-3). The AISC *Specification* uses both approaches in Section J4, but AISC *Manual* Part 9 Section “Plate Elements Subjected to Out-of-Plane Loads” affirms the use of the shear yield stress approach for the nominal punching shear strength of HSS connections. Values of $\phi = 1.00$ (for LRFD) and $\Omega = 1.50$ (for ASD) are accordingly adopted for available punching shear strength of plate-to-HSS connections in Chapter 8.

2.4.1 Rectangular HSS Chord Connections

The stiffness around the branch is generally nonuniform for rectangular HSS chord connections, so only certain parts of the total perimeter can be assumed to be effective for resisting the punching shear force normal to the connecting face.

For T-, Y-, and cross-connections, as well as transverse plate connections, the branch edges parallel to the chord side walls are the stiffest part of the perimeter, and the branch load is concentrated in these portions. As can be seen in Figure 2-10, rupture initiates at the plate extremities close to the chord side walls. Hence, for the branch dimensions transverse to the chord member, only a portion is treated as being effective, and thus contributing to a total effective length for punching shear. This is given as AISC *Manual*

Equation 9-43, which can be written for a T-, Y-, or cross-connection under branch axial loading, P_n , (also see AISC *Specification* Commentary Table C-K1.1) as:

$$P_n \sin \theta = 0.6F_y t \left(\frac{2H_b}{\sin \theta} + 2B_{ep} \right) \quad (2-10)$$

where the punching effective width of a branch transverse to the chord has been determined experimentally (Wardenier, 1982) by Equation 2-11. This effective-width term occurs in a number of rectangular HSS connection formulas and is hence displayed in AISC *Specification* Section K1.2a as Equation K1-2.

$$B_{ep} = \left(\frac{10}{B/t} \right) B_b \leq B_b \quad (2-11)$$

Equation 2-11 shows that the portion of the branch width, B_b , that is effective is highly dependent on the slenderness of the chord connecting face, B/t . The stockier the chord section, the larger the effective width. The coefficient of 10 in Equation 2-11 represents an available strength estimate and has been reduced from a higher “nominal strength” coefficient (Davies and Packer, 1982). Equation 2-10 can also be rewritten for a transverse plate branch by replacing H_b by the plate thickness, t_p .

For gapped K-connections, the gap size has a significant influence on the effective width of the branch transverse walls adjacent to the gap area. For a very large gap size, the stiffness distribution around a branch approximates that of a T- or Y-connection. However, for a relatively low gap size, the gap region becomes highly loaded, with force being directly transferred from one branch to the other, and the



Fig. 2-10. Chord shear yielding (punching) for plate-to-rectangular HSS connection at ultimate load.

transverse walls of both branches adjacent to the gap become fully effective. Thus, three sides of each branch become fully effective. Then, the nominal punching shear strength of a typical rectangular HSS gapped K-connection can be expressed (in terms of the axial load in a branch, P_n) as:

$$P_n \sin \theta = 0.6 F_y t \left(\frac{2H_b}{\sin \theta} + B_b + B_{ep} \right) \quad (2-12)$$

Equation 2-12 is expressed (but with different symbols) in AISC *Specification* Equation K3-8 and is subject to the limits of applicability of Table K3.2A, particularly with regard to the gap dimension. For the connection available strength, AISC *Specification* Equation K3-8 uses $\phi = 0.95$ (for LRFD) and $\Omega = 1.58$ (for ASD) to be in accord with prior editions of the AISC *Specification* and its historical introduction in AWS D1.1/D1.1M. As noted in Table K3.2, this limit state will not control (and therefore need not be checked) for square branches. Tests have shown that the presence of chord normal stress only has a minor effect, and therefore Equations 2-10 and 2-12 do not contain the chord stress factor, Q_f .

2.4.2 Round HSS Chord Connections

In the same manner as for rectangular HSS chord connections, the nominal punching shear strength can be obtained by the product of the shear yield stress, the thickness of the chord wall, and the branch effective perimeter. Due to variations in chord stiffness beneath the branch, the punching shear stress distribution will be nonuniform, even after yielding. However, tests on round HSS connections have shown that, within the range of validity examined, the full branch

perimeter can be considered as effective. This failure mode is depicted in Figure 2-11. The connection nominal punching shear strength for round-to-round HSS T-, Y-, cross-, and gapped K-connections, expressed as an axial force in a branch, P_n , is hence:

$$P_n \sin \theta = 0.6 F_y t \pi D_b \left(\frac{1 + \sin \theta}{2 \sin \theta} \right) \quad (2-13)$$

The $(1 + \sin \theta)/2 \sin \theta$ term increases the branch perimeter, πD_b , to an elliptical length (considering the intersection with a flat plate at the chord crown), for branch angles less than 90° . This term is also used in AWS D1.1/D1.1M, clause 10.5.5, as a K_a factor. Equation 2-13 is given as AISC *Specification* Equation K3-1 and is subject to the limits of applicability of Table K3.1A. In common with rectangular HSS chord connections, $\phi = 0.95$ (for LRFD) and $\Omega = 1.58$ (for ASD) for available punching shear strength, and the Q_f factor is not applied.

2.5 LIMIT STATE OF LOCAL YIELDING OF BRANCH DUE TO UNEVEN LOAD DISTRIBUTION

This failure mode applies to transverse plates or transverse walls of a rectangular HSS across a rectangular HSS chord member. It is analogous to the effective punching shear width concept described in Section 2.4.1, except now the effective width is applied to the transverse element itself rather than the HSS chord, resulting in premature failure of that element. With this limit state, parts of the transverse element yield prematurely at a load less than the gross area of the element multiplied by the yield stress. This is caused



Fig. 2-11. Chord shear yielding (punching) for a round HSS T-connection at ultimate load.

principally by the flexibility of the base (the HSS connecting face) to which the transverse element is attached, rendering only part of the element width as effective. Thus, the branch effective width depends on the properties of both the branch and the chord. The transverse plate-to-HSS “effective width” phenomenon was studied in the late 1970s and early 1980s and is encapsulated in the IIW (1989) recommendations. This limit state is possible for any β ratio.

In tension, local yielding and eventually premature fracture of the transverse element occurs, as shown in Figure 2-12(a) for a full-width transverse plate connection to an HSS chord with high B/t . The parts of the plate width that are fully effective are illustrated by the two outer dark regions where the whitewash has flaked off. At the plate local yield load, these outer regions are simplified to two rectangular stress blocks, each with an effective width of $B_e/2$, where B_e is the total effective width of the transverse element. [See Figure 2-12(b) and AISC *Specification* Commentary Figure C-K1.3.] A lower bound to experimental results (Wardenier, 1982; Davies and Packer, 1982) is given by AISC *Specification* Equation K1-1, shown here as Equation 2-14:

$$B_e = \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b \quad (2-14)$$

Equation 2-14 is similar to Equation 2-11 and again shows that the total effective width becomes larger as B/t decreases. This also underscores the design objective of achieving high HSS connection strengths by using stocky chord members.

In compression, local yielding usually results in premature local buckling of the element. This limit state is the most common failure mode for rectangular HSS overlapped

K-connections, wherein local buckling of the compression branch occurs (see Figure 2-13). The local yielding due to uneven load distribution limit state check is applied to many HSS connections with rectangular chord members: T-, Y-, and cross-connections under branch axial loading; gapped and overlapped K-connections; T- and cross-connections under branch moment loading; and axially loaded transverse branch plate-to-HSS connections.

For axially loaded T-, Y-, and cross-connections, there are two transverse branch wall elements—hence, two B_e terms—and the connection nominal strength is given by:

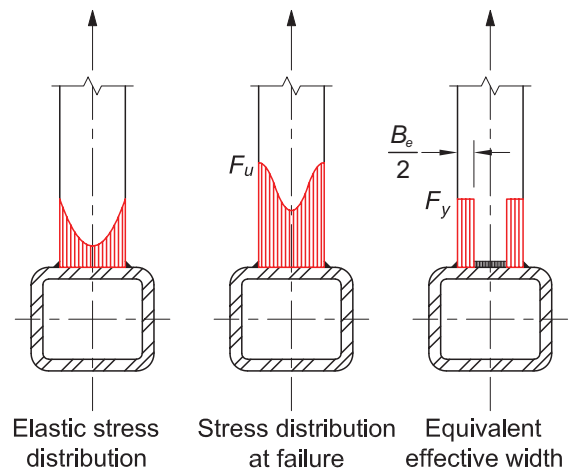
$$P_n = F_{yb} t_b (2H_b + 2B_e - 4t_b) \quad (2-15)$$

This equation sums the yield strengths of the four branch walls (assuming a box shape); hence, the $4t_b$ term avoids the corners being counted twice. There is no $\sin\theta$ term in Equation 2-15 because it is the force in the branch that causes branch yielding, unlike chord punching shear where the force normal to the chord face causes failure (Equation 2-10). An alternative to Equation 2-15 based on a box shape is to take $P_n = F_{yb} A_{eb}$, where A_{eb} is a branch effective area equal to $[(H_b + B_e)/(H_b + B_b)] A_b$, with A_b being the gross area of the branch. This is presented in AISC *Specification* Commentary Table C-K1.1 and might better accommodate the rounded corners of cold-formed HSS (but there is still some empiricism associated with B_e).

For gapped K-connections, providing the gap dimension satisfies the limits of applicability of AISC *Specification* Table K3.2A, Equation 2-15, can be rewritten by replacing $2B_e$ with $B_b + B_e$, which leads to AISC *Specification* Equation K3-9 for this limit state. When the branches are square



(a) Tension-loaded transverse branch plate at failure



(b) Effective width model for a transverse element

Fig. 2-12. Local yielding of branch in tension due to uneven load distribution.

or when $B/t \geq 15$, AISC *Specification* Table K3.2 points out that this limit will not control the design of gapped K-connections, and hence need not be checked.

For overlapped K-connections, the principle of summing the yield strengths of the four branch walls (assuming a box shape) is again used, but in this case, the transverse walls of the overlapping branch member have effective width terms that need to consider the relevant landing surface. For example, for the overlapping branch (member i) of a partially overlapped K-connection, one transverse wall lands on the chord face, and Equation 2-14 applies as written, with $F_{yb}t_b$ pertaining to the overlapping branch—hence, $F_{ybi}t_{bi}$ —and the effective width of this branch transverse wall is termed B_{eoi} and is given as AISC *Specification* Equation K3-13. Because the other transverse wall of the overlapping branch lands on the overlapped branch (member j), Equation 2-14 needs to be rewritten as:

$$B_{eov} = \left(\frac{10}{B_{bj}/t_{bj}} \right) \left(\frac{F_{ybj}t_{bj}}{F_{ybi}t_{bi}} \right) B_{bi} \leq B_{bi} \quad (2-16)$$

where the subscript i refers to the overlapping branch, and the subscript j refers to the overlapped branch. Equation 2-16 is given as AISC *Specification* Equation K3-14 in Table K3.2.

For determining connection available strengths for this limit state, $\phi = 0.95$ (for LRFD) and $\Omega = 1.58$ (for ASD) for all types of connections. These factors were carried over to the AISC *Specification* from historical use in AWS D1.1/D1.1M, but they do serve to reduce the computed

cross-sectional area of HSS branches from the theoretical box section area in the formulas to a more practical cold-formed HSS area (recognizing the rounded corners).

2.6 LIMIT STATE OF LOCAL YIELDING OF CHORD SIDE WALLS

This limit state applies to the two side walls of a square or rectangular HSS chord or column member when subjected to a localized compressive or tensile force, acting on just one side of the member or on both sides of the member (as shown in Figure 2-14, where transverse compression was applied). The concentrated load may be due to local bearing or a moment connection. Because the force needs to be applied to the HSS sidewalls, rather than the connecting face, this failure mode has the potential to be critical when the branch width is close to, or equals, the chord (main member) width. Although this limit state has traditionally only been checked when $\beta \approx 1.0$, Kim and Lee (2021) have shown that it may govern over chord face failure at lower β values, particularly for cold-formed rectangular HSS that have large corner radii.

The nominal strength under such loading is given by AISC *Specification* Section J10.2, where the normal application is presumed to be the single web of a wide-flange section or a similar, built-up, I-shaped section. In that case, the applied load is assumed to disperse at a slope of 2.5:1 from the outer face of the flange to the web toe of the fillet (or the “ k -line”). The k -line for an HSS is taken to correspond with the termination of the outside corner radius that, if unknown, may be



Fig. 2-13. Local yielding of a branch in compression due to uneven load distribution.

conservatively assumed in this case to be $1.5t$ [as implied in AISC *Specification* Section B4.1b(d) and discussed in the AISC *Specification* Commentary].

When the concentrated force is applied at a distance from the member end of $l_{end} > H$, dispersion of the applied load to both sides of the bearing length can be assumed, and AISC *Specification* Equation J10-2 may be rewritten for an HSS with two webs and $k = 1.5t$ as:

$$P_n \sin \theta = 2F_y t (7.5t + l_b) \quad (2-17)$$

where l_b is the bearing length along the HSS. For a T-, Y-, or cross-connection, this length would be equal to $H_b/\sin \theta$.

When the concentrated force is applied at a distance from the member end of $l_{end} \leq H$, dispersion of the applied load to only one side of the bearing length is assumed, and AISC *Specification* Equation J10-3 may be rewritten for an HSS with two webs and $k = 1.5t$ as:

$$P_n \sin \theta = 2F_y t (3.75t + l_b) \quad (2-18)$$

The available connection strength for this limit state is obtained from Equation 2-17 or Equation 2-18 with $\phi = 1.00$ (for LRFD) and $\Omega = 1.50$ (for ASD), per AISC *Specification* Section J10.2. These ϕ and Ω values are also supported by recent research (Wei and Packer, 2021; Rudman et al., 2022). Equations 2-17 and 2-18 are analytical yield models and are not subject to parameter limits of applicability. For concentrated compression forces acting over the full HSS width ($\beta = 1.0$), experiments and numerical modeling have indicated that this failure mode is likely to control over web crippling and web buckling when either the bearing length is very low (such as for transverse plate-to-HSS connections)

or when the chord sidewall is stocky ($H/t < \sim 15$ to 20) (Kuhn et al., 2019).

2.7 LIMIT STATE OF LOCAL CRIPPLING OF CHORD SIDE WALLS

This limit state applies to the two side walls of a square or rectangular HSS chord or column member when subjected to a localized compressive force, acting on just one side of the member or on both sides of the member (i.e., under transverse compression). The concentrated load may be due to local bearing or a moment connection. As with the limit state of local yielding of the chord sidewalls, the branch-to-chord width ratio, β , needs to be relatively close to unity. This failure mode is again based on the single web of a wide-flange section or a similar, built-up, I-shaped section, and is defined by crumpling of the web into buckled waves directly beneath the load. The AISC *Specification* Commentary notes that this failure mode should prevail with more slender webs, whereas web local yielding (Section 2.6) in the same web region should prevail with stocky webs.

The nominal strength for this limit state is given by AISC *Specification* Section J10.3, again based on research on the single web of an I-shaped section (Roberts, 1981). When the concentrated force is applied at a distance from the member end of $l_{end} \geq H/2$, or $l_{end} < H/2$ but the webs are supported, AISC *Specification* Equation J10-4 may be rewritten for an HSS, which has equal flange and chord thickness plus two webs, as:

$$P_n \sin \theta = 1.6t^2 \left(1 + \frac{3l_b}{H} \right) \sqrt{EF_y} Q_f \quad (2-19)$$

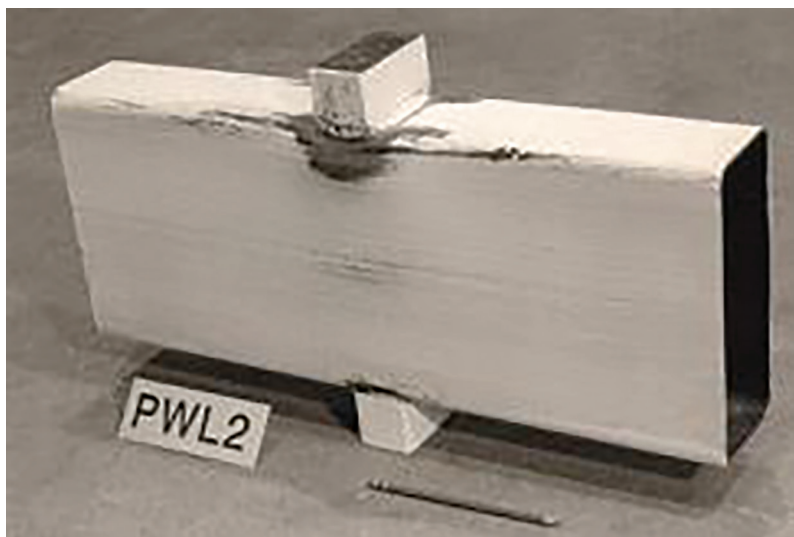


Fig. 2-14. Local yielding of chord side walls in compression.

In Equation 2-19, the term H corresponds to the full nominal depth of the member, as defined in AISC *Specification* Section J10.3. This is a slight (and conservative) change from the presentation in the 2010 AISC *Specification* Equation K2-10 or the 2005 AISC *Specification* Equation K2-16, where $(H - 3t)$ was used.

When the concentrated force is applied at a distance from the member end of $l_{end} < H/2$ and the HSS webs are also unsupported, the AISC *Specification* gives alternate forms (Equations J10-5a and J10-5b) for reduced strength that depend on the bearing length. For application to HSS these become:

For $l_b/H \leq 0.2$,

$$P_n \sin \theta = 0.8t^2 \left(1 + \frac{3l_b}{H} \right) \sqrt{EF_y} Q_f \quad (2-20)$$

For $l_b/H > 0.2$,

$$P_n \sin \theta = 0.8t^2 \left(0.8 + \frac{4l_b}{H} \right) \sqrt{EF_y} Q_f \quad (2-21)$$

Q_f is given by Equation K1-4 in AISC *Specification* Section K1.3. The available connection strength for this limit state is obtained from Equation 2-19, 2-20, or 2-21, as applicable, with $\phi = 0.75$ (for LRFD) and $\Omega = 2.00$ (for ASD), per AISC *Specification* Section J10.3.

The AISC *Specification* Commentary notes that all the I-section tests on which these web crippling provisions are based were conducted on bare steel beams. For a considerable number of HSS T- and cross-connection tests in branch compression with the branches welded, not near the chord end, and with $\beta = 1.0$, the failure mode of web local crippling has never been found to govern up to $H/t = 57$ (Wei and Packer, 2021). Recent HSS connection design guidance by IIW (2012), ISO (2013), and CIDECT (Packer et al., 2009; Wardenier et al., 2010) also does not include this limit state. However, these international guides are constrained to specific limits of parameter applicability and particular connection types, in addition to the expressions for the web yielding and web buckling limit states being different. Thus, there may be situations in which the limit state of HSS web local crippling may control in compression over web local yielding (Section 2.6) and web buckling (Section 2.8), such as for very slender HSS webs combined with low bearing lengths.

2.8 LIMIT STATE OF BUCKLING OF CHORD SIDE WALLS

Another possible failure mode for the two side walls of a square or rectangular HSS chord (or main) member is buckling, when subjected to transverse compression, as illustrated in Figure 2-15. This is therefore a limit state check for HSS cross-connections with branches in compression and

$\beta \approx 1.0$, but it is not applied to T- or Y-connections. It also applies to the compression region of beam-to-HSS column moment connections under gravity loading. The dark lines in the tested connection shown in Figure 2-15(a) indicate regions of large plastic strain causing flaking of the white-wash. As with the limit state of yielding of the chord side-walls, Kim and Lee (2021) have shown that chord sidewall buckling may govern over chord face failure at β values lower than unity, particularly for cold-formed rectangular HSS that have large corner radii.

This failure mode resembles the buckling of a rectangular cross section column about its weak axis, with an ill-defined effective width. International HSS design guidance by IIW (2012), ISO (2013), and CIDECT (Packer et al., 2009; Wardenier et al., 2010) in fact utilizes a column buckling curve for this limit state check. AISC *Specification* Commentary Section J10.5 also advocates this approach (using Chapter E of the AISC *Specification*) for when $\eta = l_b/H \geq 1.0$, where l_b is the bearing length. In such situations it is recommended that the slenderness of each web, KL/r , be determined using an effective length factor, K , equal to 1.0 as recommended by AISC *Specification* Appendix 7, applied to a column length, L , of the sidewall flat dimension equal to $(H - 3t)$. As discussed in AISC *Specification* Commentary Section J10, the $(H - 3t)$ height corresponds to the clear distance between flanges, less the inside HSS corner radius. When the latter is not known, as is usual, an outside corner radius of $1.5t$ is typically assumed. For box sections, the clear inside depth between the flanges may be used in lieu of $(H - 3t)$. The radius of gyration, r , of a rectangular HSS wall can be computed as $t/\sqrt{12}$. The available flexural buckling strength of the two HSS side walls can then be calculated from AISC *Specification* Section E3, with allowance for an inclined branch (Packer et al., 2009; IIW, 2012; ISO, 2013; Wei and Packer, 2021), using:

$$\frac{KL}{r} = 3.46 \left(\frac{H}{t} - 3 \right) \sqrt{\frac{1}{\sin \theta}} \quad (2-22)$$

and a “column” gross cross-sectional area given by $t(7.5t + l_b)$ from Equation 2-17 for $l_{end} > H$ or $t(3.75t + l_b)$ from Equation 2-18 for $l_{end} \leq H$. For consistency with the approach in Equation 2-23, the chord stress influence factor, Q_f , should also be employed when determining the available compressive strength.

For short bearing lengths, with $l_b/H < 1.0$ and the load applied at interior points with $l_{end} \geq H/2$, the AISC *Specification* advocates an empirical web buckling solution derived for wide-flange or I-section webs, which is independent of the bearing length, given by AISC *Specification* Equation J10-8. This equation can be rewritten to express the nominal compressive strength of two HSS webs (again using notation for HSS branches) by:

$$P_n \sin \theta = \left(\frac{48t^3}{H - 3t} \right) \sqrt{EF_y} Q_f \quad (2-23)$$

Q_f is given by Equation K1-4 in AISC *Specification* Section K1.3. When $l_b/H < 1.0$ and the load is applied near the member end such that $l_{end} < H/2$, the nominal strength given by Equation 2-23 is to be reduced by 50%. In harmony with the design of compression members in AISC *Specification* Chapter E, the available compressive strength for the limit state of buckling of the chord side walls is determined with $\phi = 0.90$ (for LRFD) and $\Omega = 1.67$ (for ASD), per AISC *Specification* Section J10.5. Because these expressions for web buckling strength are adapted from research on wide-flange or I-section webs, there are no known limits of applicability for HSS.

Following the discussion in Section 2.6, for transverse compression forces acting over the full HSS width ($\beta = 1.0$), this failure mode is likely to control over both web crippling and web local yielding when the bearing length is moderate to large or when the chord side walls are slender. A recent study of experimental and numerical data on matched-width, transversely compressed rectangular HSS (Kuhn et al., 2019) has shown that the limit state of buckling of the

chord sidewalls will in fact control for $l_b/H > 0.25$, rather than > 1.0 . The column buckling approach (using AISC *Specification* Chapter E in conjunction with Equation 2-22) was thus recommended by Wei and Packer (2021) and Rudman et al. (2022) for this limit state when $l_b/H > 0.25$. Moreover, correlation with test data shows that the chord web can even be treated as a fixed-end column rather than pin-ended, hence using $K = 0.65$, providing the connection is constrained from any lateral instability (Lan et al., 2021; Wei and Packer, 2021; Rudman et al., 2022).

2.9 LIMIT STATE OF SHEARING OF CHORD CROSS SECTION

If a chord (or main) member is subject to a shearing force across the member, a failure mode of cross section shearing is possible, as illustrated in Figure 2-16. In general, this limit state can apply to either a rectangular or round HSS. The nominal strength is given by AISC *Specification* Chapter G, with the chord shear area, A_v , being taken as half the gross area for round HSS (Equation G5-1), or $2t(H - 3t)$ for rectangular HSS per Section G4.

For most manufactured round HSS sizes, shear yielding controls over shear buckling (both of which are covered



(a) Cross-connection with web buckling



(b) Cross section through connection shown in part (a)

Fig. 2-15. Buckling of chord side walls under transverse compression.

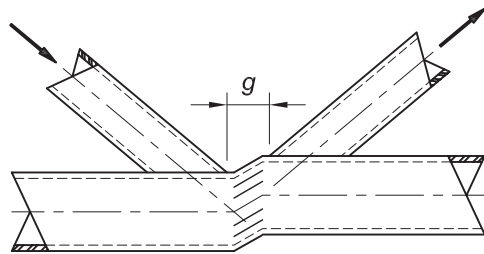
in AISC *Specification* Section G5) and the nominal shear strength (expressed in terms of branch force vertical components) is given by:

$$P_n \sin \theta = 0.6F_y A_v \quad (2-24)$$

For most manufactured rectangular HSS sizes—in fact, for $(H - 3t)/t \leq 59$ —shear yielding controls over shear buckling (both of which are covered in AISC *Specification* Sections G2.2 and G4) and the nominal shear strength is again given by Equation 2-24. For rectangular HSS-to-HSS truss-type connections, there are two instances where chord shearing needs to be considered:

1. Gapped K-connections, where a gap, g , as shown in Figure 2-16(a), exists between the footprints of the two branch members (one in compression, one in tension) on one side of the chord. This limit state is included in AISC *Specification* Table K3.2 for rectangular HSS gapped K-connections, where it is pointed out that it need not be checked for square chords (because it has only been found to be a potential governing limit state for $H < B$). The use of $2t(H - 3t)$ for the total shear area should be conservative, as international HSS design guidance by IIW (2012), ISO (2013), and CIDECT (Packer et al., 2009; Wardenier et al., 2010) allows the full web height plus a portion of the connecting face to be taken as the shear area in rectangular HSS gapped K-connections.
2. Cross-connections with inclined branches, where a projected gap exists and a shear plane through the chord can be identified, as in Figure 2-16(b). For a gap, g , to be produced, with two aligned (co-axial) branches, $\cos \theta > H_b/H$.

The available strength for the limit state of shearing of the cross section is determined with $\phi = 0.90$ (for LRFD) and $\Omega = 1.67$ (for ASD), per AISC *Specification* Section G1.



(a) Rectangular HSS gapped K-connection

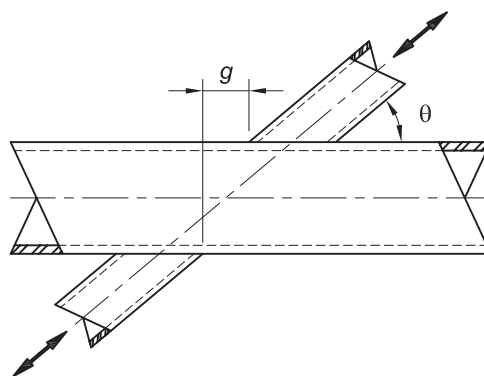
2.10 LIMIT STATE OF LOCAL BUCKLING OF CHORD CONNECTING FACE

A known failure mode for rectangular HSS K-connections—and, in particular, overlapped K-connections—is local buckling of the chord connecting face behind the heel of the tension branch, as illustrated in Figure 2-17. Adjacent to such connections, the engineer is expected to have performed a member check for the combined effect of chord axial loading plus bending on the chord cross-section capacity [locations indicated by (2) in Figure 2-18], with the bending moment computed from an overall structural analysis.

However, the axial force and bending moment are not distributed in a regular manner over the entire chord cross section because shear lag causes a disproportionate load in the chord connecting face in this region. This overstressing could potentially lead to chord local buckling. This limit state check is omitted in AISC *Specification* Table K3.2 because research has shown that premature failure due to shear lag can be precluded by placing strict wall slenderness limits on the chord connecting face. Hence, in AISC *Specification* Table K3.2A, B/t has an upper limit of 35 for gapped K-connections and 30 for overlapped K-connections. For rectangular HSS K-connections with chords having B/t beyond these limits, this limit state must be considered using a rational method of analysis.

2.11 LIMIT STATE OF SHEAR YIELDING OF OVERLAPPED BRANCHES

As noted in Section 2.5, local yielding of a branch due to uneven load distribution is the classic failure mode for rectangular HSS overlapped K-connections. The failure location is indicated symbolically by line (1) in Figure 2-18. It has been postulated that another failure mode, of branch shearing along line (3) caused by the horizontal components of the branch forces, is possible in some overlapped



(b) Rectangular HSS cross-connection

Fig. 2-16. Chord shear failure mode examples.

K-connections. This failure mode has not been observed in laboratory tests, possibly because the chord connecting face is also invoked in the shear resistance. A numerical study by Bu and Packer (2023) also found that the limit state of branch shear failure along line (3) could be ignored, within the bounds of the AISC *Specification*. This theoretical limit state may need to be considered if designing beyond the limits of AISC *Specification* Table K3.2A. Wardenier et al. (2016) give a proposed nominal connection strength for this limit state. Local yielding of the chord at location (2) is possible, due to the combined stress caused by axial loading

and bending in the chord, but this can be deemed a routine member capacity check.

2.12 LIMIT STATE OF CHORD DISTORTIONAL FAILURE

In rectangular HSS T- and cross-connections, where out-of-plane bending action is applied to the T-connection branch or to both branches of a cross-connection producing moments in the same sense, torsion on the chord section results, and it is possible for the chord to distort into a rhomboidal shape.



Fig. 2-17. Local buckling of chord due to uneven load distribution.

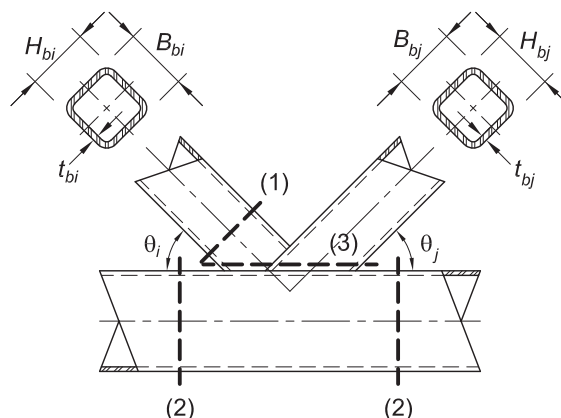


Fig. 2-18. Potential failure locations for overlapped K-connections.

For connections that are near full width ($\beta > 0.85$), this limit state will dominate failure unless distortion of the cross section is prevented. The plastic torsional capacity of a thin-walled, rectangular HSS box-shaped cross section is given by:

$$M_t = \frac{F_y}{\sqrt{3}} [2t(B-t)(H-t)] \quad (2-25)$$

Out-of-plane bending moment applied to the branch(es) of such connections, however, causes both torsion and distortion of the chord shape over a length of influence. Niemi (1986) derived a plastic mechanism capacity for rectangular HSS which was subsequently adopted by EN 1993-1-8 (CEN, 2005), and this is included in AISC *Specification* Table K4.2 as Equation K4-7:

$$M_{n-op} = 2F_y t \left[H_b t + \sqrt{BHt(B+H)} \right] \quad (2-26)$$

If $\beta = 1.0$ and all HSS are identical square shapes, AISC *Specification* Equation K4-7 (i.e., Equation 2-26) simplifies to:

$$M_{n-op} = 2F_y t^2 B (2\sqrt{\gamma} + 1) \quad (2-27)$$

where γ is the chord slenderness ratio. For square and rectangular HSS, $\gamma = B/2t$.

A numerical finite element study by Yu (1997) on square HSS with $\beta = 1.0$, which also utilized the deformation limit of $0.03B$ as a connection ultimate limit state, showed that Equation 2-27 represented a lower bound on the ultimate moment capacity for various chord lengths beyond the connection. The available connection strength for this limit state is obtained using $\phi = 0.95$ (for LRFD) and $\Omega = 1.58$ (for ASD), per AISC *Specification* Table K4.2.

This limit state can be negated by the use of diaphragms or stiffeners, located close to the connection to maintain the chord rectangular shape.

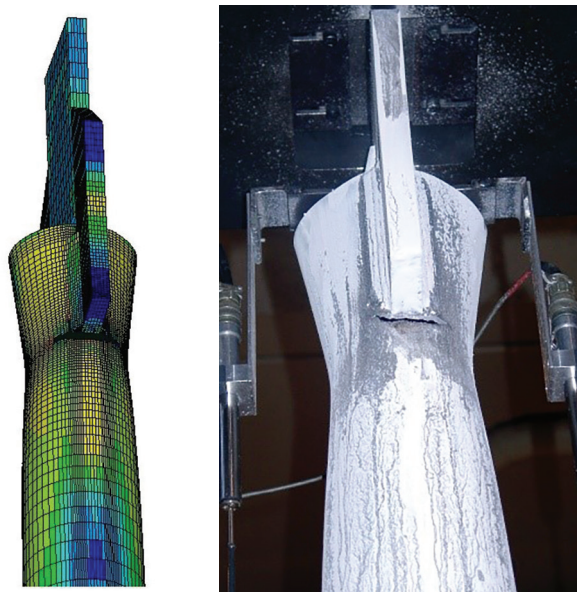
2.13 DESIGN OUTSIDE THE LIMITS OF APPLICABILITY

Many of the limit states presented have design recommendations in the AISC *Specification* that are empirical or partly empirical and are thus bounded by limits of applicability. A User Note in AISC *Specification* Section K1 points out that the limits of applicability reflect limitations on tests conducted to date (which can be experimental or numerical), measures to eliminate undesirable limit states, and other considerations discussed in the Commentary. In Chapters 8,

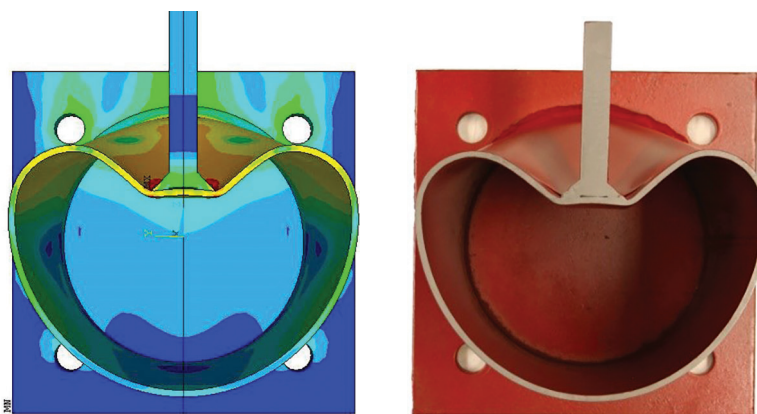
9, and 10, pertaining to welded connections to HSS main members, the limits of applicability relevant to each limit state or failure mode, for each connection type, are dissected. The design of connections beyond the limits of applicability is still allowed, but the designer then needs to investigate all of the foregoing limits of applicability using rational methods of analysis. If one is designing marginally beyond a connection limit, and the trend in connection nominal strength is increasing towards that limit (e.g., with yield stress), it would be prudent to cap the connection strength at the value corresponding to the limit on that parameter. Unfortunately, the opposite is usually the case (i.e., connection strength decreases toward the limit), especially with geometric limits (e.g., with wall slenderness).

The Commentary to AISC *Specification* Chapter K cites inelastic finite element (FE) analysis as a tool for design beyond the limits of applicability for experienced engineers, and this option is permitted by AISC *Specification* Appendix 1. Both welded and bolted connections may be designed by FE methods, with elastic analysis being sufficient for fatigue design but inelastic analysis being generally necessary to capture the ultimate limit state. Any limitations of the chosen software should always be checked to ensure that the software has the capability of properly investigating and capturing the limit state being investigated. This may involve confirmation of nonlinear analysis, failure criteria, sufficient number of nodes and elements permitted, element types in the library, and so forth.

The ability of modern FE models to replicate the behavior of laboratory connection tests is illustrated in Figure 2-19. Besides qualitative pictorial agreement of the failure mode and deformations, the numerically simulated and experimental branch load versus displacement relationships need to be quantitatively matched. Features to be considered in FE modeling are now even described in some steel design codes, such as EN 1993-1-14 (CEN, 2020). The eventual FE model is, ideally, validated against one or more laboratory experiments on identical connections, but this is commonly not possible in design situations. In this case, the FE modeling technique and chosen mesh and material parameters should be verified against some FE analyses from the research literature or from collected benchmark cases (which in turn have been validated), such as by Wald et al. (2017). Further details about FE-based design of HSS connections are available elsewhere (Packer, 2020). This information covers element types, meshing, material properties, boundary conditions, loading, and ultimate load criteria, with the latter based on a consideration of both peak load and the load corresponding to a limiting strain or deformation (Kozich et al., 2022).



(a) A slotted HSS end connection loaded in tension, failing by circumferential fracture



(b) A longitudinal plate connection under branch axial compression, failing by chord plastification (section view)

Fig. 2-19. Examples of deformed shapes of round HSS connections, comparing FE model output (left) with the laboratory experiment (right).

Chapter 3

Welding

The design of welds is covered in AISC *Specification* Section J2 with additional requirements for welds of plates and branches to rectangular and circular HSS given in Section K5. AISC *Manual* Part 8 adds some further “Welding Considerations for HSS.” The AISC *Specification* and *Manual* are supported by the comprehensive national standard AWS D1.1/D1.1M, which includes clause 10 dealing specifically with HSS construction. The Canadian counterpart to AWS D1.1/D1.1M is the (very similar) welding standard CSA W59 (CSA, 2018b). AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (Miller, 2017), provides excellent general advice for engineers on welding, including some particular remarks on welding HSS. This chapter deals with issues that are especially pertinent, and sometimes unique, to the welding of statically loaded HSS connections.

With HSS branch-to-HSS welded connections, or plate branch-to-HSS welded connections, there are two design philosophies that can be used for weld design:

1. The weld can be proportioned so that it develops the yield strength of the connected branch wall at all locations around the branch. This will represent an upper limit on the weld size and a conservative design procedure. This approach is particularly appropriate if plastic stress redistribution is required in the connection. The same effective size for the weld should be maintained all around the attached branch, except possibly for the “hidden weld” in HSS-to-HSS partially overlapped K- or N-connections (discussed further in Section 3.5).
2. The weld can be proportioned so that it resists the applied forces in the branch. This approach is particularly appropriate if the branch forces are low relative to the branch member strength, such as when branches are sized for aesthetic reasons (e.g., the branches of a simply supported truss may be the same throughout the truss, yet, at the center of the truss, the web member forces will be very low under uniformly distributed load). If this design philosophy is adopted, weld effective lengths must be taken into account (see Section 3.5), and the same effective size for the weld should still be maintained all around the attached branch. The entire branch perimeter should be welded, including the “hidden toe” in an HSS-to-HSS partially overlapped K- or N-connection (see also Section 3.3).

3.1 TYPES OF HSS WELDS

The five common types of welds used in HSS connections, are:

1. Fillet welds
 2. PJP groove welds
 3. PJP groove welds reinforced with fillet welds
 4. Flare-bevel and flare-V-groove welds (special cases of PJP groove welds)
 5. CJP groove welds
- The cost of fabricated structural steel is primarily driven by labor costs, not material costs. The cost of welded details is best evaluated by comparing the number of passes required to deposit various welds and consideration of other labor such as the cost associated with preparation of material at groove welds. Weld volume alone is not a good indication of welding costs (see AISC Design Guide 21 for further information related to economy in welding).
- Engineers should avoid overspecifying welds; only the effective size required to transfer the design loads is critical to the structural performance of the weld. In general, the effective size is all that should be specified in the contract documents. AWS D1.1/D1.1M requires that contract documents specify the effective throat for PJP groove welds and the weld size for fillet welds joining perpendicular parts. To obtain the most economical structure, engineers may also want to give the fabricator the option of using fillet welds, PJP groove welds, or some combination of the two. This can be accomplished by providing the fabricator with actual loads if connection design is delegated, or by providing an effective throat in addition to specifying the fillet weld size for fillet welds joining perpendicular parts.
- Whenever possible, welds should be designed to be “fit for purpose” (i.e., able to resist the actual forces present in a branch member). Overwelding should be discouraged as it is an expensive practice. For example, the fabrication cost of a $\frac{3}{8}$ in. fillet weld could be three to four times that of a $\frac{5}{16}$ in. fillet weld, even though the weld size (and therefore the strength) has increased only 20%. Specifying welds that are larger than necessary can also increase the likelihood of unacceptable heat-related distortion of the parts and weld cracking.

3.1.1 Fillet Welds

The strength of fillet welds is given in AISC *Specification* Section J2.4 and Table J2.5. It should be noted that the nominal strength is determined from the weld effective throat area, with no consideration of the fusion face area. Filler metal with a strength level less than or equal to “matching” should be used (or even one strength level greater than “matching” by up to 10 ksi based on minimum specified tensile

strength is permissible provided the assumed strength does not exceed matching). Weld strength is determined by the limit state of shear rupture along the weld effective throat, regardless of the direction of loading. For HSS with a minimum specified yield strength of 50 ksi, E70 filler metal with a specified tensile strength of 70 ksi is considered matching (AWS D1.1/D1.1M, Table 5.4).

Directional Strength Increase Factor

AISC *Specification* Section J2.4(a) permits, in general, the available strength of fillet welds to be increased by a directional strength increase factor of $(1.0 + 0.50 \sin^{1.5} \theta)$, termed k_{ds} , when the direction of loading is at an angle of θ° to the axis of the weld, providing strain compatibility of the various weld elements is considered. However, it has been shown that the use of this fillet weld directional strength increase factor does not provide adequate structural reliability (or safety) when used in conjunction with the rules for weld effective lengths in AISC *Specification* Section K5 for certain rectangular HSS-to-HSS connections (McFadden and Packer, 2014; Tousignant and Packer, 2015). Consequently, AISC *Specification* Section J2.4(a) does not allow the use of the fillet weld directional strength increase factor when designing “fit-for-purpose” fillet welds to the ends of rectangular HSS members that are loaded in tension in truss-type connections (i.e., $k_{ds} = 1.0$). On the other hand, the AISC *Specification* does allow the directional strength increase factor to be applied to fillet welds at the ends of round HSS under any loading, with k_{ds} calculated according to Equation J2-5. This is a consequence of the research described in the following.

Laboratory testing and finite element analysis studies have been performed on fillet-welded joints to the ends of rectangular and round HSS members, where the HSS end is connected to a rigid plate and the HSS is subjected to axial tension (Packer et al., 2016; Tousignant and Packer, 2016, 2017a). In these weld-critical tests, where the failure mode is weld fracture (Figure 3-1), the entire weld length is effective due to the rigid base material. This research has shown that single-sided welds to a tension-loaded HSS wall element are partially unrestrained and are prone to local bending about the axis of the weld, as shown in Figure 3-2, leading to opening of the weld root. AISC *Specification* Section J2 Commentary states that the use of single-sided fillet welds in joints subject to rotation around the weld toe is discouraged, and AISC *Manual* Part 8 also elaborates on this effect. The restraint provided to the fillet weld depends on the connected element thickness and shape (linear versus curved), as well as the weld size and amount of penetration. The single-sided weld effect, however, is much more severe for square and rectangular HSS than for round HSS when viewed separately. It has been found that tension-loaded rectangular HSS-to-rigid plate welded joints do not achieve the expected target safety (reliability) index of 4.0 (AISC *Specification* Commentary Section B3.1) at failure, if the fillet weld directional strength enhancement factor is applied, thus supporting the prohibition in AISC *Specification* Section J2.4. However, tension-loaded round HSS-to-rigid plate welded joints did approximately achieve the required target safety (reliability) index, thus supporting the use of the directional strength increase factor for fillet welds to the ends of round HSS (Tousignant and Packer, 2019b).

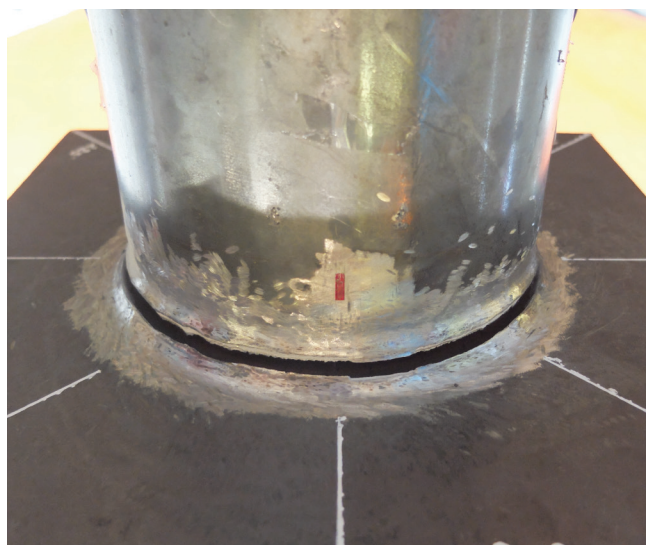


Fig. 3-1. Fillet weld fracture tests on square and round HSS-to-rigid end-plate connections.

Use of the directional strength increase factor is permitted for double-sided fillet welds to longitudinal or transverse plate branches, attached to either rectangular or round HSS, regardless of branch loading. The fillet weld directional strength increase factor is universally permitted for single-sided welds in joints under compression, or combined shear and compression, because such loading tends to close the weld root. Thus, for example, fillet welds to the ends of the branches of a rectangular HSS cross-connection, with the branches loaded in axial compression, may take advantage of the directional strength increase factor. If a rectangular HSS branch is in bending, a portion of the weld is in compression and a portion of the weld is in tension. On the tension side the fillet weld root will tend to open, and the prohibition of the directional strength increase applies. On the compression side the weld root will tend to close, and the prohibition of the directional strength increase should not apply. At present, available research evidence (McFadden and Packer, 2014) indicates nonuse of the increase factor for rectangular HSS branch bending, in conjunction with the weld effective properties in AISC *Specification* Section K5.

Weld Size to Develop the Branch Yield Strength

Use of design philosophy 1, whereby a fillet weld is proportioned to develop the yield strength of the connected branch wall at all locations around the branch, represents a simple design procedure that is often employed. The required fillet weld size for achieving this depends on the HSS and filler metal material properties, the angle of the weld joint to the direction of loading, the specific steel code being satisfied, and the target structural reliability (or safety) level. AISC *Specification* Section K5 Commentary states that the branch

yield strength is presumed to be achieved, for HSS with $F_y \leq 50$ ksi and matching (70 ksi) filler metal, for a fillet weld with an effective throat equal to 1.1 times the branch member (design wall) thickness. This is extracted from international guidance based on Eurocode 3 (CEN, 2005; Wardenier et al., 2008a; Packer et al., 2009), but has also been confirmed to be appropriate (Tousignant and Packer, 2017a) for branches at 90° in round HSS connections using AISC calibration criteria. For rectangular HSS connections, however, because of the more severe eccentric loading on single-sided fillet welds in such joints, Tousignant and Packer (2017a) have shown that, in theory, a fillet weld effective throat size of 1.3 times the branch member (design wall) thickness is required, to achieve the branch yield strength with the AISC target safety index of 4.0 for welded joints (AISC *Specification* Section B3.1 Commentary). Example 3.5 illustrates the application of this branch capacity design method by applying the weld and member available strength equations in the AISC *Specification*.

Practical Fabrication

Fillet welds will often be the most economical choice for HSS connections, but the weld size (and hence the number of passes) will be affected by the availability of the directional strength increase factor. To facilitate fillet welding in square or rectangular HSS-to-HSS connections, the branch should preferably sit on the “flat” of the main through (chord) member, as shown in Figure 3-3, for a “stepped” configuration rather than a “matched” configuration (branch and chord of equal width). Given that the outside corner radius averages approximately $2t$ for North American cold-formed rectangular HSS, where t is the chord wall design

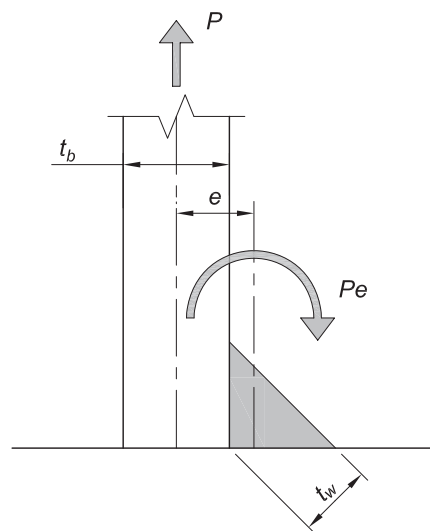


Fig. 3-2. Eccentric loading on a single-sided fillet weld resulting in local bending.

Table 3-1. Effective Weld Size Factors for Skewed Joints	
Dihedral Angle (°)	Factor
60	1.41
70	1.23
80	1.10
90	1.00
100	0.923
110	0.863
120	0.816
130	0.780

thickness, this implies that the branch and its associated fillet weld legs have a preferred width $\leq (B - 4t)$. In cases where matched-width ($\beta = 1.0$) HSS connections are required, and groove welds are necessary along the matched longitudinal edges, note that fillet welds can still be used for the transverse welds across the chord member flats.

Skewed Joints

In some situations, the attached element in the joint may be skewed, or nonperpendicular, to the continuous element as shown in Figure 3-4. As the dihedral angle decreases from 90° , the theoretical throat increases. On the other side with the larger angle, the theoretical throat decreases. Table 3-1 provides the equivalent weld size factors to account for the skew angle for one fillet weld. The actual weld (leg) size, w (Figure 3-4), is multiplied by the factor to obtain the equivalent weld size, w_{eq} , with the same throat and a 90° dihedral angle. If the attached element is cut square, or does not fit tight to the chord wall on the side with the larger angle (as

shown in Figure 3-4), the fillet weld size must be increased to compensate for the root opening, R , which cannot exceed $\frac{3}{16}$ in. according to AWS D1.1/D1.1M, clause 7.21.1. Example 3.1 shows the calculation of a skewed fillet weld.

3.1.2 PJP Groove Welds

PJP groove welds are preferable to CJP groove welds when it is possible to use them. In HSS construction, PJP groove welds occur in butt joints (e.g., welded column splices, with predominantly compression loading), in corner joints (e.g., built-up box columns), and in T-joints (e.g., as an alternative to fillet welds). The effective throat or size of a PJP groove weld is given in AISC *Specification* Table J2.1 (subject to a minimum in Table J2.3, as a function of the thickness of the thinner part joined) and is illustrated in AWS D1.1/D1.1M, Figure 4.2. The effective throat or size depends on the welding process, welding position, and the type of groove or bevel angle; however, the effective throat size is equal to the groove depth for all J-grooves, U-grooves, and 60°

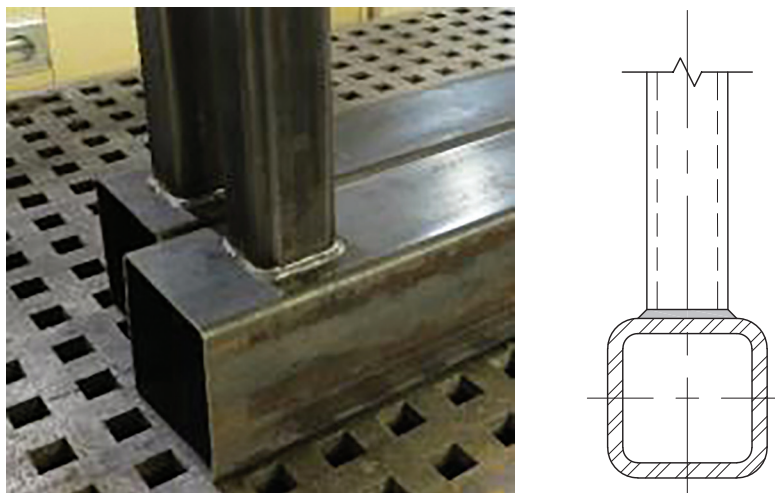


Fig. 3-3. Stepped rectangular HSS connections.

bevels (AISC *Specification* Table J2.1). The effective throat and groove depth may differ for 45° bevels, as illustrated in AISC Design Guide 21, Figure 3-9, and in Section 3.1.3, Figure 3-5 herein. It is recommended that design drawings just show either the required strength or the required effective throat or size, E or S . For all PJP groove welds, matching or under-matching filler metal may be used (AISC *Specification* Table J2.5).

For calculation of the strength there are, in general, two limit states to consider for PJP groove welded joints: failure along the weld effective throat and failure at the fusion face by either yielding or rupture of the base metal. The type of loading is an important factor in determining the available strength of a PJP groove weld joint (AISC *Specification* Table J2.5). The fusion face check is required because the base metal fusion size is often the same as the weld metal throat size (unlike with fillet welds).

3.1.3 PJP Groove Welds Reinforced with Fillet Welds

A PJP groove weld reinforced with a fillet weld may be an attractive option in lieu of a large, multi-pass fillet weld, although the cost of edge preparation must be considered. The effective throat of a joint with a PJP bevel groove weld with a reinforcing fillet weld is described in AWS D1.1/D1.1M, clause 4.4.2.7, and illustrated in Figures 4.3 and 4.4. In such cases, clause 4.4.2.7 points out that the total capacity of a PJP groove weld reinforced with a fillet weld is not simply the mathematical sum of the two individual weld capacities. The correct throat dimension of a PJP groove weld reinforced with a fillet weld combination is the least dimension from the weld root to the weld face of the diagrammatic fillet weld, with no allowance for any weld face convex reinforcement, and considering the dimensional loss at the weld root associated with 45° bevels (for such bevels only), as

shown in Figure 3-5. Then, using this effective throat (E or S), the nominal strength of the entire weld can be determined from AISC *Specification* Table J2.5 as a PJP groove weld.

3.1.4 Flare-Bevel and Flare-V Groove Welds

These welds are actually special cases of PJP groove welds wherein the convex surface of the part creates the joint preparation, which occurs when welding at the rounded corners of rectangular and square HSS. They are used to attach flat elements that are wider than the HSS (flare-bevel), they arise in matched-width ($\beta = 1.0$) connections, and they are used to attach two HSS together at mutual corners (flare-V) such as in back-to-back, double-chord trusses. Figure 3-6 shows examples of flare-groove welds, where the welds are not filled flush to the edge of the HSS. The gap that can be seen at the weld root, producing a so-called Z-loss dimension, is also very typical.

AISC *Specification* Table J2.2 gives the effective throat sizes for flare groove welds, depending on the welding process, and presumes that the welds are filled flush with the outside face of the HSS. (Some acceptable “flush” conditions are illustrated in AISC Design Guide 21, Figure 3-13.) These values are based on experimental research by Packer and Frater (2005) and take into account the typical gap size (or Z loss) at the root, as well as typical HSS corner radii (which are often less than a 90° arc, as indicated in Figure 3-6). The weld effective throat sizes for flare-bevel groove welds and flare-V groove welds are expressed in terms of the HSS outside corner radius (in AISC *Specification* Table J2.2), which, since unknown to the weld detailer, can be taken as being equal to $2t$, where t is the design wall thickness.

- For flare-bevel groove welds, this then gives an effective throat size of $1.25t$ for gas metal arc welding (GMAW) and flux-cored arc welding-gas-shielded (FCAW-G)

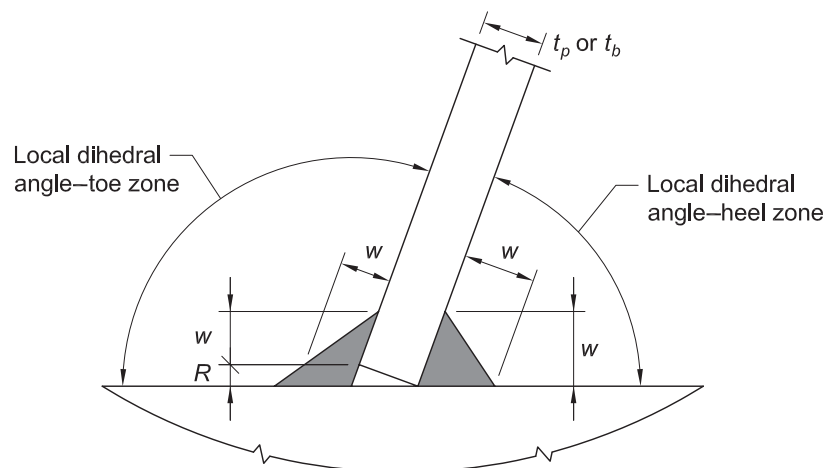


Fig. 3-4. Skewed joint.

processes and an effective throat size of $0.625t$ for shielded metal arc welding (SMAW), flux-cored arc welding–self-shielded (FCAW-S), and submerged arc welding (SAW).

- For flare-V groove welds, this then gives an effective throat size of $1.5t$ for GMAW and FCAW-G, $1.25t$ for SMAW and FCAW-S, and $1.0t$ for SAW.
- In today’s environment, GMAW and FCAW-G are typically shop-welding processes, while SMAW and FCAW-S are typically field-welding processes.

If the weld is not filled flush to the HSS face (which is the situation shown in Figure 3-6), these effective throat sizes are reduced by subtracting the “underfill” dimension, which is the distance from the weld face to the HSS face, D_f , as shown diagrammatically in Figures 3.6(c) and 3.6(d), to obtain the effective throat, E . There is no requirement for flare groove welds to be always filled flush; if the throat of an underfilled joint is sufficient to transfer the applied loads, considerable economy can be achieved by underfilling (Miller, 2017), but the welder and inspector then have to measure the amount of underfill.

AISC *Specification* Table J2.2 does not place an upper limit on the HSS outside corner radius, and thus not on the HSS wall thickness, and is conservative in the resulting effective weld sizes for large corner radii. However, the AISC *Specification* does note that effective throats other than those in Table J2.2 may be used for a given welding procedure specification (WPS) if demonstrated by tests. AISC *Specification* Table J2.2 presumes that high heat input is applied (e.g., GMAW in full spray mode). Because flare-bevel and flare-V groove welds are PJP welds, the available strength of a joint is obtained from AISC *Specification* Table J2.5 for a particular effective throat size.

It is possible to increase the effective weld size of a flare-bevel groove weld between an HSS corner and a flat element, even beyond that of the filled-flush situation. This can be achieved by the addition of a reinforcing fillet, which is

described in AWS D1.1/D1.1M, clause 4.4.2.7, and illustrated in Figure 4.6. This creates a situation similar to that shown in Figures 3-5(c) and 3-5(d).

Matched-width connections between square and rectangular HSS represent another special flare-bevel groove weld situation. If the corner radius of the through (chord) member is large (due to a large wall thickness or manufacturing tolerances), there may be a considerable root gap, as illustrated in Figure 3-7. The root opening of prequalified welds is generally limited to $\frac{1}{16}$ in. as detailed, or $\frac{1}{8}$ in. as fit up. Given the uncertainty about the dimensions of the corner radius, some judgement and experience must be exercised regarding fabrication.

If the root gap is excessive it needs to be closed, and the AISC *Manual* proposes use of one of the solutions shown in Figure 3-8:

1. Profile shaping of the branch, such as by machining, so that it fits the chord member contour.
2. Building up the weld progressively by “buttering” after laying down a careful root pass.
3. Using backing, which can be tacked in place to the branch, either parallel to the HSS chord face, or more commonly parallel to the inside wall of the branch (as shown in Figure 3-8).

If the corner radius of the through (chord) member is not too large, it may be possible to straight cut the branch and weld directly in the corner region without further weld preparation, as shown in Figure 3-9. No preparation of the branch wall, which is the situation in Figure 3-9(a), then needs to be performed, which is the ideal situation.

There are some instances in which matched-width HSS T-connections must be chosen, and flare-bevel groove PJP corner welds cannot be avoided. If 2D or 3D moment-resisting frames are chosen, such as in Figure 3-10, a connection with $\beta = 1.0$ is one of the requirements for a fully restrained (FR) moment connection. For Vierendeel frames

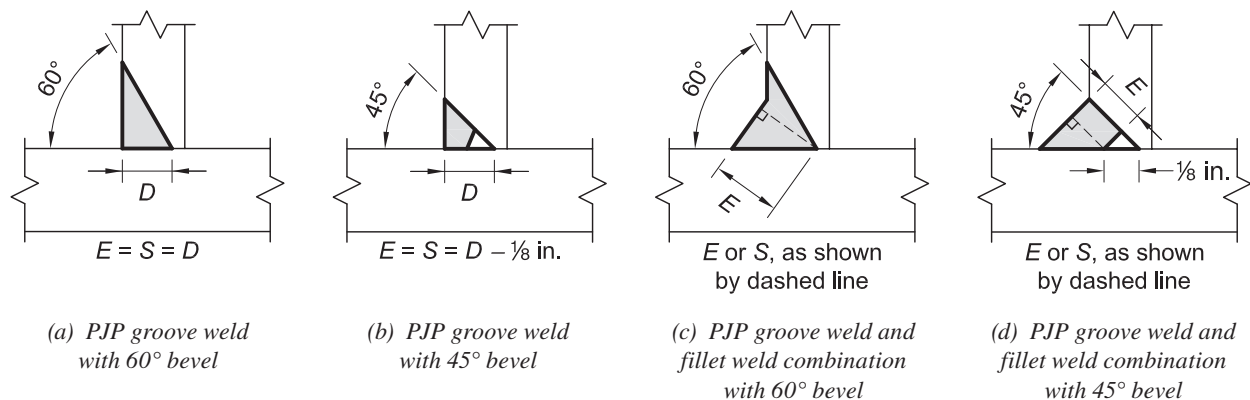


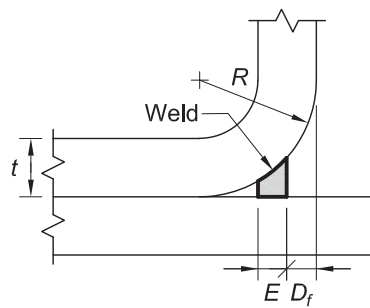
Fig. 3-5. Effective throat or size, E or S , for SMAW process. (See AISC *Specification* Table J2.1 for other welding processes.)



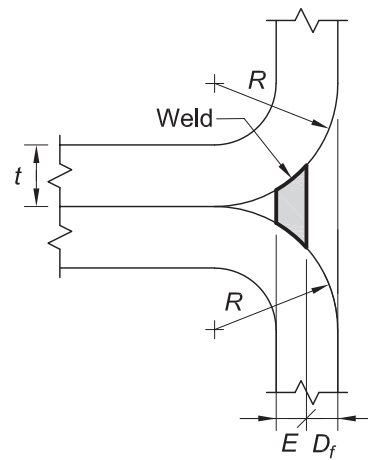
(a) Flare-bevel weld, cross section



(b) Flare-V weld, cross section



(c) Effective throat, E , for flare-bevel weld



(d) Effective throat, E , for flare-V weld

Fig. 3-6. Examples of flare-groove welds to square HSS not filled flush to the HSS outside face.



Fig. 3-7. Matched-width T-connection with large fit-up gap.

using square or rectangular HSS members, it has been found that unstiffened T-connections between the same steel grade with $\beta = 1.0$, $B/t \leq 16$ and $t/t_b \geq 2.0$ can be treated as rigidly connected and can be assumed to achieve the moment capacity of a compact branch (Korol et al., 1977; Wardenier, 1982; Packer and Henderson, 1997). This is discussed in Section 10.1.1. In such FR connections satisfying these geometric requirements, the effective weld size should be specified as t_b , as shown in Figure 3-9(a).

Caution is recommended when welding in zones of high cold working (such as HSS corners) in some HSS connection guidance documents (e.g., ISO 14346, Table A.1). Minimum preheat and interpass temperature requirements for welding are covered by AWS D1.1/D1.1M, clause 5.7 and Table 5.8. For ASTM grades of HSS and most welding processes, the minimum temperature is 32°F to 50°F (for thicknesses up to $\frac{3}{4}$ in.) Thus, the AWS requirement for preheating is mainly applicable to site welding conditions. Note that AWS D1.1/D1.1M does not address welding in regions of high cold forming and, thus, may be unconservative in that case.

3.1.5 CJP Groove Welds

Although very large tubular or box-member connections may be an exception, welding can only be done from one side with HSS-to-HSS connections. Therefore, any groove-welded joint detail that requires preparation and welding on both sides—or backgouging—is not possible. The specification of a CJP groove weld should be an exceptional case, considering their difficulty and expense; especially because HSS connections will rarely need to develop the full yield strength of the connected branch member anyway. When CJP groove welds are required, steel backing should be used—where possible—with a detailed root dimension to allow placement of sound filler metal in the full depth of the joint. In many cases, specially shaped inserts may be required to accommodate the HSS geometry. AWS D1.1/D1.1M, clause 7.9.1.2, generally requires the use of continuous backing for the length of the joint. However, discontinuous backing is allowed at the ends of a pair of spliced HSS members, providing that the backing is made from one or

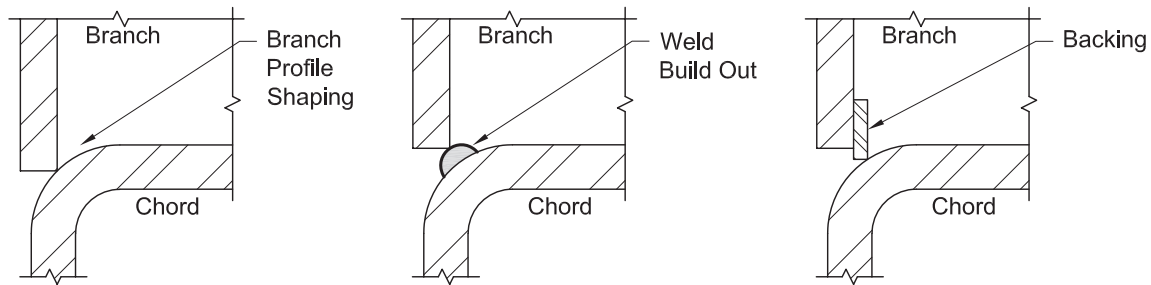
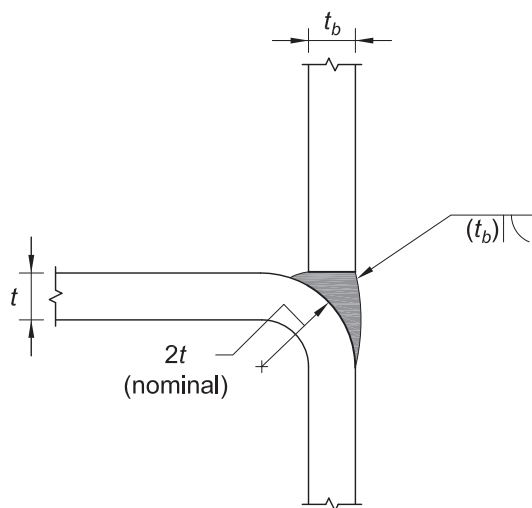


Fig. 3-8. Solutions for closing a root gap at an equal-width HSS weld joint.



(a) As specified



(b) As welded, with penetration to the root

Fig. 3-9. Flare-bevel weld for an equal-width HSS weld joint.

two pieces, the connection is statically loaded, and the backing is also continuous around the inside corners of a rectangular HSS. (Note that backing consisting of four pieces, each tacked to the inside of each flat of a rectangular HSS member, is not allowed.) This special AWS D1.1/D1.1M exemption for discontinuous backing is not extended to the ends of HSS members in T-, Y-, and K-connections because only one HSS end is involved in such connections. Another AWS D1.1/D1.1M exemption to the continuous backing rule is for field splices in statically loaded box-section columns, where four pieces of steel flush with the inside walls may be used, with interruptions at the corners. In statically loaded structures, backing may be left in place. A good discussion of weld backing types and requirements is given in AISC Design Guide 21, Section 4.2.3.

In cyclically loaded structures, or other structures where backing cannot be left in place, an alternative approach is required because it is not possible to remove the backing that is concealed inside the HSS. In such cases, CJP groove welds must be made from one side without the use of backing, commencing with a backup weld at the root placed from the outside. Careful preparation of the joint is required to ensure the proper gap at the root. AWS D1.1/D1.1M does include prequalified CJP groove welds made from one side without backing for T-, Y-, and K-connections. Because of the limited access and skill required to fill the open root, a demanding 6GR welder certification will be required if welding is to be done in any position (AWS, 2020). Coordination with the fabricator is required if single-sided CJP welds are to be specified. The fit-up, skill, and qualification requirements for CJP-welded T-, Y-, and K-connections make them good candidates for cast steel connectors.

Per AISC *Specification* Table J2.5, CJP groove-welded joints simply develop the available strength of the base metal when matching filler metal is used for any type of static loading. In some loading cases, less-than-matching filler metal is

also allowed. Prequalified joint details for CJP groove welds in HSS-to-HSS connections are given in AWS D1.1/D1.1M, clause 10.10.2 and Figures 10.7 through 10.11.

3.2 WELD INSPECTION

AISC *Manual* Part 8 lists and discusses the five most commonly used types of NDT methods described in AWS D1.1/D1.1M. The discussion is in general terms and applies to all types of welding. The following is a summary of NDT methods applied to HSS:

1. Visual testing (VT) must be performed on all welds (AWS, 2020) to ensure that they meet structural quality in HSS fabrication. All joints are examined prior to the commencement of welding to check fit-up, preparation of bevels, gaps, alignment, and other variables. After the joint is welded, it is then visually inspected in accordance with AWS D1.1/D1.1M. If a discontinuity is suspected, either the weld is repaired, or other inspection methods are used to validate the integrity of the weld. In most cases, timely visual inspection by an experienced inspector is sufficient and offers the most practical and effective inspection alternative to other more costly methods.
2. Penetrant testing (PT), using a sprayed-on red penetrating dye followed by a white developer, is useful for the detection of surface indications including tight cracks that are open to the surface. PT tends to be messy and slow, but it can be helpful when determining the extent of a defect found by visual examination. This is especially true when the defect is being removed by gouging or grinding for the repair of a weld to ensure that the defect is completely removed.
3. Magnetic-particle testing (MT), as shown in Figure 3-11, is useful for the detection of surface indications and sub-surface indication to a depth of approximately 0.1 in. MT



Fig. 3-10. 3D moment-resisting frame using equal-width HSS members.

can be used when the absence of cracking in areas of high restraint must be confirmed.

4. Ultrasonic testing (UT) has limited applicability in HSS fabrication. Relatively thin sections and variations in joint geometry can lead to difficulties in interpreting the signals, although technicians with specific experience on weldments similar to those to be examined may be able, in some instances, to decipher UT readings. Ultrasonic testing is not suitable for use with fillet welds and smaller PJP groove welds. CJP groove welds with and without backing also give readings that are subject to differing interpretations.
5. Radiographic testing (RT) has very limited applicability in HSS fabrication because of the irregular shape of common joints and the resulting variations in thickness of material as projected onto film. RT can be used successfully for butt splices but can only provide limited information about the condition of fusion at backing (bars) near the root corners. The general inability to place either the radiation source or the film inside the HSS means that exposures must usually be taken through both the front and back faces of the section with the film attached outside the back face. Several shots progressing around the member are needed to examine the complete joint.

AISC *Specification* Section N5.4 provides specific requirements for the inspection of welding, and AISC Design Guide 21, Chapter 10, provides more discussion of this topic.

3.3 CUTTING AND WELDING OVERLAPPING HOLLOW SECTIONS

Hollow section connections are highly stressed zones, and it is advisable to make welded butt joints (splices) between adjoining HSS, such as between co-axial chord members or column members, beyond the HSS-to-HSS connection region. The main chord or column member should be continuous through the connection, and branch members then

welded to this through member. Figure 3-12(a) shows a poor fabrication detail, producing a potentially weak connection, in which the intersecting members of a round HSS cross-connection are all interrupted and welded to each other—two opposite branches should have been saddled to sit on a continuous through (chord) member. Figure 3-12(b) also shows an incorrect method for cutting the web members at an overlapped K-connection—one branch member should be welded directly to the through (chord) member, then the second branch should be profiled to sit on these members. The through branch should be the wider of the two branches; in situations where the two branches have the same width (or diameter), the thicker branch should be the through branch. Where the two branches of a K-connection are identical, which is a common case, the more heavily loaded branch should be the through branch and welded directly to the chord; if both branch loads are similar, then it is preferable to weld the tension branch directly to the chord. Connection resistance formulas in Chapter 9 presume that members are joined in this manner.

Figure 3-13(a) illustrates a correct fabrication technique for a partially overlapped K-connection. The diagonal branch is profiled for direct welding to the chord member only; the vertical (overlapping) branch is profiled for welding to both the chord member and the diagonal (overlapped) branch member. Overlapped K-connections can be detailed with the branches partially overlapping, as shown in Figure 3-13(a), or fully overlapping, as shown in Figure 3-13(b). The latter produces an even stronger connection and is also simpler to fabricate because one branch sits on the other branch. Moving the overlapping branch slightly up the overlapped branch (producing >100% overlap) to facilitate welding is also permitted. However, gapped connections are still generally preferred over overlapped connections because they are easier to fabricate.

Round HSS-to-HSS joints are complex, and the bevel preparation varies around the perimeter of the joining member. A good discussion of various types of backing and joint



Fig. 3-11. Magnetic-particle testing of an HSS connection.

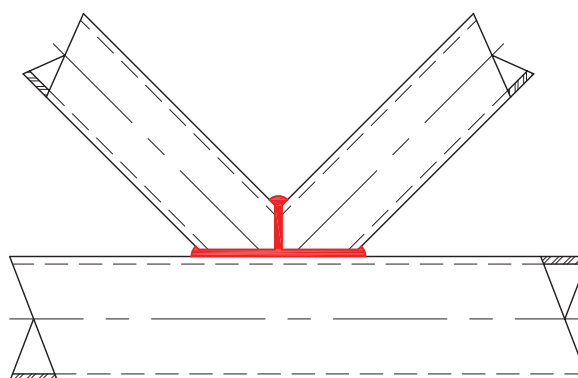
details is given by Post (1990). Prequalified tubular T-, Y-, and K-connection details are given in AWS D1.1/D1.1M, clause 10. Computer numerical control (CNC) tube-profiling machines [e.g., Figure 3-14(a)], that are programmed to produce clean, accurate cuts with the correct bevels on the edges [Figure 3-14(b)], are available. Equipment varies with the number of axes of movement, and cutting can be by oxy-acetylene, plasma, or laser, depending on the thickness involved. In addition to the special end preparations required in round-to-round HSS welded connections, consideration also needs to be given to the member fabrication sequence. In a truss assembly, it may not be possible to insert a particular branch member in place if most of the truss is already welded together. For example, the vertical branch in Figure 3-13(a) cannot be moved laterally into position if both chords are already fixed in place (whereas this is not an issue with the welding of square and rectangular HSS trusses).

A number of hollow section design guides stipulate that the acute angle between a branch member and the chord should not be less than 30° . This is not mandatory but, as pointed out in a User Note in AISC *Specification* Section K1, angles between the chord and branch(es) of less than 30° can make welding and inspection difficult and should be avoided. At the very least, coordination with the fabricator is required. Weldability can be dependent on the process used (e.g., the nozzle size associated with gas-shielded processes) and the member shapes (e.g., with round-to-round HSS the tightest dihedral angle only occurs at the heel crown position, and thereafter the access “opens up”).

With partially overlapped K-connections, the toe of the overlapped branch member is hidden in the final welded joint. The weld to this toe can obstruct the fit-up of the overlapping branch member, hence fabricators prefer to leave the so-called hidden toe unwelded or just tacked in place. As



(a) Cross-connection

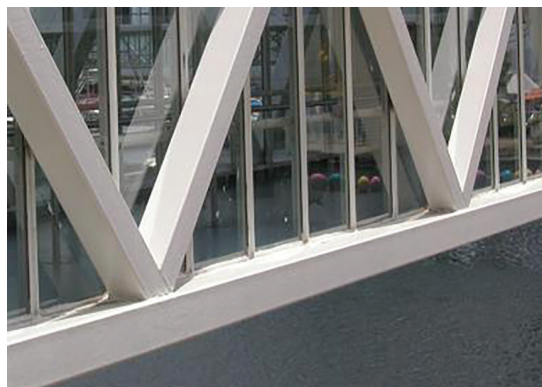


(b) K-connection

Fig. 3-12. Unacceptable fabrication details for intersecting hollow sections.



(a) Partially overlapped round HSS K-connection



(b) Fully (100%) overlapped square HSS K-connections

Fig. 3-13. Acceptable fabrication details for intersecting hollow sections.

pointed out in AISC *Specification* Sections K3 and K5 Commentary, it has been found experimentally that it is permissible to not weld the hidden toe providing:

1. The components of the two branch member forces normal to the chord substantially balance each other (i.e., are within 20%).
2. The welds to the remainder of the overlapped branch are designed to develop the yield capacity of the connected branch walls (philosophy 1 in the introduction to this chapter).

If both of these conditions are not met, the *Specification* requires the hidden toe to be welded. Then, the weld around the overlapped branch should have the same effective throat size all around the perimeter. Thus, welding of the hidden toe is automatically required if welds to the overlapped branch are proportioned to just resist the force(s) in that member. Recent research has shown that more-liberal conditions for leaving the hidden toe unwelded are possible for some overlapped K-connections (Bu and Packer, 2023).

Sometimes, in round-to-round HSS T-, Y-, and gapped K-connections, it is desired to machine the ends of branches so that they saddle onto the chord member without any edge beveling. This preparation is ideal for fillet welding around a branch member, but fillet welding can only be performed for small β ratios. To avoid any branch end profiling, and to still employ fillet welding in lightly loaded round-to-round HSS trusses, an option is to flatten the ends of the branches as shown in Figure 3-15. End flattening of rectangular HSS is not a common procedure. Advice on end flattening techniques, especially to avoid cracking at the flattened edges where the largest strain occurs, is given by Dutta et al. (1998) and Packer and Henderson (1997). The latter also presents some connection design methods.

Welding of HSS connections often involves transitions between weld types when welding around the perimeter of a branch, especially for high β -ratio connections. Matched-width, round-to-round HSS connections result in the branch being profiled to a point at the saddle position. This point is often clipped to facilitate welding (e.g., in cross-connections) without any detriment to the connection capacity.



(a) Cutting and beveling while rotating the tube



(b) Tube ends prepared with saddle cuts for fabrication

Fig. 3-14. CNC 3D profiling of round HSS (photos courtesy of HGG Profiling Equipment BV).

3.4 FILLET WELDS IN SHEAR CONNECTIONS

When a connecting element is fillet-welded to an HSS, AISC *Specification* Table J2.5 requires that the weld metal be designed for shear and the base metal be considered for shear, with the base metal being governed by AISC *Specification* Section J4. Such a base metal shear check is not for shear along a weld fusion face, but refers to shear through the thickness of the base metal. This is a consideration in shear connections to HSS when a relatively thick element (e.g., a plate as in Example 3.2) is welded to a relatively thin HSS. (This is an example of “welding thick to thin.”) The fillet weld may be designed to carry the applied shear load, but the ability of the HSS wall to resist the shear load is unclear. This is described in AISC *Manual* Part 9 under the topic “Connecting Element Rupture Strength at Welds.” AISC *Specification* Section J4.2 lists the limit states of both shear yielding and shear rupture in connected elements; both of these are considered here. With a particular weld size having been determined, a minimum HSS wall thickness can be calculated such that the weld is more critical than the HSS wall (therefore deeming the HSS acceptable). At the beginning of this chapter, two philosophies were described for designing welds, to achieve either (1) the tensile or compressive yield strength of a branch or (2) to resist a particular force in a branch. In this case, the weld is designed to resist a particular force in a branch (specifically shear), but the issue is now the capability of the main member wall to resist the shear load. If the branch force was not in shear, and there was a significant branch force component normal to the axis of the HSS, then an HSS connection failure mode (see Chapter 8, Table 8-1) would likely govern.

The nominal weld metal strength, per unit length, for a fillet weld between two elements at 90° to each other, in AISC *Specification* Section J2.4, is:

$$\begin{aligned} R_n &= F_{nw} \frac{D}{16\sqrt{2}} \\ &= 0.60F_{EXX} \frac{D}{16\sqrt{2}} \end{aligned} \quad (3-1)$$

where

D = fillet weld size, sixteenths of an inch

F_{EXX} = filler metal classification strength, ksi

The nominal strength of the base metal, per unit length next to the weld, in AISC *Specification* Section J4.2 is:

$$R_n = F_{nBM}t \quad (3-2)$$



Fig. 3-15. Round-to-round HSS connection with end-flattened branches.

where

$F_{nBM} = 0.60F_y$ for shear yielding per AISC *Specification* Equation J4-3, ksi,

$= 0.60F_u$ for shear rupture per AISC *Specification* Equation J4-4, ksi

t = thickness of the base metal (not the branch), in.

If a thin HSS base metal was to govern, in the presence of a large weld, the weld could be downsized to an effective weld size, and this could be checked to ensure that it was still capable of carrying the applied shear load. This effective weld size, D_{eff} , can be obtained by equating the available shear strength of the weld metal to the available shear strength of the base metal. Weld metal shear strength resistance and safety factors are from AISC *Specification* Table J2.5. Base metal resistance and safety factors are from AISC *Specification* Section J4.2.

For shear yielding in the base metal:

LRFD	ASD
$\phi_w = 0.75$	$\Omega_w = 2.00$
$\phi_{BM} = 1.00$	$\Omega_{BM} = 1.50$
$\phi_w F_{nw} \left(\frac{D_{eff}}{16\sqrt{2}} \right) = \phi_{BM} F_{nBM} t$	$\frac{F_{nw}}{\Omega_w} \left(\frac{D_{eff}}{16\sqrt{2}} \right) = \frac{F_{nBM} t}{\Omega_{BM}}$
$0.75(0.60F_{EXX}) \left(\frac{D_{eff}}{16\sqrt{2}} \right) = 1.00(0.60F_y) t$	$\frac{0.60F_{EXX}}{2.00} \left(\frac{D_{eff}}{16\sqrt{2}} \right) = \frac{(0.60F_y) t}{1.50}$
$D_{eff} = 30.2 \left(\frac{F_y}{F_{EXX}} \right) t$ (3-3a)	$D_{eff} = 30.2 \left(\frac{F_y}{F_{EXX}} \right) t$ (3-3b)

For shear rupture in the base metal:

LRFD	ASD
$\phi_w = 0.75$	$\Omega_w = 2.00$
$\phi_{BM} = 0.75$	$\Omega_{BM} = 2.00$
$\phi_w F_{nw} \left(\frac{D_{eff}}{16\sqrt{2}} \right) = \phi_{BM} F_{nBM} t$	$\frac{F_{nw}}{\Omega_w} \left(\frac{D_{eff}}{16\sqrt{2}} \right) = \frac{F_{nBM} t}{\Omega_{BM}}$
$0.75(0.60F_{EXX}) \left(\frac{D_{eff}}{16\sqrt{2}} \right) = 0.75(0.60F_u) t$	$\frac{0.60F_{EXX}}{2.00} \left(\frac{D_{eff}}{16\sqrt{2}} \right) = \frac{(0.60F_u) t}{2.00}$
$D_{eff} = 22.6 \left(\frac{F_u}{F_{EXX}} \right) t$ (3-4a)	$D_{eff} = 22.6 \left(\frac{F_u}{F_{EXX}} \right) t$ (3-4b)

Because the smaller D_{eff} determines the controlling limit state, yielding will control if $30.2(F_y/F_{EXX})t < 22.6(F_u/F_{EXX})t$, which simplifies to $F_y/F_u < 0.75$. Table 3-2 gives the governing limit state for D_{eff} for some popular contemporary HSS materials, and it can be seen that only ASTM A53/A53M tube is controlled by base metal yielding instead of base metal rupture.

By setting D_{eff} equal to a particular weld size, the minimum HSS wall thickness, t_{min} , to develop the full strength of the weld can be determined. The matching filler metal for the various HSS grades has a strength level of 70 ksi. Table 3-3 gives the minimum HSS wall thickness to develop the 70-ksi-strength filler metal for fillet weld sizes typically used with HSS. This concept is based on one shear plane per weld. If there are two welds for one plate (e.g., a single-plate shear connection welded on both sides), each weld is associated with half the thickness of the plate. It should be noted that the thicknesses, t and t_{min} , in Tables 3-2 and 3-3 refer to the HSS wall design thickness.

In addition to the limit states associated with the weld, limit states associated with the HSS connection and the connecting element must also be considered. Example 3.2 shows a calculation for the effective weld size in a shear tab connection.

Table 3-2. Controlling Limit State for D_{eff} for HSS Materials					
ASTM Material	F_y , ksi	F_u , ksi	F_y/F_u	Limit State	D_{eff} , 1/16ths
A53/A53M Grade B	35	60	0.583	Base metal yielding	$30.2 \left(\frac{F_y}{F_{EXX}} \right) t$
A500/A500M Grade C	50	62	0.806	Base metal rupture	$22.6 \left(\frac{F_u}{F_{EXX}} \right) t$
A1085/A1085M	50	65	0.769	Base metal rupture	$22.6 \left(\frac{F_u}{F_{EXX}} \right) t$

Table 3-3. HSS Minimum Thickness, t_{min} , to Develop F_{EXX} Strength, in.			
Weld Size, in.	ASTM A53/A53M Grade B	ASTM A500/A500M Grade C	ASTM A1085/A1085M
3/16	0.199	0.150	0.143
1/4	0.265	0.200	0.191
5/16	0.331	0.250	0.238
3/8	0.397	0.300	0.286
7/16	0.464	0.350	0.334
1/2	0.530	0.400	0.381
9/16	0.596	0.450	0.429

3.5 WELD EFFECTIVE LENGTHS

Philosophy 2 in the introduction to this chapter described the concept of designing welds to be “fit for purpose” (i.e., able to resist the actual forces present in the branch member). To account for the nonuniform loading of the weld perimeter due to differences in the relative flexibilities of the HSS chord loaded normal to its surface, and the branch carrying membrane stresses parallel to its surface, weld effective properties—including weld effective lengths and weld effective section moduli—are used. These properties are determined by discounting segments of the weld that do not contribute to its overall resistance. Reduced effective lengths apply both to the weld and the attached branch element; thus, effective lengths for weld sizing are based on a rational extrapolation of the effective length concept used in HSS connection design. The nominal strength of weld groups, under branch axial loading or moment loading, is given in AISC *Specification* Equations K5-1 to K5-3. These equations in turn utilize weld effective lengths and weld section moduli from AISC *Specification* Tables K5.1 and K5.2. However, weld properties for connections to round HSS are more limited than for connections to rectangular HSS main members. The scope of AISC *Specification* Section K5 applies to fillet welds and PJP groove welds; if CJP groove welds are used, the welds can be considered 100% effective because they develop the available strength of the base metal, providing matching filler metal is used (Section 3.1.5).

An integrated summary of weld effective lengths for HSS connections is also provided by Tousignant and Packer (2020).

3.5.1 Welds of Plates and Branches to Rectangular HSS

For elements transverse to a rectangular or square HSS chord or column (a “through member”), the branch load is highest at its extremities closest to the walls of the HSS, and it decreases to the smallest value at the center of the element. This distribution of transverse branch stress is shown in Figure 2-12(b) and is also described in Chapter 8. The net effect is that the weld may not be fully effective and could “unzip” (in AWS parlance), with cracking starting at the branch extremities (see Figure 2-10). The AISC *Specification* accounts for this by specifying a weld effective length, l_e , to be used in calculating the available weld strength. Even though the full weld length may not be effective, a continuous weld is still required around the whole branch perimeter, and the governing effective throat is applied to the whole weld length.

AISC *Specification* Equation K5-4 is used for welds to transverse plates, and it can be seen that the weld total effective length of Equation K5-4 is equal to twice (for two welds, one on each side of the plate) the effective width of a transverse element given by B_e in Equation K1-1. This underlines the concept that where the philosophy of designing the welds in HSS connections to resist specific branch or plate loads is used, the effective weld lengths to be used can be taken as the effective lengths of the connected elements considering the limit state of “local yielding of the branch due to uneven load distribution.” This effective length of transverse welds and transverse elements (B_e , per Equation K1-1) dates back to the 1960s and research by Rolloos (1969), Wardenier et al. (1981), and Davies and Packer (1982), and was adopted for the effective width of the branch transverse wall by IIW (1989). Example 3.3 shows a calculation for a transverse weld to a square HSS. Attached elements (and welds) longitudinal (or parallel) to the axis of a square or rectangular HSS can be assumed to be fully effective; thus, the full weld length is used for the longitudinal plate in Example 3.2.

AISC *Specification* Equation K5-5 gives the weld total effective length for T-, Y-, and cross-connections under branch axial load, with supporting experimental validation being supplied by Packer and Cassidy (1995). For such connections under branch in-plane bending, the effective elastic section modulus of welds for in-plane bending, S_{ip} , can be derived from the effective moment of inertia of the weld group under in-plane bending, I_{ip} , as follows:

$$S_{ip} = \frac{I_{ip}}{H_b/2 \sin \theta} \quad (3-5)$$

$$\begin{aligned} I_{ip} &= 2 \left(\frac{t_w}{12} \right) \left(\frac{H_b}{\sin \theta} \right)^3 + 4 t_w \left(\frac{B_e}{2} \right) \left(\frac{H_b}{2 \sin \theta} \right)^2 \\ &= \frac{t_w}{6} \left(\frac{H_b}{\sin \theta} \right)^3 + \left(\frac{t_w B_e}{2} \right) \left(\frac{H_b}{\sin \theta} \right)^2 \end{aligned} \quad (3-6)$$

Substituting Equation 3-6 into Equation 3-5 gives:

$$S_{ip} = \frac{t_w}{3} \left(\frac{H_b}{\sin \theta} \right)^2 + t_w B_e \left(\frac{H_b}{\sin \theta} \right) \quad (\text{Spec. Eq. K5-6})$$

Similarly, the effective elastic section modulus of welds for out-of-plane bending, S_{op} , can be derived from the effective moment of inertia of the weld group under out-of-plane bending, I_{op} , as follows:

$$S_{op} = \frac{I_{op}}{B_b/2} \quad (3-7)$$

$$\begin{aligned} I_{op} &= 2 t_w \left(\frac{H_b}{\sin \theta} \right) \left(\frac{B_b}{2} \right)^2 + 2 \left(\frac{t_w}{12} \right) B_b^3 - 2 \left(\frac{t_w}{12} \right) (B_b - B_e)^3 \\ &= \frac{t_w}{2} B_b^2 \frac{H_b}{\sin \theta} + \frac{t_w}{6} B_b^3 - \frac{t_w}{6} (B_b - B_e)^3 \end{aligned} \quad (3-8)$$

Substituting Equation 3-8 into Equation 3-7 gives:

$$S_{op} = t_w \left(\frac{H_b}{\sin \theta} \right) B_b + \frac{t_w}{3} (B_b^2) - \frac{(t_w/3)(B_b - B_e)^3}{B_b} \quad (\text{Spec. Eq. K5-7})$$

where t_w is the smallest effective weld throat around the perimeter of the branch (because the weld should have a uniform effective throat size along its entire length).

The requirement in AISC *Specification* Table K5.1 that $B_e/2$ shall not exceed $B_b/4$, when $\beta > 0.85$ or $\theta > 50^\circ$, is based on research on weld-critical HSS-to-HSS T-connections under in-plane bending by McFadden and Packer (2014), as shown in Figure 3-16. They showed that adequate structural reliability (safety index) was achieved with the proposed effective lengths for T-connections under in-plane branch bending and T-, Y-, and cross-connections under branch axial loading, coupled with the B_e condition, provided fillet weld strength was not enhanced by any directional strength increase (i.e., θ was taken as zero in the

“sinθ factor”). All connections tested had $t_b \leq t$, which is considered good design practice for HSS connections. Although no research on weld-critical HSS T-connections subject to branch out-of-plane bending has been performed, it was presumed that AISC *Specification* Equation K5-7 was equally valid, with the same B_e limit.

For gapped K-connections under branch axial load, experimental research was performed by Frater and Packer (1992b, 1992c) on large-scale Warren trusses with weld-critical connections. It was concluded simplistically that the welds along all four sides of an HSS branch member could be taken as fully effective when the chord-to-branch member angle is 50° or less, but that the weld along the heel should be considered as completely ineffective when the angle is 60° or more. A linear interpolation was recommended when the chord-to-branch member angle is between 50° and 60° . This resulted in AISC *Specification* Equations K5-8 and K5-9, except the lengths of the branch walls were each reduced by $1.2t_b$ to allow for the HSS corner radius. The simplified nature of these effective length formulas for the popular gapped K-connection was preferred to the more complex formulas that would result if branch effective widths based on B_e were adopted. Example 9.5 in this Design Guide demonstrates the effective weld length calculation for a gapped K-connection.

For overlapped K-connections under branch axial load, experimental research was performed by Tousignant and Packer (2015) on a large-scale Warren truss with weld-critical connections. All connections had branch members at 60° to the chord, overlaps varied from 30% to 90%, and the “hidden toe” of the overlapped branch was welded in all cases. As for the gapped K-connection weld research, full truss testing was invoked because it correctly accounts for connection boundary conditions, i.e., member continuity and truss deflection effects. Analysis of the fractures of the welded joints, which consisted of both fillet and PJP groove welds because the branches were of equal size (see Figure 3-17), confirmed that all welds lying in planes parallel to the plane of the truss were fully effective. Those welds transverse to the plane of the truss had effective lengths that were analogous to the effective widths of the transverse branch walls.

Thus, weld total effective length for the overlapping branch member i (which was tension-loaded and weld-critical in the experiments performed), $l_{e,i}$, as given by AISC *Specification* Equations K5-10 to K5-12 in Table K5.1, were justified. These equations are similar to those for local yielding of the overlapping branch due to uneven load distribution (AISC *Specification* Equations K3-10 to K3-12 in Table K3.2). Note that the effective width term, B_e , becomes B_{eoi} (in AISC *Specification* Equations K5-10 and K5-11) in cases where the transverse weld lands on the surface of the chord at the overlapping branch heel, per Equation 3-9 and AISC *Specification* Equation K3-13. Similarly, B_e becomes B_{eov} (in AISC *Specification* Equations K5-10, K5-11, and K5-12) in cases where the transverse weld lands on the surface of the overlapped branch at the overlapping branch toe, per Equation 3-10 and AISC *Specification* Equation K3-14. When the connection overlap is close to 100% (AISC *Specification* Equation K5-12), the transverse weld at the overlapping branch heel almost abuts both the chord and the overlapped branch, which is a stiff region; thus, the full branch width, B_{bi} , can be used for the weld effective width in that situation.



Fig. 3-16. Typical fillet weld fracture for an HSS-to-HSS 90° T-connection under branch in-plane bending.

$$B_{eoi} = \frac{10}{B/t} \left(\frac{F_y t}{F_{ybi} t_{bi}} \right) B_{bi} \leq B_{bi} \quad (3-9)$$

$$B_{eov} = \frac{10}{B_{bj}/t_{bj}} \left(\frac{F_{ybj} t_{bj}}{F_{ybi} t_{bi}} \right) B_{bi} \leq B_{bi} \quad (3-10)$$

The requirements in AISC *Specification* Table K5.1 for $B_{eoi}/2$ and $B_{eov}/2$ to not exceed $B_{bi}/4$, in certain circumstances, replicate limits set by McFadden and Packer (2014), but were again justified by Tousignant and Packer (2015). They showed that adequate structural reliability (safety index) was achieved with the proposed effective lengths for overlapped K-connections, coupled with the B_{eoi} and B_{eov} conditions, provided fillet weld strength was not enhanced by any directional strength increase (i.e., θ was taken as zero in the “sin θ factor”).

The weld total effective length for the overlapped (or through) branch member j (which was compression-loaded and noncritical in the experiments performed), $l_{e,j}$, is given by AISC *Specification* Equation K5-13 in Table K5.1. This mimics the weld total effective length of a Y-connection branch, as given by AISC *Specification* Equation K5-5, but with the subscripts of the symbols related to the overlapped branch member, j . Because the transverse welds attached to this branch land on the chord member, B_{ej} in Equation K5-13 should be computed as shown in Equation 3-11 and AISC *Specification* Equation K5-14.

$$B_{ej} = \frac{10}{B/t} \left(\frac{F_y t}{F_{ybj} t_{bj}} \right) B_{bj} \leq B_{bj} \quad (3-11)$$

AISC *Specification* Equation K5-14 subjectively (but conservatively) discounts the contribution of the two transverse welds towards $l_{e,j}$ for steep branch angles or high width ratios, adopting features of AISC *Specification* Equation K5-9 for gapped K-connections. Example 3.4 demonstrates the weld effective length design method for an overlapped rectangular HSS K-connection.

3.5.2 Welds between Round HSS

AISC *Specification* Table K5.2 provides weld effective lengths for certain types of round HSS connections. This table is the result of an extensive experimental and numerical study that was recently performed on weld-critical round HSS-to-HSS cross-connections with the branches subjected to axial tensile loading (Tousignant and Packer, 2017b, 2018). It was found that the effective length phenomenon was also present in round HSS connections, with the weld effective length ranging from 58% to



Fig. 3-17. Typical fillet weld and PJP groove weld fracture for an HSS-to-HSS overlapped K-connection.

100% of the weld total length in the geometric cases studied, but varying with particular geometric parameters. AWS D1.1/D1.1M, clause 10.5.5, implies that the weld effective length in axially loaded round HSS connections is equal to 1/1.5 or 67% of the total weld length for ultimate limit state design, regardless of the joint geometry. While believed to be conservative, this AWS D1.1/D1.1M “ Q_z rule” had no experimental evidence at the time of its implementation.

Branch welds are most highly loaded at the saddle points of the connection (see Figure 3-18); conversely, branch welds are most lightly loaded at the crown points, with the branch stress even sometimes being in the opposite sense to the branch load [as shown in Figure 3-18(b)]. The weld effective length in round HSS cross-connections decreases as the chord wall slenderness, D/t , increases, the branch-to-chord diameter ratio, β , increases, and the branch-to-chord thickness ratio, τ , increases. The branch inclination angle, θ , has an insignificant effect on the weld effective length. It was also determined that branch welds could be deemed 100% effective if $\beta\gamma \leq 4$ [i.e., $(B_b/B)(D/2t) \leq 4$] (Tousignant and Packer, 2017b, 2018).

Round HSS-to-HSS connections can sometimes be fillet-welded; if feasible, this is likely the most economical solution. To obtain the full fillet weld throat thickness, the local dihedral angle, ψ , of the joint needs to be maintained between 60° and 120° . According to Note 4 in AWS D1.1/D1.1M, Figure 10.5, when $\psi < 60^\circ$, the Z-loss values in Table 10.5 for PJP groove welds apply. To keep $60^\circ \leq \psi \leq 120^\circ$, the complex effect of β and θ on ψ , which changes continuously around the joint, needs to be considered. All the weld-critical cross-connection specimens tested by Tousignant and Packer (2017b) had $0.25 \leq \beta \leq 0.47$, $\theta = 60^\circ$ or 90° , and were fillet-welded. As noted in Section 3.1.1, the directional strength increase factor for fillet welds to the ends of all round HSS members has been justified (Tousignant and Packer, 2019b) and is permitted by the AISC *Specification*. Using this “ $\sin\theta$ factor,” an analysis of the fillet weld fracture data supported the use of the following weld effective length, l_e , for T-, Y-, and cross-connections (Tousignant and Packer, 2019c), which is also adopted in AISC *Specification* Table K5.2:

$$l_e = \frac{4}{\sqrt{2\beta(D/t)}} l_w \leq l_w \quad (\text{Spec. Eq. K5-16})$$

where l_w is the total weld length. This weld effective length, l_e , is illustrated in Figure 3-19 by the sum of the two arcs around the saddle regions. Determination of the total weld length around a branch in a round HSS-to-HSS connection is also a difficult matter. In today’s practice, this is best achieved with the aid of computer-aided design (CAD) software, using 3D models of intersecting cylinders. Alternatively, the total weld length given by AWS D1.1/D1.1M, clause 10.5.5, is conservative and sufficiently accurate for $\beta \leq 0.5$ (Tousignant and Packer, 2017b):

$$l_w = \pi D_b \left(\frac{1 + 1/\sin\theta}{2} \right) \quad (3-12)$$

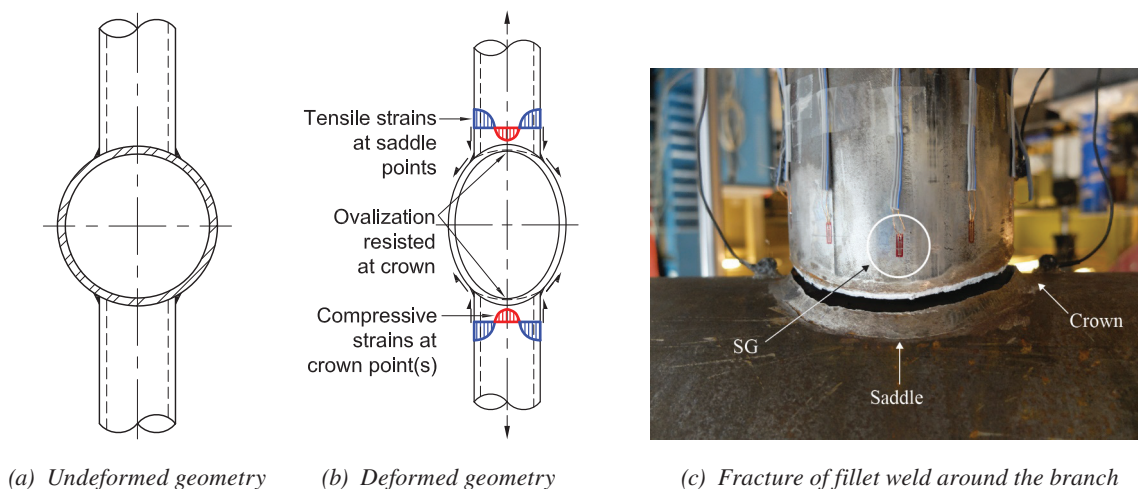


Fig. 3-18. Round HSS-to-HSS cross-connection—stress distribution and weld failure mode.

3.6 WELDED JOINT DESIGN EXAMPLES

Example 3.1—Skewed Fillet Welds

Given:

Using the geometry illustrated in Figure 3-4, with $\alpha = 20^\circ$ and $t_p = \frac{1}{2}$ in., determine the available strength per unit length in shear for welds with size $w = \frac{1}{4}$ in. and 70-ksi filler metal. Also determine the adjustment to the weld size required for the left weld to account for the root opening that occurs due to the skew angle.

Solution:

For the right weld, the dihedral angle is $90^\circ - 20^\circ = 70^\circ$. From Table 3-1, the weld size factor is 1.23. Thus, the equivalent weld size, w_{eq} , is:

$$\begin{aligned} w_{eq} &= (\text{weld size factor})(w) \\ &= 1.23\left(\frac{1}{4} \text{ in.}\right) \\ &= 0.308 \text{ in.} \end{aligned}$$

From AISC *Specification* Section J2.4 and Table J2.5, using a directional strength increase factor $k_{ds} = 1.0$, the nominal strength of the right weld per inch is:

$$\begin{aligned} R_n &= F_{nw} A_{we} k_{ds} && (\text{Spec. Eq. J2-4}) \\ &= 0.60 F_{EXX} \left(\frac{w_{eq}}{\sqrt{2}} \right) k_{ds} \\ &= 0.60 (70 \text{ ksi}) \left(\frac{0.308 \text{ in.}}{\sqrt{2}} \right) (1.0) \\ &= 9.15 \text{ kips/in.} \end{aligned}$$

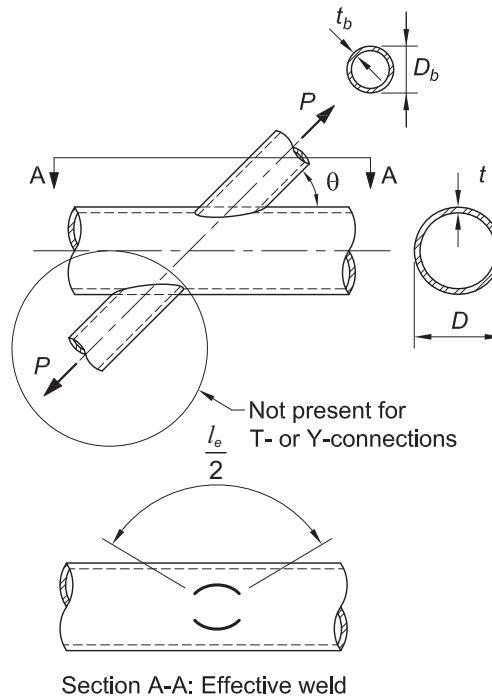


Fig. 3-19. Weld effective length for round-to-round HSS T-, Y-, and cross-connections.

For the left weld, the dihedral angle is $90^\circ + 20^\circ = 110^\circ$. From Table 3-1, the weld size factor is 0.863. Thus, the equivalent weld size, w_{eq} , is:

$$\begin{aligned} w_{eq} &= (\text{weld size factor})(w) \\ &= 0.863\left(\frac{1}{4} \text{ in.}\right) \\ &= 0.216 \text{ in.} \end{aligned}$$

From AISC *Specification* Section J2-4 and Table J2.5, using a directional strength increase factor $k_{ds} = 1.0$, the nominal strength of the left weld per inch is:

$$\begin{aligned} R_n &= F_{nw} A_{we} k_{ds} && (\text{Spec. Eq. J2-4}) \\ &= 0.60 F_{EXX} \left(\frac{w_{eq}}{\sqrt{2}} \right) k_{ds} \\ &= 0.60 (70 \text{ ksi}) \left(\frac{0.216 \text{ in.}}{\sqrt{2}} \right) (1.0) \\ &= 6.41 \text{ kips/in.} \end{aligned}$$

Using the resistance and safety factors from AISC *Specification* Table J2.5, the available strength is:

LRFD	ASD
$\phi_w = 0.75$ $\phi_w R_n = 0.75(9.15 \text{ kips/in.} + 6.41 \text{ kips/in.})$ $= 11.7 \text{ kips/in.}$	$\Omega_w = 2.00$ $\frac{R_n}{\Omega_w} = \frac{9.15 \text{ kips/in.} + 6.41 \text{ kips/in.}}{2.00}$ $= 7.78 \text{ kips/in.}$

Root opening, R , due to skew:

$$\begin{aligned} R &= t_p \sin \alpha \leq \frac{3}{16} \text{ in.} \\ &= \left(\frac{1}{2} \text{ in.}\right) (\sin 20^\circ) < \frac{3}{16} \text{ in.} \\ &= 0.171 \text{ in.} < \frac{3}{16} \text{ in.} \quad \mathbf{o.k.} \end{aligned}$$

Adjusted size for left weld:

$$\begin{aligned} w_{adj} &= w + R \\ &= \frac{1}{4} \text{ in.} + 0.171 \text{ in.} \\ &= 0.421 \text{ in.} \end{aligned}$$

Use a $\frac{7}{16}$ in. fillet weld to account for the root opening.

Example 3.2—Fillet Weld Size in Shear Connection

Given:

Determine the weld size required to support the dead and live loads for the plate-to-HSS connection shown in Figure 3-20. Also evaluate the capability of the HSS wall to support the loads in direct shear. Use an ASTM A1085/A1085M HSS8.625×0.375 member, ASTM A572/A572M Grade 50 plate (ASTM, 2021e), and 70 ksi filler metal.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

HSS
 ASTM A1085/A1085M
 $F_y = 50$ ksi
 $F_u = 65$ ksi

Plate
 ASTM A572/A572M Grade 50
 $F_{yp} = 50$ ksi
 $F_{up} = 65$ ksi

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the geometric properties of the HSS are as follows:

HSS8.625×0.375
 $t = 0.375$ in.

Required strength

From ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2022), hereafter referred to as ASCE/SEI 7, the required strength based on the vertical loads is:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(23 \text{ kips}) + 1.6(70 \text{ kips})$ $= 140 \text{ kips}$	$P_a = P_D + P_L$ $= 23 \text{ kips} + 70 \text{ kips}$ $= 93.0 \text{ kips}$

Weld size for the eccentric load on the weld group

From AISC *Manual* Table 8-4, for the eccentrically loaded weld group with $e_x = 3$ in. and $l = 9$ in.:

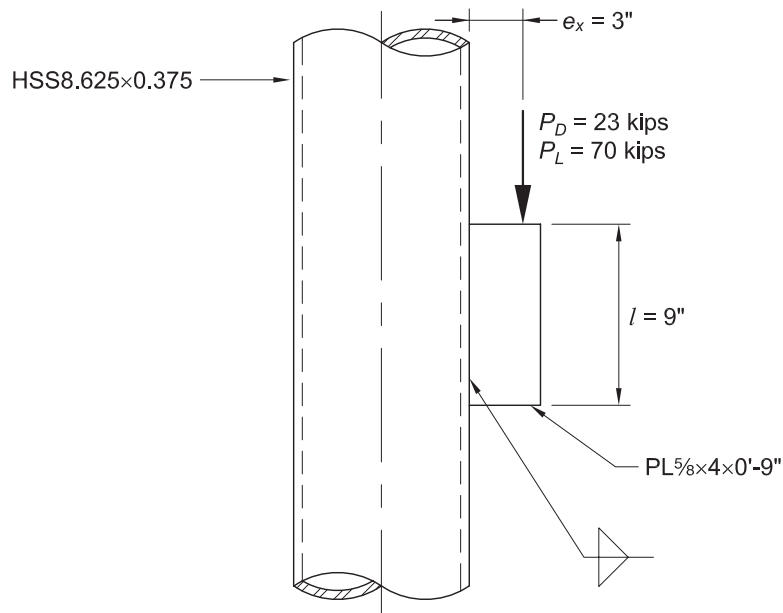


Fig. 3-20. Welded plate-to-round HSS shear connection for Example 3.2.

$$\begin{aligned}
 a &= \frac{e_x}{l} \\
 &= \frac{3 \text{ in.}}{9 \text{ in.}} \\
 &= 0.333
 \end{aligned}$$

For an angle $\theta = 0^\circ$ and the special case shown for the load not in the plane of the weld group, by linear interpolation using the values in the table for $k = 0$:

$$\begin{aligned}
 C &= 3.09 - 0.333(3.09 - 2.66) \\
 &= 2.95
 \end{aligned}$$

From AISC *Manual* Table 8-4, the minimum weld size in sixteenths of an inch is:

LRFD	ASD
$\phi_w = 0.75$ $D_{min} = \frac{P_u}{\phi_w C C_1 l}$ $= \frac{140 \text{ kips}}{0.75(2.95)(1.00)(9 \text{ in.})}$ $= 7.03 \text{ sixteenths}$	$\Omega_w = 2.00$ $D_{min} = \frac{\Omega_w P_a}{C C_1 l}$ $= \frac{2.00(93.0 \text{ kips})}{2.95(1.00)(9 \text{ in.})}$ $= 7.01 \text{ sixteenths}$

Use $\frac{1}{2}$ in. fillet welds.

From Table 3-3, the minimum wall thickness for ASTM A1085/A1085 material to develop a $\frac{1}{2}$ in. fillet weld is 0.381 in. Because $t = 0.375 \text{ in.} < 0.381 \text{ in.}$, the base metal controls.

Effective weld size

From Table 3-2, the effective weld size, D_{eff} , is:

$$\begin{aligned}
 D_{eff} &= 22.6 \left(\frac{F_u}{F_{EXX}} \right) t \\
 &= 22.6 \left(\frac{65 \text{ ksi}}{70 \text{ ksi}} \right) (0.375 \text{ in.}) \\
 &= 7.87 \text{ sixteenths}
 \end{aligned}$$

From AISC *Manual* Table 8-4, the available strength is:

$$\begin{aligned}
 R_n &= C C_1 D_{eff} l \\
 &= 2.95(1.00)(7.87)(9 \text{ in.}) \\
 &= 209 \text{ kips}
 \end{aligned}$$

LRFD	ASD
$\phi_w = 0.75$ $\phi_w R_n = 0.75(209 \text{ kips})$ $= 157 \text{ kips} > 140 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_w = 2.00$ $\frac{R_n}{\Omega_w} = \frac{209 \text{ kips}}{2.00}$ $= 105 \text{ kips} > 93.0 \text{ kips} \quad \mathbf{o.k.}$

Note: The purpose of this example is to illustrate the effective weld size concept. In a complete connection design, other limit states need to be considered, as discussed in Chapter 5 of this Design Guide.

Example 3.3—Transverse Weld to a Square or Rectangular HSS

Given:

Determine the weld size required to support the dead and live tensile loads for the transverse plate-to-HSS connection shown in Figure 3-21. Use an ASTM A500/A500M Grade C HSS8×8×½ member, ASTM A572/A572M Grade 50 plate, and 70 ksi filler metal.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

HSS

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50$ ksi

$F_{up} = 65$ ksi

From AISC *Manual* Table 1-12, the geometric properties are as follows:

HSS8×8×½

$B = 8.00$ in.

$t = 0.465$ in.

Required strength

From ASCE/SEI 7, Chapter 2, the required strength is:

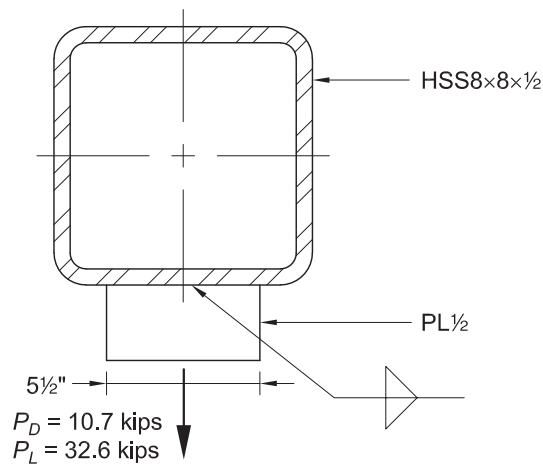


Fig. 3-21. Transverse welded plate-to-square HSS connection for Example 3.3.

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(10.7 \text{ kips}) + 1.6(32.6 \text{ kips})$ $= 65.0 \text{ kips}$	$P_a = P_D + P_L$ $= 10.7 \text{ kips} + 32.6 \text{ kips}$ $= 43.3 \text{ kips}$

Effective weld length

From AISC *Specification* Section K5, with fillet welds on both sides of the plate, the total effective weld length, l_e , is:

$$\begin{aligned}
 B_e &= \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yp} t_p} \right) B_p \leq B_p && \text{(from Spec. Eq. K1-1)} \\
 &= \left[\frac{10}{(8.00 \text{ in.}/0.465 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.465 \text{ in.})}{(50 \text{ ksi})(\frac{1}{2} \text{ in.})} \right] (5\frac{1}{2} \text{ in.}) \leq 5\frac{1}{2} \text{ in.} \\
 &= 2.97 \text{ in.} < 5\frac{1}{2} \text{ in.} \\
 &= 2.97 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 l_e &= 2B_e && \text{(Spec. Eq. K5-4)} \\
 &= 2(2.97 \text{ in.}) \\
 &= 5.94 \text{ in.}
 \end{aligned}$$

Required weld size

From AISC *Specification* Section J2.2a, the effective area of the weld, A_{we} , is:

$$\begin{aligned}
 A_{we} &= \frac{l_e w}{\sqrt{2}} \\
 &= \frac{(5.94 \text{ in.})(w)}{\sqrt{2}} \\
 &= (4.20 \text{ in.})(w)
 \end{aligned}$$

The nominal weld strength, R_n , with a directional strength increase factor, k_{ds} , from AISC *Specification* Equation J2-5 is:

$$\begin{aligned}
 R_n &= F_{nw} A_{we} k_{ds} && \text{(Spec. Eq. J2-4)} \\
 &= 0.60 F_{EXX} A_{we} (1.0 + 0.50 \sin^{1.5} \theta) \\
 &= 0.60 (70 \text{ ksi}) (4.20 \text{ in.})(w) (1.0 + 0.50 \sin^{1.5} 90^\circ) \\
 &= (265 \text{ kips/in.})(w)
 \end{aligned}$$

Note that the fillet weld directional strength increase factor has been used because the attached plate, although loaded in tension, has double-sided welds (see Section 3.1.1). Comparing the available strength to the required strength, the required weld size is determined, using the resistance and safety factors from AISC *Specification* Table J2.5, as follows:

LRFD	ASD
$\phi_w = 0.75$ $\phi_w R_n = 0.75(265 \text{ kips/in.})(w)$ $= (199 \text{ kips/in.})(w)$ $\phi_w R_n \geq P_u$ $(199 \text{ kips/in.})(w) \geq 65.0 \text{ kips}$ $w \geq 0.327 \text{ in.}$	$\Omega_w = 2.00$ $\frac{R_n}{\Omega_w} = \frac{(265 \text{ kips/in.})(w)}{2.00}$ $= (133 \text{ kips/in.})(w)$ $\frac{R_n}{\Omega_w} \geq P_a$ $(133 \text{ kips/in.})(w) \geq 43.3 \text{ kips}$ $w \geq 0.326 \text{ in.}$

Use $\frac{3}{8}$ in. fillet welds. The welds are placed over the full width, on both sides of the plate, giving a total weld length of 11 in. Note that the size of these welds accounts for the uneven force distribution across the plate width. The plate, having a calculated effective width of $B_e = 2.97$ in., and therefore an effective area of 1.49 in.^2 , has an available tensile strength for LRFD (per AISC Specification Section D2) of $67.1 \text{ kips} > 65.0 \text{ kips}$, and thus is also adequate.

Example 3.4—Welds to a Rectangular HSS Overlapped K-Connection Using Effective Lengths

Given:

A rectangular HSS truss contains the 60° overlapped K-connection given in Figure 3-22. The forces shown are in equilibrium. The overlapped branch is the wider and thicker branch member. For fabrication, the overlapped compression (through) branch will be fully welded [overlapped (hidden) toe included] to the chord. The diagonal overlapping branch is then tacked into place, and finally, the whole connection is welded together. The loads shown consist of live loads and dead loads in the ratio of 3:1. Determine the required weld throat for each of the branches using the weld effective length approach. Assume matched electrodes with a specified ultimate strength of 70 ksi. All HSS members are ASTM A1085/A1085M material.

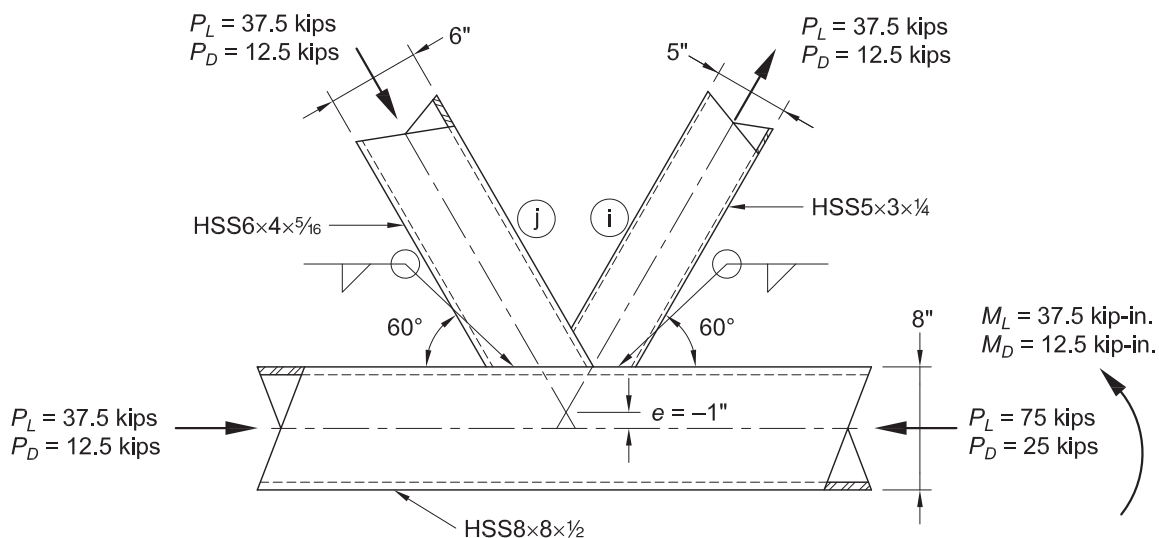


Fig. 3-22. Welded overlapped K-connection with rectangular HSS for Example 3.4.

Solution:

From AISC *Manual* Table 2-4, the HSS material properties are as follows:

ASTM A1085/A1085M

$$F_y = F_{yb} = 50 \text{ ksi}$$

$$F_u = F_{ub} = 65 \text{ ksi}$$

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the geometric properties are as follows:

Chord

HSS8×8×1/2

$$A = 14.4 \text{ in.}^2$$

$$H = 8.00 \text{ in.}$$

$$B = 8.00 \text{ in.}$$

$$t = 0.500 \text{ in.}$$

Branch (overlapped)

HSS6×4×5/16

$$A_{bj} = 5.62 \text{ in.}^2$$

$$H_{bj} = 6.00 \text{ in.}$$

$$B_{bj} = 4.00 \text{ in.}$$

$$t_{bj} = 0.313 \text{ in.}$$

Branch (overlapping)

HSS5×3×1/4

$$A_{bi} = 3.59 \text{ in.}^2$$

$$H_{bi} = 5.00 \text{ in.}$$

$$B_{bi} = 3.00 \text{ in.}$$

$$t_{bi} = 0.250 \text{ in.}$$

Required strength

From ASCE/SEI 7, Chapter 2, the required strength of the connection, expressed as a force in the compression branch and the tension branch, is:

LRFD	ASD
For compression branch and tension branch, $P_u = 1.2P_D + 1.6P_L$ $= 1.2(12.5 \text{ kips}) + 1.6(37.5 \text{ kips})$ $= 75.0 \text{ kips}$	For compression branch and tension branch, $P_a = P_D + P_L$ $= 12.5 \text{ kips} + 37.5 \text{ kips}$ $= 50.0 \text{ kips}$

Determine the amount of overlap

The branch overlap is calculated using Equations 9-1 through 9-6 (see Section 9.2.1).

The connection nodding (work point) eccentricity is $e = -1.00 \text{ in.}$ (negative because the branch centerlines intersect toward the branches relative to the chord centerline).

The overlap length, l_{ov} , measured along the connecting face of the chord beneath the two branches, from geometry, is:

$$\begin{aligned}
l_{ov} &= -x && \text{(from Eq. 9-1)} \\
&= (A + B) - \left(\frac{e + E}{C} \right) \\
&= \left(\frac{H_{bj}}{2 \sin \theta_{bj}} + \frac{H_{bi}}{2 \sin \theta_{bi}} \right) - \left(\frac{e + \frac{H}{2}}{\frac{\sin \theta_{bj} \sin \theta_{bi}}{\sin(\theta_{bj} + \theta_{bi})}} \right) \\
&= \left(\frac{6.00 \text{ in.}}{2 \sin 60^\circ} + \frac{5.00 \text{ in.}}{2 \sin 60^\circ} \right) - \left[\frac{-1 \text{ in.} + \frac{8.00 \text{ in.}}{2}}{\frac{(\sin 60^\circ)^2}{\sin 120^\circ}} \right] \\
&= 6.35 \text{ in.} - 3.46 \text{ in.} \\
&= 2.89 \text{ in.}
\end{aligned}$$

The projected length of the overlapping branch on the chord, l_p :

$$\begin{aligned}
l_p &= \frac{H_{bi}}{\sin \theta_{bi}} \\
&= \frac{5.00 \text{ in.}}{\sin 60^\circ} \\
&= 5.77 \text{ in.} \\
O_v &= \left(\frac{l_{ov}}{l_p} \right) (100\%) \\
&= \frac{2.89 \text{ in.}}{5.77 \text{ in.}} (100\%) \\
&= 50.1\%
\end{aligned}$$

Determine the required weld throat

A continuous weld effective throat will be provided for each branch. Both branches will be welded around the entire perimeter, including the hidden toe portion of the overlapped (through) branch.

From AISC *Specification* Table K5.1, for $50\% \leq O_v < 80\%$, the overlapping member weld effective length is:

$$l_{e,i} = 2 \left\{ \left(1 - \frac{O_v}{100} \right) \left(\frac{H_{bi}}{\sin \theta_i} \right) + \frac{O_v}{100} \left[\frac{H_{bi}}{\sin(\theta_i + \theta_j)} \right] \right\} + B_{eoi} + B_{eov} \quad (\text{Spec. Eq. K5-11})$$

$$\begin{aligned}
B_{eoi} &= \frac{10}{B/t} \left(\frac{F_y t}{F_{ybi} t_{bi}} \right) B_{bi} \leq B_{bi} && (\text{Spec. Eq. K3-13}) \\
&= \frac{10}{(8.00 \text{ in.}/0.500 \text{ in.})} \left[\frac{(50 \text{ ksi})(0.500 \text{ in.})}{(50 \text{ ksi})(0.250 \text{ in.})} \right] (3.00 \text{ in.}) \leq 3.00 \text{ in.} \\
&= 3.75 \text{ in.} > 3.00 \text{ in.} \\
&= 3.00 \text{ in.}
\end{aligned}$$

B_{eoi} is limited to 3.00 in.; however, AISC *Specification* Table K5.1 also states that when $B_{bi}/B > 0.85$ or $\theta_i > 50^\circ$ (which is the case in this example), $B_{eoi}/2$ shall not exceed $B_{bi}/4 = 0.75$. Thus, $B_{eoi} = 1.50$ in.

$$\begin{aligned}
B_{eov} &= \frac{10}{B_{bj}/t_{bj}} \left(\frac{F_{ybj}t_{bj}}{F_{ybi}t_{bi}} \right) B_{bi} \leq B_{bi} & (\text{Spec. Eq. K3-14}) \\
&= \frac{10}{(4.00 \text{ in.}/0.313 \text{ in.})} \left[\frac{(50 \text{ ksi})(0.313 \text{ in.})}{(50 \text{ ksi})(0.250 \text{ in.})} \right] (3.00 \text{ in.}) \leq 3.00 \text{ in.} \\
&= 2.94 \text{ in.} < 3.00 \text{ in.}
\end{aligned}$$

However, AISC *Specification* Table K5.1 also states that when $B_{bi}/B_{bj} > 0.85$ or $(180^\circ - \theta_i - \theta_j) > 50^\circ$ (which is the case in this example), $B_{eov}/2$ shall not exceed $B_{bi}/4 = 0.75$. Thus, $B_{eov} = 1.50 \text{ in.}$

Therefore, the effective length of the weld to the overlapping branch is:

$$\begin{aligned}
l_{e,i} &= 2 \left[\left(1 - \frac{50.1}{100} \right) \left(\frac{5.00 \text{ in.}}{\sin 60^\circ} \right) + \frac{50.1}{100} \left(\frac{5.00 \text{ in.}}{\sin 120^\circ} \right) \right] + 1.50 \text{ in.} + 1.50 \text{ in.} \\
&= 14.5 \text{ in.}
\end{aligned}$$

The effective length of the weld to the overlapped (through) branch is:

$$l_{e,j} = \frac{2H_{bj}}{\sin \theta_j} + 2B_{ej} \quad (\text{Spec. Eq. K5-13})$$

where B_{ej} is given by AISC *Specification* Equation K5-14. However, AISC *Specification* Table K5.1 also states that when $B_{bj}/B > 0.85$ or $\theta_j > 50^\circ$ (which is the case in this example), $l_{e,j}$ is given by:

$$\begin{aligned}
l_{e,j} &= 2(H_{bj} - 1.2t_{bj})/\sin \theta_j & (\text{Spec. Eq. K5-15}) \\
&= 2[6.00 \text{ in.} - 1.2(0.313 \text{ in.})]/\sin 60^\circ \\
&= 13.0 \text{ in.}
\end{aligned}$$

Note that a weld should be provided across the full width of the overlapped branch, B_{bj} , at the toe and the heel locations, transverse to the chord, even though these transverse welds are not considered to be at all effective.

The required weld throat, assuming fillet welds all around, for the tension (overlapping) branch is calculated as follows using AISC *Specification* Equation K5-1 and the resistance and safety factors from AISC *Specification* Table J2.5. This assumes that the flat width of the overlapped branch is adequate for fillet welding.

$$\begin{aligned}
R_n &= F_{nw}t_{weld}l_e \\
&= 0.60(70 \text{ ksi})(t_{weld})(14.5 \text{ in.}) \\
&= (609 \text{ kips/in.})t_{weld}
\end{aligned}$$

LRFD	ASD
$\phi_w = 0.75$ $\phi_w R_n \geq P_u$ $0.75(609 \text{ kips/in.})t_{weld} \geq 75.0 \text{ kips}$ $t_{weld} \geq 0.164 \text{ in.}$	$\Omega_w = 2.00$ $\frac{R_n}{\Omega_w} \geq P_a$ $\frac{(609 \text{ kips/in.})t_{weld}}{2.00} \geq 50.0 \text{ kips}$ $t_{weld} \geq 0.164 \text{ in.}$

The required weld throat, assuming fillet welds all around, for the compression (overlapped/through) branch is as follows using AISC *Specification* Equation K5-1 and the resistance and safety factors from AISC *Specification* Table J2.5. In this case, the fillet weld directional strength increase factor, k_{ds} , may be used because the member is in axial compression (see Section 3.1.1); however, this is not utilized because of local secondary bending moments in the overlapped region of the connection.

$$\begin{aligned}
R_n &= F_{nw} t_{weld} l_e \\
&= 0.60(70 \text{ ksi})(t_{weld})(13.0 \text{ in.}) \\
&= (546 \text{ kips/in.}) t_{weld}
\end{aligned}$$

LRFD	ASD
$\phi_w = 0.75$ $\phi_w R_n \geq P_u$ $0.75(546 \text{ kips/in.}) t_{weld} \geq 75.0 \text{ kips}$ $t_{weld} \geq 0.183 \text{ in.}$	$\Omega_w = 2.00$ $\frac{R_n}{\Omega_w} \geq P_a$ $\frac{(546 \text{ kips/in.}) t_{weld}}{2.00} \geq 50.0 \text{ kips}$ $t_{weld} \geq 0.183 \text{ in.}$

For both branches, a required weld throat has been determined. The actual weld sizes around the branches will then depend on the various weld geometries at different positions around the connection. Note that in a complete connection design, one would also check that the connection available axial strength, according to AISC *Specification* Table K3.2, met or exceeded the required strength. Limits of applicability from the *Specification* Table K3.2A must also be observed, for use of Table K3.2. This procedure is demonstrated in Example 9.4 of this Design Guide.

Example 3.5—Welds to a Rectangular HSS Overlapped K-Connection to Develop Branch Capacity

Given:

The same overlapped K-connection as in Example 3.4, subjected to the same loads, is reconsidered with a different weld design approach. In this example, the welds to the branches will be designed to develop the branch member yield strength at all locations around the branch perimeter. Note that here the effective area of the weld, A_{we} , is the product of the weld throat, t_{weld} , and a unit length of weld. The nominal yield strength of the attached branch wall is the product of the branch yield stress, F_{yb} , the branch design wall thickness, t_b , and a unit length of branch.

Solution:

The weld rupture available strength will be compared to the available yield strength of the attached branch (base) metal. The required weld throat, t_{weld} , assuming fillet welds are used, can be determined from AISC *Specification* Section J2.4. For the tension (overlapping) branch, with $k_{ds} = 1.0$:

$$\begin{aligned}
R_n &= F_{nw} A_{we} k_{ds} && (\text{Spec. Eq. J2-4}) \\
&= 0.60 F_{EXX} t_{weld} (1 \text{ in.}) k_{ds} \\
&= 0.60(70 \text{ ksi})(t_{weld})(1 \text{ in.})(1.0) \\
&= (42.0 \text{ kips/in.}) t_{weld}
\end{aligned}$$

$$\begin{aligned}
P_n &= F_y A_g && (\text{Spec. Eq. D2-1}) \\
&= F_y t_{bi} (1 \text{ in.}) \\
&= (50 \text{ ksi})(0.250 \text{ in.})(1 \text{ in.}) \\
&= 12.5 \text{ kips}
\end{aligned}$$

Using the resistance and safety factors from AISC *Specification* Table J2.5 and Section D2:

LRFD	ASD
$\phi_w = 0.75$ $\phi_t = 0.90$ $\phi_w R_n \geq \phi_t P_n$ $0.75(42.0 \text{ kips/in.})t_{weld} \geq 0.90(12.5 \text{ kips})$ $t_{weld} \geq 0.357 \text{ in.}$	$\Omega_w = 2.00$ $\Omega_t = 1.67$ $\frac{R_n}{\Omega_w} \geq \frac{P_n}{\Omega_t}$ $\frac{(42.0 \text{ kips/in.})t_{weld}}{2.00} \geq \frac{12.5 \text{ kips}}{1.67}$ $t_{weld} \geq 0.356 \text{ in.}$

For the compression (overlapped or through) branch, the required weld throat is as follows. In this case, the fillet weld directional strength increase factor may be used because the member is in axial compression (see Section 3.1.1); however, it is not utilized for parity with Example 3.4 (i.e., k_{ds} is taken as 1.0).

$$\begin{aligned}
 R_n &= F_{nw} A_{we} k_{ds} && (\text{Spec. Eq. J2-4}) \\
 &= 0.60 F_{EXX} t_{weld} (1 \text{ in.}) k_{ds} \\
 &= 0.60 (70 \text{ ksi}) (t_{weld}) (1 \text{ in.}) (1.0) \\
 &= (42.0 \text{ kips/in.}) t_{weld}
 \end{aligned}$$

$$\begin{aligned}
 P_n &= F_n A_g && (\text{Spec. Eq. E3-1}) \\
 &= F_y t_{bi} (1 \text{ in.}) \\
 &= (50 \text{ ksi}) (0.313 \text{ in.}) (1 \text{ in.}) \\
 &= 15.7 \text{ kips}
 \end{aligned}$$

Note that the full yield strength of the compression branch is used here, neglecting the reduction in compressive strength due to buckling. This approach is conservative as it will overestimate the required weld throat.

Using the resistance and safety factors from AISC *Specification* Table J2.5 and Section E1:

LRFD	ASD
$\phi_w = 0.75$ $\phi_c = 0.90$ $\phi_w R_n \geq \phi_c P_n$ $0.75(42.0 \text{ kips/in.})t_{weld} \geq 0.90(15.7 \text{ kips})$ $t_{weld} \geq 0.449 \text{ in.}$	$\Omega_w = 2.00$ $\Omega_c = 1.67$ $\frac{R_n}{\Omega_w} \geq \frac{P_n}{\Omega_c}$ $\frac{(42.0 \text{ kips/in.})t_{weld}}{2.00} \geq \frac{15.7 \text{ kips}}{1.67}$ $t_{weld} \geq 0.448 \text{ in.}$

For the compression (overlapped or through) branch, the weld need only be made around three sides of the branch because the “hidden toe” may be left unwelded or just tacked. This is permissible according to AISC *Specification* Section K5 because the connection will be designed in accordance with AISC *Specification* Table K3.2, the branch member component forces normal to the chord do not differ by more than 20%, and the remaining welds to the overlapped branch everywhere develop the full capacity of the overlapped branch member walls.

For this branch capacity design method, the required weld throat to the tension branch (0.357 in.) is more than double that required for the alternate effective length design method demonstrated in Example 3.4 (0.164 in.). In Design Examples 3.4 and 3.5, this tension member was only loaded to 46% of its design (available) tensile strength; this illustrates the significant reduction

in weld volume that can sometimes be achieved, for lightly loaded members, by using the effective length approach. Also, for this branch capacity design method, the required weld throat to the compression branch (0.449 in.) is 2.4 times that required for the alternate effective length design method demonstrated in Example 3.4 (0.183 in.), with the member loaded in this case to 30% of its available yield strength, but the weld need only be made to 82% of the branch perimeter. Other aspects do need to be considered with the weld effective length method, such as the truss fabrication implications of welding the hidden toe.

Chapter 4

Mechanical Fasteners and Bolted Joints

The use of mechanical fasteners, such as bolts, in steel building structures is often desirable because of ease of erection and the reliability of the quality of the connection. However, the use of mechanical fasteners with HSS poses a challenge due to difficulties in accessing the interior of the member. As a result, many HSS bolted connections utilize flange plate or gusset plate connections at the end of the member that allow for access to both the bolt head and nut. Hidden bolted connections provide a similar feature while later covering the connection to present a clean, uniform appearance.

Special consideration is needed when connecting directly to the sidewall of the HSS away from its end, such as with axially loaded single-plate connections or lap-splice connections. The use of an access hole in one of the walls in which the bolt is not installed (Figure 4-1) can allow for the ability to properly install the bolt. The access hole should be rounded so as not to introduce locations for potential crack initiation. Access hole size should be coordinated with the fabricator, with a minimum of 4½ to 6 in. considered. When left open, the access hole also leads to a loss of cross-sectional area, which needs to be considered in calculating the member's strength (Sherman, 1995). However, the loss of strength may not be critical if, at that location, the load carried does not require the full strength of the HSS. The section may be evaluated for the reduced section using a method similar to that outlined in AISC Design Guide 2 *Composite Beam with Web Openings* (Darwin, 1990) for web openings or by calculating the properties of the reduced section. Alternatively, the hole can be covered and sealed, allowing the member to achieve its original design strength. Sealing the HSS may be necessary, particularly in cases where the connection is exposed to outside elements or adverse environments. See Section 1.6.3.

The use of proprietary mechanical fasteners, typically blind bolts (Section 4.5), that only require access to the outside of the HSS, provides an alternative means of connecting HSS (see AISC *Manual* Part 7, Special Considerations for Hollow Structural Sections). In deciding what approach to take in connecting HSS when the use of mechanical fasteners is desired, three considerations should be taken into account: the magnitude of the load being transferred, the thickness of the HSS wall, and the location of the connection along the member.

Similar to standard connections, mechanical fasteners in HSS connections may be loaded in shear, tension, or a combination of the two. Traditional approaches to connection design, as outlined in the AISC *Specification* and AISC *Manual* can be applied with consideration of additional limit states specific to HSS bolted connections—tension pull-out and HSS wall distortion. Section 4.1 details these limit states which are dependent on the type of loading and whether the connection is bearing-type or slip-critical. The influence of prying action due to the flexibility of the HSS wall and other connection elements often also needs to be considered when calculating the strength of the mechanical fasteners depending on the connection configuration (Section 4.3).

AISC *Specification* Sections A3.3 and A3.4 provide material specifications for bolting components (bolts, washers, and nuts) and threaded rods, respectively. Nominal strengths for fasteners and connected parts can be found in AISC *Specification* Table J3.2. The Research Council on Structural Connections (RCSC) *Specification for Structural Joints Using High-Strength Bolts* (RCSC, 2020), hereafter referred to as the RCSC *Specification*, provides further information on fastener components, typical limit states, installation, and inspection associated with bolted joints.



Fig. 4-1. Splice in a double-chord truss where the bolts have been installed with the aid of access holes.

4.1 BOLTED HSS LIMIT STATES

Conventional fastener limit states found in AISC *Specification* Section J3 apply to mechanically fastened HSS connections. Beyond these limit states, there are additional HSS specific limit states that may also need to be considered.

4.1.1 Tension Pull-Out

Tension pull-out (or punching shear of the fastener through the HSS face) can occur when a fastener directly attached to an HSS wall is placed in tension. Korol et al. (1993) showed that the HSS wall thickness is the critical parameter for this limit state based on tests of extended end-plate moment connections between wide-flange beams and HSS columns, using either traditional high strength bolts or blind bolts. The nominal strength for pull-out in tension can then be based on the factored resistance of the column face in punching shear, leading to the following nominal strength for pull-out of a single fastener through an HSS wall (Packer and Henderson, 1997):

$$R_n = 0.6F_u\pi d_p t \quad (4-1)$$

where

F_u = specified minimum tensile strength of the HSS, ksi

d_p = diameter of the part (head or nut) in contact with the inner surface of the HSS, in.

t = thickness of the HSS wall, in.

A resistance factor, $\phi = 0.75$ for LRFD, or a safety factor, $\Omega = 2.00$ for ASD, is required for this limit state. By ensuring

that the factored resistance for pull-out of a single fastener in tension is greater than the factored tensile strength of the fastener, a minimum wall thickness necessary to avoid pull-out can be obtained.

4.1.2 HSS Wall Distortion

The HSS wall distortion limit state is associated with the flexibility of the HSS wall and is determined from yield-line theory. It can occur when the slenderness ratio of the column face is medium to large (Packer and Henderson, 1997). To calculate the nominal resistance to this failure mode, a yield-line analysis can be treated similar to an HSS 90° T-connection (see Section 2.3.2) with the branch member represented by the bolt group. AISC *Manual* Equation 9-44 can be applied with the width of the bolt group, c , measured across the width of the HSS wall, being taken as $w_b = (n-1)b_b + d_p$, and the length of the bolt group along the length of the HSS wall, L , being taken as $l_b = (m-1)h + d_p$, where w_b is the width of the bolt group, l_b is the length of the bolt group, n is the number of bolts in one line across the width of the HSS, b_b is the center-to-center fastener spacing across the width of the HSS, m is the number of bolts in one line along the length of the HSS, and h is the center-to-center fastener spacing along the length of the HSS—assuming that the holes are evenly spaced along the HSS width and length, respectively, as shown in Figure 4-2. If the bolt group is centered on the face of the HSS, then a and b in AISC *Manual* Figure 9-5(a) equal $0.5(B - w_b)$, where B is the width of the HSS member. The resulting nominal strength for HSS wall distortion for a bolt group applying tension to the face of an HSS wall is:

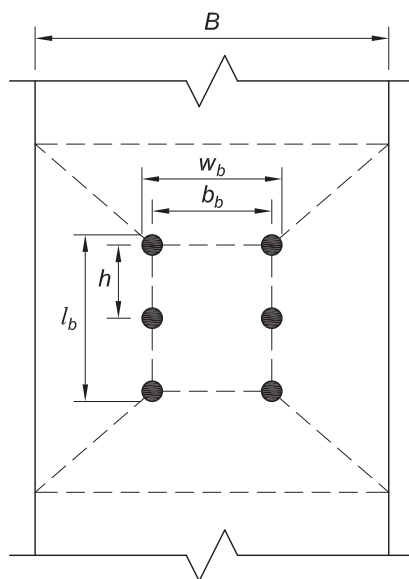


Fig. 4-2. Yield-line pattern for a group of tension fasteners.

$$R_n = F_y t^2 \left[\frac{2 \left(\frac{l_b}{B} \right)}{\left(1 - \frac{w_b}{B} \right)} + \frac{4}{\sqrt{\left(1 - \frac{w_b}{B} \right)}} \right] Q_f \quad (4-2)$$

A resistance factor, $\phi = 1.00$ for LRFD, or a safety factor, $\Omega = 1.50$ for ASD, is required for this limit state.

It is expected, and acceptable, that some distortion may occur if all the lines do not yield.

4.2 FASTENERS IN SHEAR

For the case where the connection is loaded in shear, such as a WT-shape or angle bolted to the face of an HSS column where the WT-shape or angle are loaded parallel to the column, the fasteners will be placed primarily in shear. The design of these connections is no different than the design of other typical shear connections made to W-shape webs or flanges. For the case where the fasteners in a bearing-type connection, such as those mentioned, are subject to shear only, the limit states of bolt shear strength, bolt bearing, and bolt tearout need to be checked along with the limit states associated with the strength of the HSS and connecting element. If the shear connection is at the end of an HSS member, block shear failure needs to be considered.

The nominal shear strength of a bolt group in AISC *Specification* Section J3.7 is:

$$R_n = (F_{nv} A_b) n \quad (4-3)$$

where

A_b = the unthreaded body area of the bolted part, in.²

F_{nv} = nominal shear stress given in AISC *Specification* Table J3.2, ksi

n = the number of bolts resisting the shear

An increase in the number of plies within the connection can lead to an increase in the number of shear planes, which will increase the strength of the connection. However, if a through-bolt connection is used to carry the shear, and the load is only applied along one face of the HSS member, the plies associated with the loaded face are the only ones that are considered in the calculation of the connection capacity. A resistance factor, $\phi = 0.75$ for LRFD, or a safety factor, $\Omega = 2.00$ for ASD, is required for this limit state.

The limit states of bolt bearing and tearout are independent of the bolt and are a function of the material being connected. Provided that the bolt hole is standard, oversized, short-slotted (any direction), or long-slotted (parallel to the load) and the deformation of the hole at service loads is a design consideration, the nominal bolt bearing strength from AISC *Specification* Section J3.11 is:

$$R_n = 2.4 d t F_u \quad (\text{Spec. Eq. J3-6a})$$

where

F_u = specified minimum tensile strength of the HSS wall, ksi

d = nominal fastener diameter, in.

t = thickness of the HSS wall, in.

Given the same conditions, the nominal bolt tearout strength in AISC *Specification* Section J3.11 is:

$$R_n = 1.2 l_c t F_u \quad (\text{Spec. Eq. J3-6c})$$

where

l_c = clear distance in the direction of the force between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.

A resistance factor, $\phi = 0.75$ for LRFD, or a safety factor, $\Omega = 2.00$ for ASD, is required for these limit states. If the deformation of the hole is not a design consideration, then the bearing and tearout nominal strength can be increased based on AISC *Specification* Section J3.11. The exception to AISC *Specification* Equation J3-6a is for through bolts where AISC *Specification* Section J3.11b(1) refers to Section J7 and Equation J7-1. The presence of an unconfined surface in combination with the flexible HSS wall requires special attention for through bolt connections, as is discussed in Section 4.5. Example 4.1 illustrates the design of a shear connection to an HSS column that has through bolts loaded in shear.

4.3 FASTENERS IN TENSION

When fasteners act in tension, such as with a hanger connection where the hanger is attached to the face of an HSS wall, the limit states are the same as those when attached to a W-shape (tensile strength of bolt) with two additional limit states of tension pull-out and HSS wall distortion. These two additional limit states are discussed in detail in Section 4.1.

The nominal tensile strength of a bolt group in AISC *Specification* Section J3.7 is:

$$R_n = (F_{nt} A_b) n \quad (4-4)$$

where

F_{nt} = nominal tensile stress given in AISC *Specification* Table J3.2, ksi

A resistance factor, $\phi = 0.75$ for LRFD, or a safety factor, $\Omega = 2.00$ for ASD, is recommended for this limit state.

4.3.1 Prying Action

One unique aspect of bolted connections in tension is the potential for prying action. In a connection where the fasteners are put into tension, the connected element can deform in single- or double-curvature bending, accompanied by

increased tensile force in the bolts, as illustrated in Figures 4-4 and 4-5. This will tend to increase the available strength of the connecting element, while decreasing the available tensile strength of the bolts to resist the applied tension.

Prying forces should be avoided in bolted joints under dynamic loading (e.g., fatigue). As indicated in the scope of this document, fatigue loading is not considered herein. In statically loaded structures, the effects of potential prying action on bolt behavior must be considered, with a connection design based on an appropriate force distribution.

A design that eliminates prying action will tend to optimize the strength of the bolts for the applied tension. A force distribution that accommodates the effect of prying action will tend to optimize the flexural strength of the affected elements for the applied tension. If the elements are sufficiently stiff, the flexural deformation will be small compared to the elongation of the fasteners, and minimal prying forces will develop. When at least one of the elements is flexible, the larger permitted deformations will induce prying forces, resulting in increased tensile forces in the bolts.

AISC *Manual* Part 9 outlines one of many force distribution models for conditions similar to that shown in Figure 4-3. As seen in AISC *Manual* Figure 9-4, the conditions covered in the AISC *Manual* only consider flexure of one attached element. When the connecting element is attached to the flexible wall of an HSS, the HSS wall also has the potential to deform. Unlike the conditions addressed in the AISC *Manual*, the deformation of the elements may not be symmetric about the faying surface. This leads to a more complex model for addressing prying action.

For the condition of an HSS bolted to an infinitely stiff support, the HSS wall has the potential to deform and cause an increase in the tensile forces in the bolts, even if the connecting element does not. Prying forces will develop in the middle of the connected HSS face (see Figure 4-4). This condition will generally tend to increase the available flexural strength of the HSS wall by introducing another potential yield line.

The connection of a flexible WT bolted to a flexible wall of an HSS creates a unique condition illustrated in Figure 4-5. The deformation of the WT flange tends to separate the joint where contact is necessary to develop prying action. This condition creates a challenge due to the multitude of variables affecting the analysis and design of the joint. Specific conditions have been addressed in two papers by Wheeler et al. (1998, 2003) in which specific parameters are set forth.

The connected element and HSS wall must also have adequate flexural strength and stiffness to resist the additional deformation and the loads resulting from prying. The load path and behavior of the connection elements must be carefully considered. Prying action may not develop in every joint. For example, if tension causes the connected element to separate away from the HSS wall, causing no contact between the plies, prying cannot exist due to the absence of a lever point.

4.4 FASTENERS IN COMBINED SHEAR AND TENSION

Most fasteners are considered to be loaded only in shear or in tension, but there are cases where combined shear and tension may act on a bearing-type connection. For these cases, the limit states of bolt tensile strength, including the influence of shear and possibly prying action; bolt bearing; and bolt tearout need to be considered. However, the User Note in AISC *Specification* Section J3.8 states that if the required stress in either tension or shear is less than or equal to 30% of the corresponding available stress, then the influence of combined shear and tension can be ignored. The ability to ignore the interaction of shear and tension for this case derives from the elliptical relationship between shear and tension that can be simplified to a set of three lines (horizontal, diagonal, and vertical). The available tensile strength can be calculated considering the influence of shear with a resistance factor, $\phi = 0.75$ for LRFD, or a safety factor, $\Omega = 2.00$ for ASD:

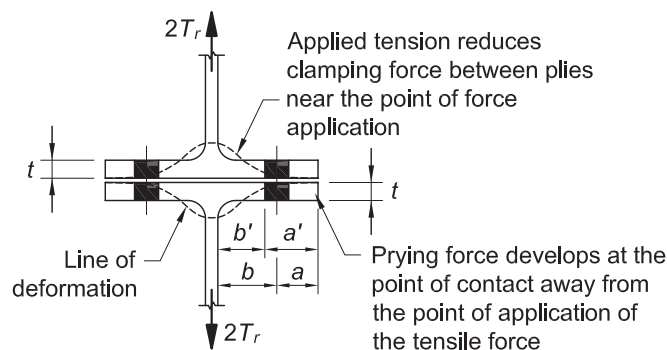


Fig. 4-3. Prying action conditions covered by the AISC Manual.

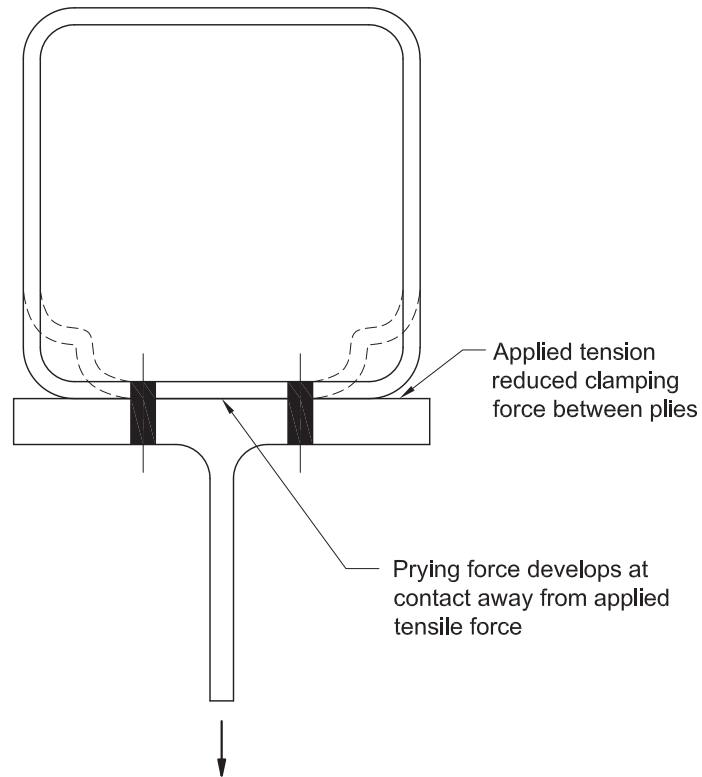


Fig. 4-4. Deformation of thick-flanged WT bolted to HSS connection.

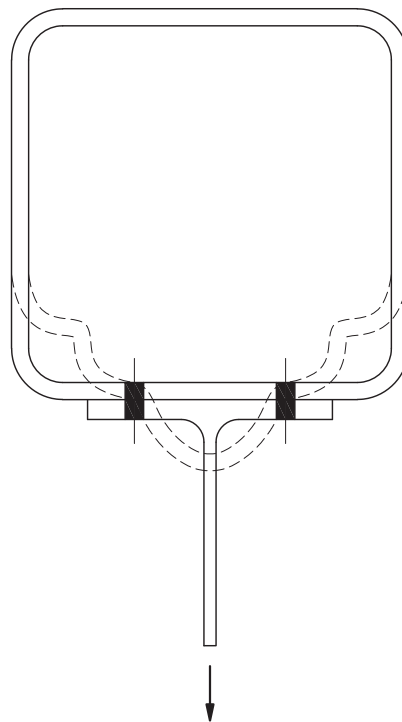


Fig. 4-5. Deformation of thin-flanged WT bolted to HSS connection.

$$R_n = F_{nt}' A_b \quad (\text{Spec. Eq. J3-2})$$

where

F_{nt}' = nominal tensile stress modified to include the effects of shear stress, ksi (Spec. Eq. J3-3a)

$$= 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt} \quad (\text{LRFD})$$

$$= 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv} \leq F_{nt} \quad (\text{ASD})$$

f_{rv} = required shear stress using LRFD or ASD load combinations, ksi (Spec. Eq. J3-3b)

4.5 OTHER MECHANICAL FASTENERS

The unique circumstances when connecting to HSS makes it difficult to always use standard bolts and nuts, particularly when the connection is made away from the end of the member. A number of alternative techniques for bolting HSS have been developed where access to the inside of the HSS is not required. These techniques include the use of through bolts and a number of different single-sided methods or bolts that are often proprietary (Figure 4-6). These connections often can be loaded in shear, tension, or a combination of shear and tension.

4.5.1 Through Bolts

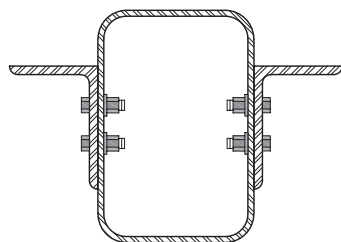
For connections where the fastener is placed in shear—such as a beam end-plate to HSS column connection or an angle attached to the face of an HSS column—through bolts that pass through the HSS are an option provided pretension is not required. The through bolt (or threaded rod) needs to be long enough to extend beyond the HSS wall to allow full engagement of the nut. In general, the required length of a through bolt can be calculated as the sum of the grip [length from the bottom of the bolt head to the inside face of the washer, which for Figure 4-6(b) is the width of the HSS + two times the thickness of the angle leg], the thickness of the washer,

and an allowance for the height of the nut and proper stick out [AISC Manual Table 7-14 for ASTM F3125/F3125M Grade A325 and A490 bolts (ASTM, 2022a)]. As the HSS member gets larger, the required length of a through bolt can limit the type of bolt that can be specified, and installation can become difficult due to alignment problems. In general, through bolting to round HSS requires contoured washers and is typically not advised due to the lack of a flat bearing surface for the bolt head and nut.

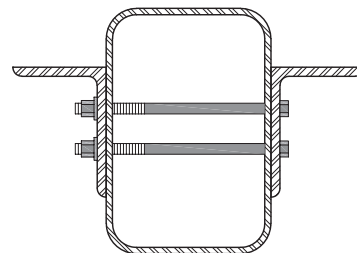
Pretensioning of through bolts cannot be accomplished when the HSS is not reinforced due to the inherent flexibility of the HSS face. The inner wall of the HSS is unconfined, which can lead to distortion when the through bolt is pretensioned as shown in Figure 4-7 (D'Antimo et al., 2017). The potential for instability in the HSS wall limits the use of through bolts to bearing connections where the bolts are only installed snug tight. Even with snug tight tensioning [approximately 10% of the minimum bolt pretension (AISC Specification Table J3.1)], care must be taken when installing through bolts in thin-walled HSS members. The flexibility of the face can also make it difficult to achieve firm contact with all the plies, which would not comply with the standard definition of snug tight (Kloiber, 2001). As a result, through bolts may not be cost effective or practical.

AISC Manual Part 7 states that when a through bolt is used to directly connect to an HSS member and the through bolt is designed for static shear, the only limit state involving the HSS member is bolt bearing. The presence of the unconfined face on the inside of the HSS member suggests that the behavior of a through-bolt connection more closely follows that of a pin in bearing than a bolt in bearing because the grip of the through bolt is not restrained against expansion (Packer et al., 2010b). The bearing resistance of a pin is reduced from that of a bolt to account for the fact that the load distribution in a through bolt differs from that of a standard nut (Rex and Easterling, 2003; Duerr, 2006). The limit state equation for through bolt bearing is then given as:

$$R_n = 1.8F_y A_{pb} \quad (\text{Spec. Eq. J7-1})$$



(a) Single-sided bolted connection



(b) Through-bolted connection

Fig. 4-6. Connections directly to an HSS sidewall.

where

$$A_{pb} = \text{projected area in bearing, in.}^2 \\ = dt$$

F_y = specified minimum yield stress of the HSS member, ksi

A resistance factor, $\phi = 0.75$ for LRFD, or a safety factor, $\Omega = 2.00$ for ASD, is required for this limit state. The projected area in bearing is a function of the diameter of the through bolt, d , and the design wall thickness of the HSS, t .

As with other bearing joints, bolt shear, tearout, and all other limit states associated with the connected member need to be checked to complete the design of the connection.

4.5.2 Blind Bolts

A number of proprietary blind bolts are available for use in structural applications. Some of these fasteners meet the requirements for pretensioned ASTM F3125/F3125M Grade A325 bolts (Henderson, 1996). In general, however,

it cannot be assumed that blind bolts will have equivalence to high-strength ASTM F3125/F3125M bolts, and certified properties for tension and shear loading need to be verified. Blind bolts are unique in that they can be installed from the outside of the HSS (Figure 4-8), and a mechanism within the bolt on the inside of the HSS ensures firm contact with the inner portion of the HSS, eliminating the need for a nut to be installed (Korol et al., 1993). All types of blind bolts use proprietary methods; the flow-drilling process produces a threaded hole to permit bolting without the need for access to the inside of the HSS (Sherman, 1995). The flow-drilling process provides benefits over simply drilling and tapping the hole because flow-drilling locally increases the wall thickness, and wall thicknesses greater than $\frac{5}{8}$ in. are often required to provide adequate pull-out capacity (Kurobane et al., 2004). Both blind-bolt and flow-drilled hole connections with bolts in tension typically see deformation of the HSS face prior to failure of the fastener (Yeomans, 1998). In general, the HSS limit states for blind bolts are the same as

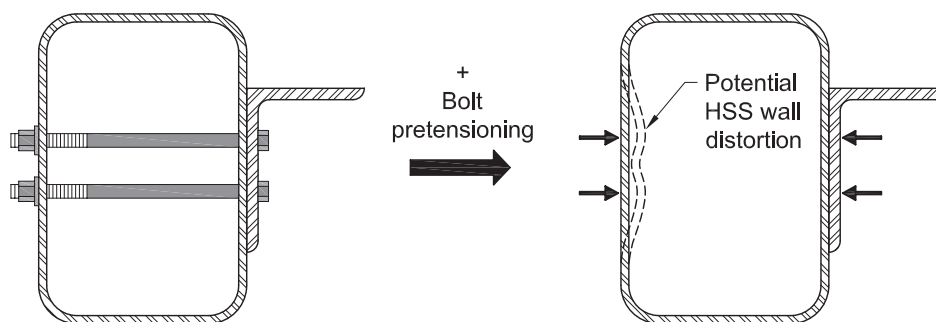


Fig. 4-7. Potential distortion of the HSS sidewall if through bolts are fully pretensioned.



Fig. 4-8. Beam-to-column end-plate connection using blind bolts.

those for standard bolts, including bolt bearing in shear, bolt tearout in shear, tearout of the bolt in tension, and HSS wall distortion due to tension in the bolt.

4.5.3 Threaded Studs

Rather than drilling a hole into the HSS wall, threaded studs may be either shop- or field-welded to the side of the HSS with a stud-welding gun. The threaded studs are typically limited to diameters of $\frac{3}{8}$ in. and $\frac{7}{8}$ in. The threaded stud acts like a bolt, without the need for access to the inside of the HSS for bolt installation. Maquoi et al. (1985) showed that, when properly welded, threaded studs behave in a similar manner to a standard bolt and can be designed as such provided an additional limit state check is conducted for punching shear and tearout. In general, the limit state of concern is distortion of the HSS sidewall, as this will typically control due to the flexible HSS face, unless the HSS wall is reinforced. For designers, it is important to remember that there is a fillet weld at the base of the stud that limits the ability for a connecting element to sit flush against the HSS wall. To rectify this situation, the connecting elements hole must be countersunk to remain flush against the HSS face.

4.5.4 Powder-Actuated Fasteners

A method for splicing circular HSS, which has potential for site fabrication of poles, involves powder-actuated fasteners (PAF). Circular HSS can be spliced together when either one HSS fits inside the other, forming a telescoping connection, or a sleeve is used around two HSS with equal diameters. In both cases, PAF are installed with a powder-actuated gun to make the connection between the two HSS or the sleeve and

the HSS. Such connections are satisfactory for shear connection but are limited to cases where the total thickness does not exceed $\frac{1}{2}$ in. Packer (1996) conducted tests (Figure 4-9) that identified the failure modes of HSS connected with PAF. The observed failure modes, analogous to bolted or riveted joints, are fastener shear failure, HSS bearing failure, and net section fracture of the HSS. The shear capacity of a PAF connection can be obtained as the number of fasteners multiplied by the shear strength of a single fastener. The influence of the small indentations that form as a result of the fastener installation process is typically ignored when calculating the capacity.

4.5.5 Screws

Self-tapping screws provide another alternative to bolts for light connections to HSS members. The available screws, which may or may not have self-drilling points, have diameters between 0.08 in. and 0.25 in. The combined thickness of the HSS wall and connecting element is limited to $\frac{1}{2}$ in. The American Iron and Steel Institute (AISI) *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2016) Sections J4.3 and J4.4 provide the limit states associated with self-tapping screws, including screw strength in shear as well as pull-out and pull-over strength in tension. The minimum spacing between centers of the fasteners should not be less than three times the nominal screw diameter and the edge distance from the centerline of the fastener should not be less than 1.5 times the nominal screw diameter.

The nominal shear strength, P_{nv} , of a screwed joint, per screw, is given in the following with a resistance factor, $\phi = 0.50$ for LRFD, or a safety factor, $\Omega = 3.00$ for ASD.



Fig. 4-9. Fastening two mating HSS together.

For $t_1/t_2 \leq 1.0$, P_{nv} is taken as the smallest of:

$$P_{nv} = 4.2\sqrt{t_2^3 d} F_{u2} \quad (4-5)$$

$$P_{nv} = 2.7t_1 d F_{u1} \quad (4-6)$$

$$P_{nv} = 2.7t_2 d F_{u2} \quad (4-7)$$

where

F_{u1} = tensile strength of element in contact with screw head or washer, ksi

F_{u2} = tensile strength of element not in contact with screw head or washer, ksi

d = nominal screw diameter, in.

t_1 = thickness of element in contact with screw head or washer, in.

t_2 = thickness of element not in contact with screw head or washer, in.

For $t_1/t_2 \geq 2.5$, P_{nv} is taken as the smallest of:

$$P_{nv} = 2.7t_1 d F_{u1} \quad (4-8)$$

$$P_{nv} = 2.7t_2 d F_{u2} \quad (4-9)$$

For $1.0 < t_1/t_2 < 2.5$, P_{nv} is taken as a linear interpolation between the previous two cases.

The nominal strength in tension, P_{nt} , is calculated for the limit state of pull-out strength of a joint, per screw (Equation 4-10), and pull-over strength (Equation 4-11) of a joint per screw, respectively:

$$P_{nt} = 0.85t_c d F_{u2} \quad (4-10)$$

$$P_{nt} = 1.5t_1 d'_w F_{u1} \quad (4-11)$$

where

d_{sh} = screw head diameter, in.

d_w = steel washer diameter, in.

d'_w = effective pull-over diameter, in.

$$= d_{sh} + 2t_w + t_1 \leq d_w$$

t_c = lesser of depth of penetration and thickness, t_2 , in.

t_w = steel washer thickness, in.

Failure of the screws themselves, rather than the connected plies, can be prevented by ensuring that the screws are 25% stronger than the required shear or tensile resistance.

4.5.6 Structural Nut Holders

A possible alternative to blind bolts, structural nut holders eliminate required access to install and secure a nut. The nut is placed in a holder device, which is secured to the structural member or backing plate during fabrication. The device allows bolts to be tightened to a snug tight or pretensioned condition as required for the joint design requirements. This device can be used for HSS connections and other connections with erection issues that provide ideal bolt clearance for typical installation. The associated HSS limit states are bearing and tearout at bolt holes in shear and wall distortion. Manufacturers' literature must be consulted for installation and use of specific devices.

4.6 BOLTED JOINT DESIGN EXAMPLES

Example 4.1— Through Bolts in Shear

Given:

Verify the adequacy of the through-bolt connection shown in Figure 4-10, with a dead load, $P_D = 2$ kips, and a live load, $P_L = 4$ kips. The HSS is ASTM A500/A500M Grade C, and the angle is ASTM A572/A572M Grade 50.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

HSS

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

Angle

ASTM A572/A572M Grade 50

$F_y = 50$ ksi

$F_u = 65$ ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS
HSS8×3×¼
 $t = 0.233$ in.

Required strength

From ASCE/SEI 7, Chapter 2, the required strength of the connection is:

LRFD	ASD
$P_r = P_u$ $= 1.2(2 \text{ kips}) + 1.6(4 \text{ kips})$ $= 8.80 \text{ kips}$	$P_r = P_a$ $= 2 \text{ kips} + 4 \text{ kips}$ $= 6.00 \text{ kips}$

Resultant load per bolt

The applied load is eccentric to the centroid of the bolt group with a 4 in. bolt spacing. AISC *Manual* Table 7-7 could be used with interpolation between the values obtained for a 3 in. and 6 in. spacing. Alternatively, the elastic method described in AISC *Manual* Part 7 can be used. The latter approach will be used here.

The eccentric load, P_r , is resolved into a direct shear, V_v , acting through the center of gravity of the bolt group and a moment, $P_r e$, where $e = 2$ in. is the eccentricity. The load per bolt due to the direct shear is:

$$V_v = \frac{P_r}{2}$$

$$= 0.5P_r$$

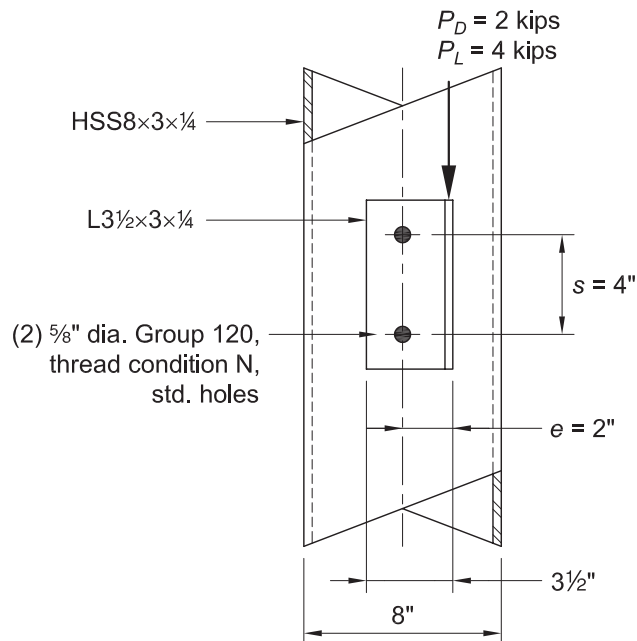


Fig. 4-10. Through-bolt connection in shear for Example 4.1.

The load per bolt due to the moment, with a bolt spacing, $s = 4$ in., is:

$$\begin{aligned} V_h &= \frac{P_r e}{s} \\ &= \frac{P_r (2 \text{ in.})}{4 \text{ in.}} \\ &= 0.5P_r \end{aligned}$$

The resultant load per bolt is:

LRFD	ASD
$\begin{aligned} V_r &= \sqrt{V_v^2 + V_h^2} \\ &= \sqrt{(0.5P_r)^2 + (0.5P_r)^2} \\ &= 0.707P_u \\ &= 0.707(8.80 \text{ kips}) \\ &= 6.22 \text{ kips} \end{aligned}$	$\begin{aligned} V_r &= \sqrt{V_v^2 + V_h^2} \\ &= \sqrt{(0.5P_r)^2 + (0.5P_r)^2} \\ &= 0.707P_a \\ &= 0.707(6.00 \text{ kips}) \\ &= 4.24 \text{ kips} \end{aligned}$

Bolt shear strength

From AISC *Manual* Table 7-1, the bolt shear strength for a $\frac{5}{8}$ -in.-diameter Group 120 bolt in single shear with threads included in the shear plane (thread condition N) is compared to the resultant load per bolt as follows:

LRFD	ASD
$\phi r_n = 12.4 \text{ kips} > 6.22 \text{ kips} \quad \mathbf{o.k.}$	$r_n/\Omega = 8.29 \text{ kips} > 4.24 \text{ kips} \quad \mathbf{o.k.}$

Bearing strength

For the bearing strength of the HSS wall and the angle, the grip is not restrained against expansion because of the through-bolt configuration. Therefore, as required in AISC *Specification* Section J3.11b, the bearing strength should be checked using the provisions for pins in Section J7 of the AISC *Specification*.

For the HSS wall,

$$\begin{aligned} R_n &= 1.8F_y A_{pb} && (\text{Spec. Eq. J7-1}) \\ &= 1.8F_y t d_b \\ &= 1.8(50 \text{ ksi})(0.233 \text{ in.})(\frac{5}{8} \text{ in.}) \\ &= 13.1 \text{ kips} \end{aligned}$$

LRFD	ASD
$\begin{aligned} \phi &= 0.75 \\ \phi R_n &= 0.75(13.1 \text{ kips}) \\ &= 9.83 \text{ kips} > 6.22 \text{ kips} \quad \mathbf{o.k.} \end{aligned}$	$\begin{aligned} \Omega &= 2.00 \\ \frac{R_n}{\Omega} &= \frac{13.1 \text{ kips}}{2.00} \\ &= 6.55 \text{ kips} > 4.24 \text{ kips} \quad \mathbf{o.k.} \end{aligned}$

For the angle,

$$\begin{aligned}
 R_n &= 1.8F_y A_{pb} \\
 &= 1.8F_y t d_b \\
 &= 1.8(50 \text{ ksi})(\frac{1}{4} \text{ in.})(\frac{5}{8} \text{ in.}) \\
 &= 14.1 \text{ kips}
 \end{aligned}
 \tag{Spec. Eq. J7-1}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(14.1 \text{ kips})$ $= 10.6 \text{ kips} > 6.22 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{14.1 \text{ kips}}{2.00}$ $= 7.05 \text{ kips} > 4.24 \text{ kips} \quad \text{o.k.}$

Note: A complete design of this connection would also include checks of the angle gross and net section and block shear rupture strength. These checks are omitted for brevity in this example.

Example 4.2—Threaded Studs in Tension

Given:

Verify the adequacy of the connection shown in Figure 4-11. The connection consists of three $\frac{3}{4}$ -in.-diameter threaded studs welded to the HSS spaced 2 in. apart. The total dead and live loads are $P_D = 3$ kips and $P_L = 9$ kips, respectively. Assume there is no axial load in the HSS. The HSS is ASTM A500/A500M Grade C. The studs conform to AWS D1.1/D1.1M, clause 9.3.1.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

$$\begin{aligned}
 &\text{ASTM A500/A500M Grade C} \\
 &F_y = 50 \text{ ksi} \\
 &F_u = 62 \text{ ksi}
 \end{aligned}$$

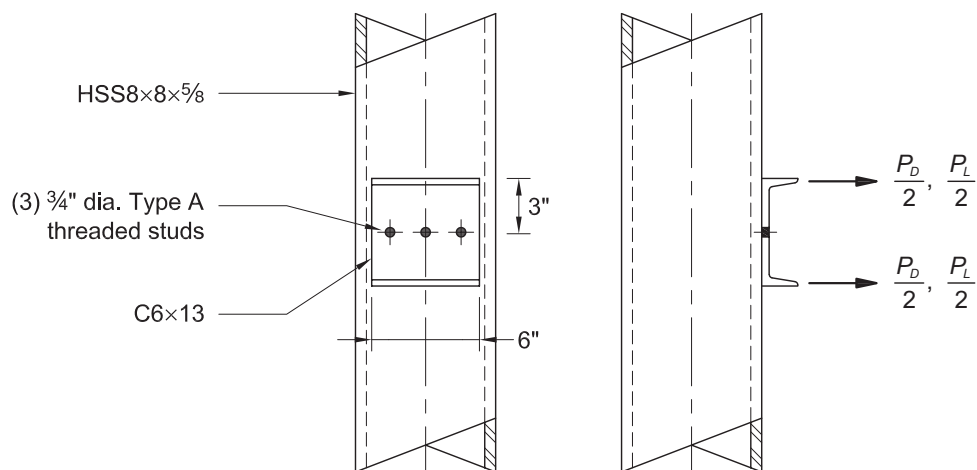


Fig. 4-11. Threaded stud-to-HSS connection in tension for Example 4.2.

From AWS D1.1/D1.1M Table 9.1, the properties of the studs are as follows:

Type A
 $F_y = 49$ ksi
 $F_u = 61$ ksi

From AISC *Manual* Table 1-12, the geometric properties are as follows:

HSS8×8× $\frac{5}{8}$
 $B = 8.00$ in.
 $t = 0.581$ in.

Required strength

From ASCE/SEI 7, Chapter 2, the required strength of the connection is:

LRFD	ASD
$P_r = P_u$ $= 1.2(3 \text{ kips}) + 1.6(9 \text{ kips})$ $= 18.0 \text{ kips}$	$P_r = P_a$ $= 3 \text{ kips} + 9 \text{ kips}$ $= 12.0 \text{ kips}$

Evaluate the possibility of prying action

For prying to occur, one of the elements in bending must be in double curvature, and the plies within the grip must bear on each other outside of the area around the bolts. Prying does not occur in this example because the channel web is in single curvature under the given loading as shown in Figure 4-12.

Tension strength of the three studs

The studs in this connection will be loaded unequally as a result of the variable transverse stiffness of the wall they are connected to. This behavior is analogous to a transverse weld. The two outer bolts will be more heavily loaded than the middle bolt because the tensile force is distributed to the stiffer portion of the HSS wall, closest to the sidewall [see Figure 2-12(b)].

Estimate the effective strength of the studs as B_e/B :

$$B_e = \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_y b t_b} \right) B_b$$

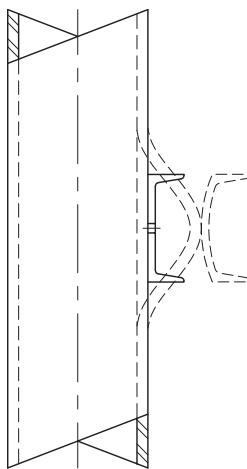


Fig. 4-12. Deformed shape of HSS and channel.

The branch width, B_b , is taken as the width across the outer edges of the studs.

$$\begin{aligned} B_b &= 2s + d \\ &= 2(2 \text{ in.}) + \frac{3}{4} \text{ in.} \\ &= 4.75 \text{ in.} \end{aligned}$$

Assume the terms in the numerator and denominator of the second term cancel each other out.

$$\begin{aligned} B_e &= \left(\frac{10}{B/t} \right) B_b \\ &= \left[\frac{10}{8.00 \text{ in.}/0.581 \text{ in.}} \right] (4.75 \text{ in.}) \\ &= 3.45 \text{ in.} \end{aligned}$$

Effective strength of the bolts:

$$\begin{aligned} \frac{B_e}{B} &= \frac{3.45 \text{ in.}}{8.00 \text{ in.}} \\ &= 0.431 \end{aligned}$$

From AISC *Specification* Section J3.7 and Table J3.2 (threaded parts), the nominal tensile strength of each stud is:

$$\begin{aligned} r_n &= F_u A_b && \text{(from Spec. Eq. J3-1)} \\ &= 0.75 F_u \frac{\pi d^2}{4} \\ &= 0.75 (61 \text{ ksi}) \frac{\pi (\frac{3}{4} \text{ in.})^2}{4} \\ &= 20.2 \text{ kips} \end{aligned}$$

For three studs, the available tensile strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 3(\phi r_n)(B_e/B)$ $= 3(0.75)(20.2 \text{ kips})(0.431)$ $= 19.6 \text{ kips} > 18.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = 3 \left(\frac{r_n}{\Omega} \right) (B_e/B)$ $= 3 \left(\frac{20.2 \text{ kips}}{2.00} \right) (0.431)$ $= 13.1 \text{ kips} > 12.0 \text{ kips} \quad \mathbf{o.k.}$

Stud pull-out strength

For pull-out of a single stud due to tension, the approach given in Section 4.1.1 can be adapted. There is no fastener head in contact with the inner surface, and pull-out is due to shear rupture through the HSS wall resulting from the tension applied through the weld around the stud perimeter. Therefore, d_p can be taken as the diameter of the stud in this case, neglecting the weld size. The pull-out strength of each stud is:

$$\begin{aligned} r_n &= 0.6 F_u \pi d_p t && \text{(from Eq. 4-1)} \\ &= 0.6 (62 \text{ ksi}) (\pi) (\frac{3}{4} \text{ in.}) (0.581 \text{ in.}) \\ &= 50.9 \text{ kips} \end{aligned}$$

For three studs, the available tensile strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 3(\phi r_n)$ $= 3(0.75)(50.9 \text{ kips})$ $= 115 \text{ kips} > 18.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = 3\left(\frac{r_n}{\Omega}\right)$ $= 3\left(\frac{50.9 \text{ kips}}{2.00}\right)$ $= 76.4 \text{ kips} > 12.0 \text{ kips} \quad \mathbf{o.k.}$

HSS wall distortion strength (plastification)

Using the approach given in Section 4.1.2, the flexural strength of the connecting face is:

$$R_n = F_y t^2 \left[\frac{2\left(\frac{l_b}{B}\right)}{\left(1 - \frac{w_b}{B}\right)} + \frac{4}{\sqrt{\left(1 - \frac{w_b}{B}\right)}} \right] Q_f \quad (4-2)$$

Referring to Figures 4-2 and 4-13:

$$B = 8.00 \text{ in.}$$

$$l_b = \frac{3}{4} \text{ in.}$$

$$t = 0.581 \text{ in.}$$

$$w_b = 4.75 \text{ in.}$$

and

$$0.4 \leq Q_f = 1.3 - 0.4 \left(\frac{U}{\beta} \right) \leq 1.0 \quad (\text{Spec. Eq. K1-4})$$

where

$$U = 0$$

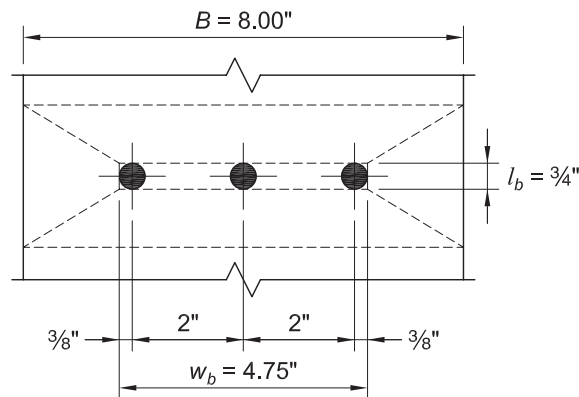


Fig. 4-13. Yield-line pattern in HSS wall.

Therefore,

$$Q_f = 1.0$$

$$R_n = (50 \text{ ksi})(0.581 \text{ in.})^2 \left[\frac{2 \left(\frac{3/4 \text{ in.}}{8.00 \text{ in.}} \right)}{\left(1 - \frac{4.75 \text{ in.}}{8.00 \text{ in.}} \right)} + \frac{4}{\sqrt{\left(1 - \frac{4.75 \text{ in.}}{8.00 \text{ in.}} \right)}} \right] (1.0)$$

$$= 114 \text{ kips}$$

The available strength of the connection for the limit state of HSS wall distortion (plastification) is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(114 \text{ kips})$ $= 114 \text{ kips} > 18.0 \text{ kips} \quad \text{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{114 \text{ kips}}{1.50}$ $= 76.0 \text{ kips} > 12.0 \text{ kips} \quad \text{o.k.}$

Note: A complete design of this connection would include a check of the channel web in flexure. This check is omitted for brevity.

Example 4.3—Bolts in Tension

Given:

Determine the connection available tensile strength, T_c , of an ASTM A992/A992M WT5×22.5 bolted to an ASTM A1065/A1065M HSS18×18× $\frac{5}{8}$ as shown in Figure 4-14. The bolts are $\frac{5}{8}$ -in.-diameter Group 120 bolts in standard holes. The problem is solved ignoring any effects due to prying action.

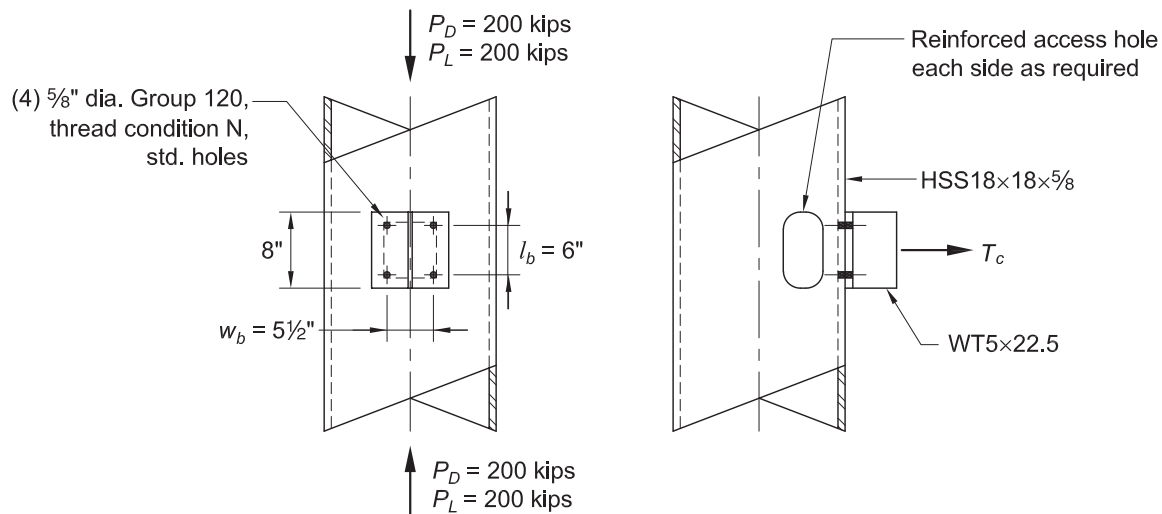


Fig. 4-14. Bolted WT-to-HSS connection in tension for Example 4.3.

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A1065/A1065M

$$F_y = 50 \text{ ksi}$$

$$F_u = 60 \text{ ksi}$$

ASTM A992/A992M

$$F_y = 50 \text{ ksi}$$

$$F_u = 65 \text{ ksi}$$

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the geometric properties of the HSS are as follows:

HSS18×18× $\frac{5}{8}$

$$B = 18.0 \text{ in.}$$

$$t = 0.625 \text{ in.}$$

$$A = 42.4 \text{ in.}^2$$

From AISC *Manual* Table 1-8, the WT geometric properties are as follows:

WT5×22.5

$$t_w = 0.350 \text{ in.}$$

$$b_f = 8.02 \text{ in.}$$

$$t_f = 0.620 \text{ in.}$$

$$g = 5\frac{1}{2} \text{ in. (workable gage)}$$

The connection has $\frac{5}{8}$ -in.-diameter ASTM A563/A563M (ASTM, 2021a) nuts inside the HSS. From AISC *Manual* Table 7-14, the width across the flats is $W = 1\frac{1}{16}$ in.

Solution:

Bolt tensile strength

From AISC *Manual* Table 7-2 (which is based on AISC *Specification* Section J3.7), the available tensile strength per bolt for the limit state of tensile rupture, without consideration of prying action, is its full strength.

LRFD	ASD
$\phi r_n = 20.7 \text{ kips}$ For the group of four bolts: $T_c = 4(\phi r_n)$ $= 4(20.7 \text{ kips})$ $= 82.8 \text{ kips}$	$\frac{r_n}{\Omega} = 13.8 \text{ kips}$ For the group of four bolts: $T_c = 4\left(\frac{r_n}{\Omega}\right)$ $= 4(13.8 \text{ kips})$ $= 55.2 \text{ kips}$

Bolt pull-out strength

For pull-out of a single bolt due to tension, the approach given in Section 4.1.1 can be adapted. In this case, d_p can be taken as the width of the nut, W . The pull-out strength of each bolt is:

$$\begin{aligned}
 r_n &= 0.6F_u\pi d_p t && \text{(from Eq. 4-1)} \\
 &= 0.6(60 \text{ ksi})(\pi)(1\frac{1}{16} \text{ in.})(0.625 \text{ in.}) \\
 &= 75.1 \text{ kips}
 \end{aligned}$$

For four bolts, the available tensile strength is:

LRFD	ASD
$\phi = 0.75$ $T_c = 4(\phi r_n)$ $= 4(0.75)(75.1 \text{ kips})$ $= 225 \text{ kips}$	$\Omega = 2.00$ $T_c = 4\left(\frac{r_n}{\Omega}\right)$ $= 4\left(\frac{75.1 \text{ kips}}{2.00}\right)$ $= 150 \text{ kips}$

HSS wall distortion strength (plastification)

Using the approach given in Section 4.1.2, the flexural strength of the connecting face is:

$$R_n = F_y t^2 \left[\frac{2\left(\frac{l_b}{B}\right)}{\left(1 - \frac{w_b}{B}\right)} + \frac{4}{\sqrt{\left(1 - \frac{w_b}{B}\right)}} \right] Q_f \quad (4-2)$$

referring to Figure 4-2:

$$B = 18.0 \text{ in.}$$

$$l_b = 6 \text{ in.}$$

$$t = 0.625 \text{ in.}$$

$$w_b = 5\frac{1}{2} \text{ in.}$$

and

$$0.4 \leq Q_f = 1.3 - 0.4 \left(\frac{U}{\beta} \right) \leq 1.0 \quad (\text{Spec. Eq. K1-4})$$

where

$$\begin{aligned} \beta &= \frac{w_b}{B} \\ &= \frac{5\frac{1}{2} \text{ in.}}{18.0 \text{ in.}} \\ &= 0.306 \end{aligned}$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0$$

When calculating the chord utilization ratio, U , the required strength of the connection is calculated from ASCE/SEI 7, Chapter 2:

LRFD	ASD
$P_{ro} = P_u$ $= 1.2P_D + 1.6P_L$ $= 1.2(200 \text{ kips}) + 1.6(200 \text{ kips})$ $= 560 \text{ kips}$ $M_{ro} = 0$ $F_c = F_y$ $= 50 \text{ ksi}$ $U = \left \frac{560 \text{ kips}}{(50 \text{ ksi})(42.4 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.264 < 1.0$ $Q_f = 1.3 - 0.4 \left(\frac{0.264}{0.306} \right)$ $= 0.955$	$P_{ro} = P_a$ $= P_D + P_L$ $= 200 \text{ kips} + 200 \text{ kips}$ $= 400 \text{ kips}$ $M_{ro} = 0$ $F_c = 0.6F_y$ $= 0.6(50 \text{ ksi})$ $= 30.0 \text{ ksi}$ $U = \left \frac{400 \text{ kips}}{(30.0 \text{ ksi})(42.4 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.314 < 1.0$ $Q_f = 1.3 - 0.4 \left(\frac{0.314}{0.306} \right)$ $= 0.890$

The available strength of the connection for the limit state of HSS wall distortion (plastification) is:

LRFD	ASD
$R_n = (50 \text{ ksi})(0.625 \text{ in.})^2$ $\times \left[\frac{2 \left(\frac{6 \text{ in.}}{18.0 \text{ in.}} \right)}{\left(1 - \frac{5\frac{1}{2} \text{ in.}}{18.0 \text{ in.}} \right)} + \frac{4}{\sqrt{\left(1 - \frac{5\frac{1}{2} \text{ in.}}{18.0 \text{ in.}} \right)}} \right] (0.955)$ $= 107 \text{ kips}$ $\phi = 1.00$ $T_c = \phi R_n$ $= 1.00(107 \text{ kips})$ $= 107 \text{ kips}$	$R_n = (50 \text{ ksi})(0.625 \text{ in.})^2$ $\times \left[\frac{2 \left(\frac{6 \text{ in.}}{18.0 \text{ in.}} \right)}{\left(1 - \frac{5\frac{1}{2} \text{ in.}}{18.0 \text{ in.}} \right)} + \frac{4}{\sqrt{\left(1 - \frac{5\frac{1}{2} \text{ in.}}{18.0 \text{ in.}} \right)}} \right] (0.890)$ $= 100 \text{ kips}$ $\Omega = 1.50$ $T_c = \frac{R_n}{\Omega}$ $= \frac{100 \text{ kips}}{1.50}$ $= 66.7 \text{ kips}$

WT flexural strength

Because the effects of prying action are neglected, the WT must remain elastic.

The distance from bolt centerline to face of tee web, b , is:

$$b = \frac{g - t_w}{2}$$

$$= \frac{5\frac{1}{2} \text{ in.} - 0.350 \text{ in.}}{2}$$

$$= 2.58 \text{ in.}$$

The distance from the face of the bolt to the face of the WT web, b' , is:

$$\begin{aligned} b' &= b - \frac{d_b}{2} \\ &= 2.58 \text{ in.} - \frac{\frac{5}{8} \text{ in.}}{2} \\ &= 2.27 \text{ in.} \end{aligned}$$

The required flexural strength of each side of the flange is:

$$\begin{aligned} M_r &= \left(\frac{T_c}{2} \right) b' \\ &= \left(\frac{T_c}{2} \right) (2.27 \text{ in.}) \end{aligned}$$

For the flange to remain elastic, the available flange moment strength at the WT flange-web junction could conservatively be limited to the flange yield moment, $F_y (Lt^2/6)$. However, the design basis for prying action in the AISC *Manual* is based on the use of the flange ultimate stress, F_u , with a corresponding flange flexural strength of $F_u (Lt^2/4)$ because it has provided better correlation with available test data than previous design methods.

The available flexural strength of each side of the flange is:

LRFD	ASD
$\phi = 0.90$ $\phi M_n = \phi F_u \left(\frac{Lt^2}{4} \right)$ $= 0.90 (65 \text{ ksi}) \left[\frac{(8.00 \text{ in.})(0.620 \text{ in.})^2}{4} \right]$ $= 45.0 \text{ kip-in.}$	$\Omega = 1.67$ $\frac{M_n}{\Omega} = \left(\frac{F_u}{\Omega} \right) \left(\frac{Lt^2}{4} \right)$ $= \left(\frac{65 \text{ ksi}}{1.67} \right) \left[\frac{(8.00 \text{ in.})(6.20 \text{ in.})^2}{4} \right]$ $= 29.9 \text{ kip-in.}$

The maximum applied tension based on flexure of the WT flange is:

LRFD	ASD
$T_c = 2 \left(\frac{\phi M_n}{b'} \right)$ $= 2 \left(\frac{45.0 \text{ kip-in.}}{2.27 \text{ in.}} \right)$ $= 39.6 \text{ kips}$	$T_c = 2 \left(\frac{M_n/\Omega}{b'} \right)$ $= 2 \left(\frac{29.9 \text{ kip-in.}}{2.27 \text{ in.}} \right)$ $= 26.3 \text{ kips}$

Thus, the flexural strength of the WT governs with the available tensile strength, $T_c = 39.6$ kips for LRFD and $T_c = 26.3$ kips for ASD.

Chapter 5

Shear Connections

A shear connection is defined as a connection where the joint between the members is not required to transfer moment and will allow for rotation associated with the member behavior under its applied loads. The most common beam shape connected to an HSS column is a wide flange. This chapter will also explore shear connections for HSS beams.

Many of the same connections that are used for wide-flange members can also be used for HSS members, with occasional variations to account for the HSS closed section shape. Connections will typically be shop-welded to the HSS column and can be bolted or welded to the beam using the same bolt types and similar welding techniques as with other structural steel connections.

These key items should be considered when designing connections to HSS members:

1. Due to the flexibility of HSS walls, the local strength of an HSS member at the connection may control the capacity of the connection. This is an important consideration when sizing the wall thickness of an HSS member.
2. Reinforcing HSS walls to locally strengthen the member is typically not an economical solution. This emphasizes the previous point that connections should be evaluated at the time the member size is determined.
3. The size of fillet welds to HSS members that can be considered in design may be controlled by the strength of the base metal. See Chapter 3.
4. For elements connected transversely to the HSS face, the force transmitted through a fillet weld is not uniform due to the variation of flexural stiffness of the HSS wall across its section width.
5. Additional limit states to consider for bolts in tension fastened to HSS members are pullout and HSS wall distortion. Both limit states are discussed in Chapter 4.

It may be questioned whether the supporting HSS column should be designed for an eccentricity due to the distance from the column centroid that a load in a simple shear connection is applied. The situation is similar to that of a shear connection to the flange of a W-shape, in which it is common practice to ignore the eccentricity produced by the load being applied at a distance of $d/2$ away from the centroid of the column. (Note: AISC *Manual* Part 10 clearly defines the conditions where connection eccentricity may be ignored—for example, a double-angle connection with standard or short-slotted holes.) The reasons we neglect these eccentricities are largely historical. Connection design evolved before we had the analysis capability to account for it, and this type

of eccentricity has not been the subject of focus for connection failures. Ioannides (1995) demonstrated that standard connections mitigate the eccentric effects in normal framing conditions. If it is decided based on engineering judgment that the eccentricities should be accounted for, it will generally be more economical to increase the HSS wall thickness than to account for the eccentricity in the connection.

5.1 APPLICABLE LIMIT STATES

Connections to HSS members are governed by similar limit states to those encountered when connecting to any other structural member and are outlined in AISC *Specification* Chapter J. Information detailing these limit states (such as bolt bearing, block shear, weld shear with eccentricity, etc.) is widely available in other publications and will not be covered in this document.

Additional limit states specific to connecting to an HSS include:

- Shear rupture strength of the HSS wall adjacent to the weld (see Chapter 3)
- Shear yielding (punching shear) of a single plate
- Yield line mechanism due to punching of the HSS wall for a stiffened seat connection to a rectangular HSS

5.1.1 Shear Yielding

Shear yielding, often referred to as punching shear, occurs when a patch of the HSS wall pulls out or punches in around the footprint of the connecting member. In the case of shear connections, this limit state frequently controls the design of single plate connections, but it should also be evaluated for the vertical stiffeners of a stiffened seat connection.

Previous research (Sherman, 1996) identified only one limit state—shear yielding (punching shear)—related to a small end rotation of the beam due to a small and insignificant eccentricity between the HSS wall and the center of gravity of the bolt line. Further, this failure only occurred when a thick shear plate was welded to a relatively thin HSS wall, thus the limitation set forth in previous editions of the AISC *Specification*. To ensure all eccentricities can be accounted for, shear yielding for single plates should be checked in accordance with AISC *Manual* Equation 10-10.

5.1.2 Yield Line Mechanism

Tests of stiffened seat connections to the webs of wide-flange columns (Sputo and Ellifritt, 1991) have shown a yield line mechanism may be an applicable limit state. There

is a correlation between the behavior of the web of a wide flange and the face of a rectangular HSS, therefore a yield line mechanism should be evaluated when connecting to an HSS using a stiffened seat.

In comparing the web of a wide flange to the face of an HSS, one can determine that both are supported along each edge for an indeterminate length. However, the web of a

wide flange is supported at each edge by a much thicker section, the flange. An HSS wall spans to an equal-thickness sidewall, therefore the boundary conditions for modeling for an HSS are assumed as pinned. The methods for designing stiffened seat connections to the web of a wide-flange column are frequently extracted for the design of the same connection to a rectangular HSS column with that one exception.

5.2 WIDE-FLANGE BEAM-TO-HSS COLUMN CONNECTIONS

The following lists the common types of connections used between wide-flange beams and HSS columns—listed, in general, from most to least economical:

- Single-plate connections (both conventional and extended configurations)
- Single-angle connections
- Double-angle connections
- WT connections
- Unstiffened seat connections
- Stiffened seat connections
- Through-plate connections

Of this list, only the through-plate connection is unique to HSS. The fillet welded option for single-angle, double-angle, and WT connections will be more economical than the flare bevel groove welded option. For round HSS, only a single plate or through plate can be employed; see Section 5.2.8.

5.2.1 Single-Plate Connections

Single-plate connections at HSS columns as shown in Figure 5-1 are made by welding both sides of a plate to the face of an HSS column, typically in the shop. The plate is then bolted to the supported beam web in the field. Single-plate connections can either be designed in conventional or extended configurations (refer to *AISC Manual Part 10* for conventional and extended configuration limitations). Depending on the distance between the bolt group and the column, the bolts may be required to be designed for the effects of the eccentricity, decreasing the capacity of the bolts. This connection was investigated in research by Sherman and Ales (1991) and Sherman (1995, 1996) that evaluated many simple framing connections between wide-flange beams and rectangular HSS columns with minimal eccentricities. Although the beam end reaction was predominantly a shear force, a small amount of end moment was always present, as is typical in practice. Over a wide range of connections tested, only one limit state was identified for the rectangular HSS column—punching shear (shear rupture) failure related to the small end rotation of the

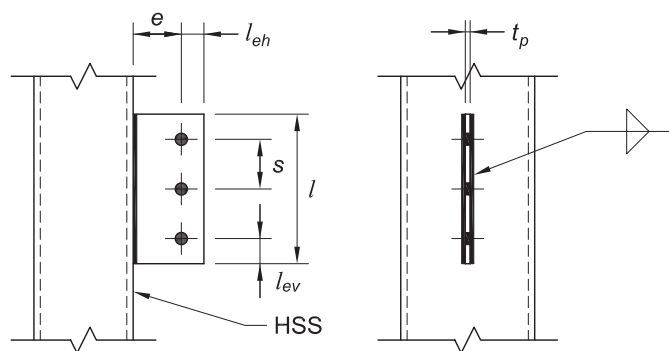


Fig. 5-1. Single-plate connection.

beam. Moreover, this only occurred when a thick shear plate was joined to a relatively thin-walled HSS. Accordingly, if the section is not slender, it is not necessary to provide a through plate connection.

Therefore, in accordance with AISC *Specification* Table B4.1a, if $b/t = (B - 3t)/t$ of the loaded face $\leq 1.40\sqrt{E/F_y}$ or 33.7 for $F_y = 50$ ksi, then a single-plate connection can be used. An extrapolation has been made for shear tabs to round HSS columns, again with the provision that the round HSS cross section is nonslender (i.e., $D/t \leq 0.11E/F_y$ or 63.8 for $F_y = 50$ ksi). Note that very few sections commonly produced have walls that are classified as slender. The connection required strength (R_u or R_a) must satisfy the following inequality pertaining to the limit state of shear yielding (punching shear):

LRFD	ASD
$\phi = 0.75$ $R_{re} \leq \frac{\phi F_u t l^2}{5}$ (from <i>Manual</i> Eq. 10-10a)	$\Omega = 2.00$ $R_{re} \leq \frac{F_u t l^2}{5\Omega}$ (from <i>Manual</i> Eq. 10-10b)

The thickness check used in the 2010 AISC *Specification* Equation K1-3, resulting from Sherman's research (Sherman, 1995) was accurate for traditional shear tab connections. However, in practice, this equation was being used, for lack of alternate guidance, for all eccentricities, including those that might fall outside of the conventional single-plate configuration. The format of the check was revised accordingly to the inequalities presented in the AISC *Manual*.

A possible additional failure mode for the single-plate connection is warping of the shear tab due to twisting of the beam, where possible. Therefore, it is recommended that long unbraced beams, such as those used in industrial structures, include lateral support in the vicinity of the connection if a single-plate connection is employed. Alternatively, a single-plate connection should be avoided.

5.2.2 Single-Angle Connections

Single-angle connections at HSS columns as shown in Figure 5-2 are made by welding an angle to the column in a vertical orientation. The angle is then bolted to one side of the supported beam web. The weld at the HSS column must be designed for the eccentricity between the weld centroid and the bolt group. Like the single-plate connection, the bolts may be required to be designed for the effects of eccentricity to account for the distance between the column face and bolt group. In order to provide adequate flexibility, the angle is shop welded to the HSS column along the toe and across the bottom of the angle, with a return at the top limited by AISC *Specification* Section J2.2b. Welding across the entire length of the top should be avoided because it would inhibit flexibility.

The size of the column and connection angle should be considered, as well as whether there are beams framing into adjacent faces of the column. Ideally, the angle leg width would be less than one-half the workable flat width of the supporting column. Because an HSS column is likely to have a smaller width than the equivalent wide-flange shape, it is possible that the angle may not fit on the width of the HSS. For a large HSS column where the connection angle fits fully on the HSS flat, there are no conflicts. However, for a smaller HSS column where the connection angle extends past the face of the column, a conflict may arise

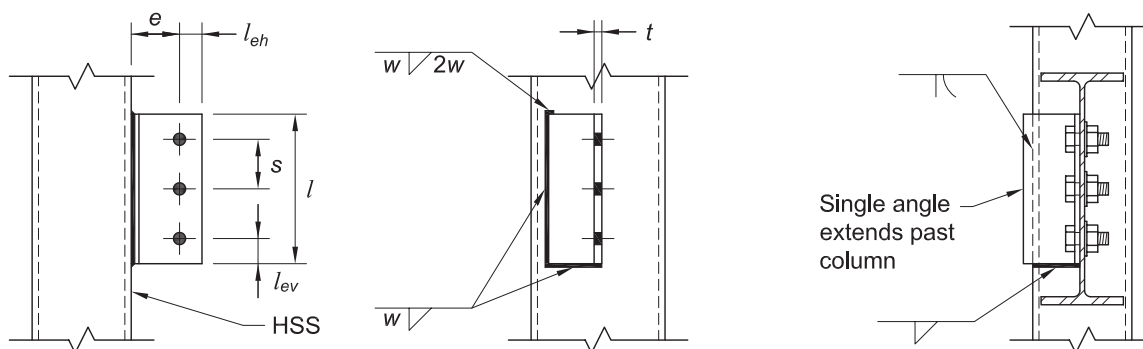


Fig. 5-2. Single-angle connection.

in the fit up of a beam framing into the adjacent column face. Additionally, the weld employed in this situation is a flare-bevel groove weld at the HSS corner (Chapter 3) and not a fillet weld.

5.2.3 Double-Angle Connections

Double-angle connections at HSS columns as shown in Figure 5-3 are similar to the single-angle connection described previously, with the exception of an angle being placed on each side of the supported beam web. Double-angle connections have the inherent advantage of placing the bolts in the beam web in double shear and doubling the length of weld to the column, both of which result in a higher connection capacity.

Because an HSS column is likely to have a smaller width than the equivalent wide-flange shape, it is possible that the angles may not fit on the width of the HSS. If the designer wishes to fillet weld the angles to the flat of the HSS, a wider section is required to account for the two outstanding angle legs, the beam width, and the fillet weld legs. Alternately, the angles may extend beyond the corner of the HSS, and a flare-bevel groove weld (and fillet reinforcement if necessary) may be employed provided the outstanding angle leg dimension is adequate.

Due to the presence of two angles that are typically shop welded to the HSS column, the bottom beam flange must be coped or stripped for the beam to be erected by lowering it between the two outstanding legs. Care should also be taken in coordinating possible conflicts with adjacent beam connections, as described in the previous section.

5.2.4 WT Connections

WT connections at HSS columns as shown in Figure 5-4 are made by welding the WT flange to the HSS face and bolting the WT web to the supported beam web. Like the double-angle connection, the WT connection also uses two weld lines to the HSS column but utilizes the bolts in single shear. This may still be beneficial in comparison to a single-plate connection, which is

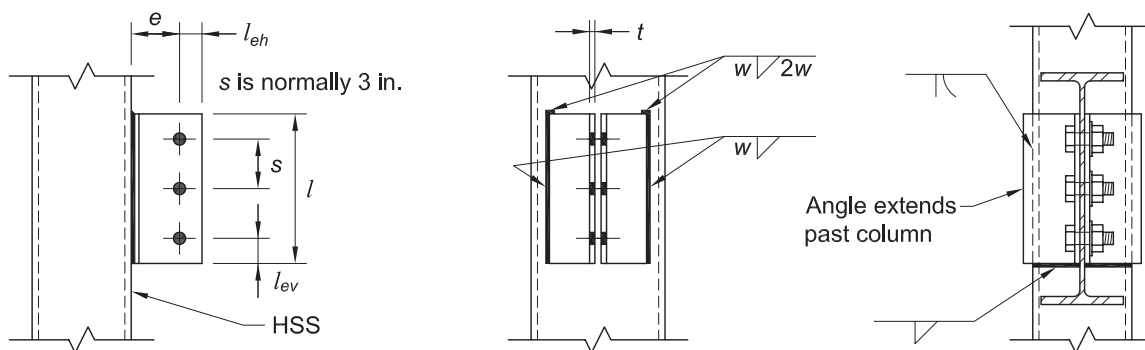


Fig. 5-3. Double-angle connection.

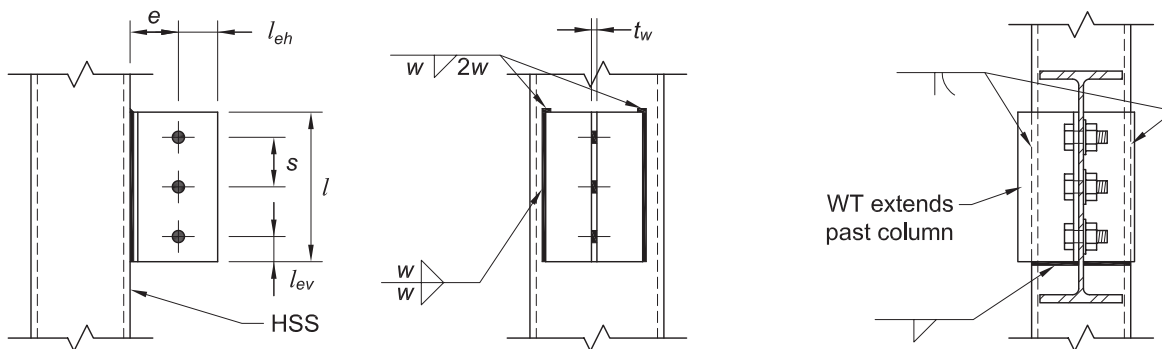


Fig. 5-4. WT connection.

typically governed by shear yielding (punching shear) of the HSS column wall. The WT flange allows for the load to be spread between the weld lines and does not lead to punching shear. Refer to AISC *Manual* Part 10 for WT geometry recommendations.

5.2.5 Unstiffened Seat Connections

Unstiffened seat connections at HSS columns as shown in Figure 5-5 are made by welding a single angle to the face of an HSS column in a horizontal orientation, and either bolting or welding the outstanding leg to the bottom flange of the supported beam. The beam shear is primarily transmitted through bearing to the bottom connection; an L4×4×¼ top angle is required, as shown in Figure 5-5, or in the optional side location for satisfactory connection performance. In order to provide adequate flexibility for the connection, only the toe of the top angle is welded to the HSS and beam flange. Unstiffened seat connections allow for the rotation associated with a simply supported beam because of the angle's inherent flexibility. When considering an unstiffened seat connection, it is imperative that the workable flat of the HSS face is large enough to accommodate both the angle itself and the weld to the column face. The supported beam must be bolted to the seat plate with two high-strength bolts to account for the prying action caused by the rotation of the connection.

5.2.6 Stiffened Seat Connections

Stiffened seat connections at HSS columns as shown in Figure 5-6 are made by welding a transverse plate to the face of the HSS column. The transverse plate is stiffened by single or double stiffeners also welded to the face of the HSS column. The beam shear is primarily transmitted through bearing to the bottom connection; an L4×4×¼ top angle is required, as shown in Figure 5-5, for an unstiffened seat connection or in the optional side location for satisfactory connection performance. In order to provide adequate flexibility for the connection, only the toes of the top angle are welded to the HSS and beam flange. The supported beam must be bolted to the seat plate with two high-strength bolts to account for the prying action caused by the rotation of the connection.

The design procedure for stiffened seat connections to HSS columns was adopted from work by Sputo and Ellifritt (1991) focused on the same connection to wide-flange webs. Like a wide-flange web, an HSS wall is supported along its two edges. However, because the side supports are the same thickness, rather than the much heavier flange side supports of a web, the equation for rotationally free edge supports (Abolitz and Warner, 1965) has been used in lieu of the fixed edge condition.

5.2.7 Through-Plate Connections

Through-plate connections as shown in Figure 5-7 are made by creating aligning slots in the front and back faces of an HSS column that permit a plate to pass through the tube and to weld to each wall. A through-plate connection is very expensive and generally not recommended for a pure shear connection. In addition, there are constructability considerations in erecting beams between shop-attached through plates, so it is common to field weld the through plates, which further contributes to the cost. The

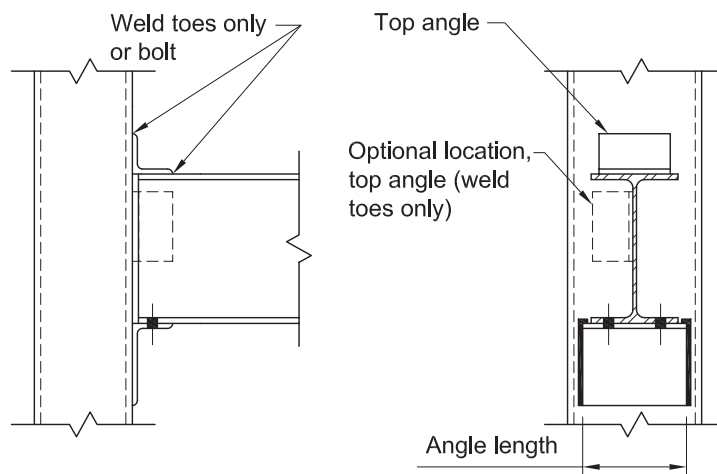


Fig. 5-5. Unstiffened seated connection.

first three connections listed have approximately similar costs and complexity and are a much more economical alternative to a through-plate connection.

While it is not recommended to specify a through-plate connection as a typical wide-flange beam to HSS connection, there are instances where it is useful. For example, when transferring axial loads from one side of the column to the other, like for a drag strut, a through-plate connection can be a very efficient mechanism for the load transfer. If the HSS column is found to be inadequate to resist shear yielding (punching shear) or the wall of the HSS is slender, a through-plate connection is an option, although it is recommended that other connection configurations, such as double angles, be evaluated before a through plate is employed. One further application of through plates arises with fire resistance. If the column is filled with concrete after fabrication of the through-plate connection to form a composite member, and there is no external fireproofing on the column, then in the event of a fire, the HSS steel yields, but the through plate will transfer the shear reaction to the concrete core.

Because of the costs associated with this type of connection, it will likely be more efficient to increase the thickness of the HSS wall to enable a connection to the HSS column face. Designers should also bear in mind that a part of the through plate protrudes beyond the far side of the face of the column and that may affect other connections or architectural features. Through-plate connections should not be specified orthogonal to each other at the same column. This requires the fabricator to cut the column,

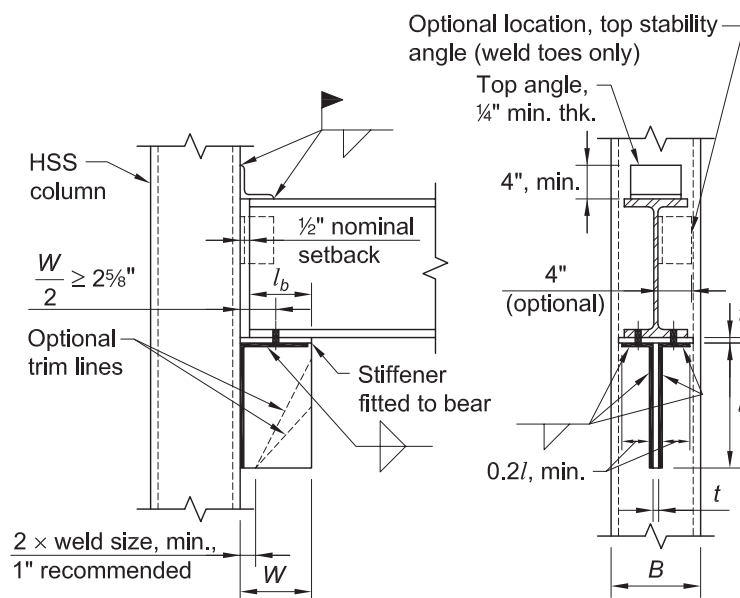


Fig. 5-6. Stiffened seat connection to HSS column.

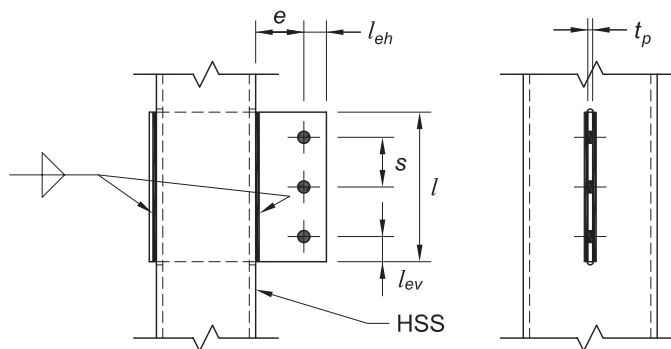


Fig. 5-7. Through-plate connection.

place the plates and weld them together, and then reweld the column pieces together with a CJP groove weld. This results in an extremely expensive and avoidable connection.

Through-plate connections have the same limit states as single-plate connections, and design aids may be used for sizing the plate thickness and number and size of the bolts. The welds, however, are subject to direct shear and may not be required to be as large as those for a single-plate connection. For equilibrium of the forces in Figure 5-8, the shear in the welds on the connection face is $V_f = R_r(H + e_w)/H$ and should not exceed the strength of the pair of welds on the opposite side of the connection. Conservatively, the welds on the face away from the connection may be the same size.

If there are connections on each side of the column, the plate will go into bending between the walls of the HSS. This should also be considered when using a large HSS where the distance between the walls is significant. The plate should be evaluated for lateral-torsional buckling, but increasing its thickness to prevent buckling will reduce the rotational flexibility assumed for the connection. Therefore, it must be recognized that the plate may buckle, and the moment will be shared with the HSS wall in a complex manner. However, if the HSS is satisfactorily checked for shear yielding (punching shear) of a single plate, the lateral-torsional buckling limit state is not a critical concern involving loss of strength.

5.2.8 Shear Connections to Round HSS

Of the connections listed, only a single plate and a through plate can be reasonably used for a shear connection to a round HSS due to the curvature of the face to which the connection is being made. A single plate can be fillet welded all around to a round HSS column face, as shown in Figure 5-9(a). As with a rectangular HSS, a single plate is permitted for round HSS columns that are not slender—that is, $D/t \leq 0.11E/F_y$. Aside from this provision, the design procedure is the same as what is described in Example 5.1 for a rectangular HSS column.

A stiffened seat connection may also be specified, although the seat plate must be profiled to match the curvature of the HSS. This can be costly, so it is recommended that a stiffened seat-to-round HSS connection is only specified when a single plate does not have adequate capacity. Alternatively, an additional piece may be added locally where the connection occurs to provide a connection point. For example, as illustrated in Figure 5-9(b), a channel may be welded to the face of the section to provide a flat connection surface.

5.3 HIGH SHEAR LOAD CONNECTIONS

Higher shear loads are common in girder connections-to-HSS columns as well as braced frame connections. These loads may be high enough that a single-plate connection cannot meet the shear yielding (punching shear) limit state requirement for the HSS column. Or similarly, the requirements for stiffeners in a stiffened seat.

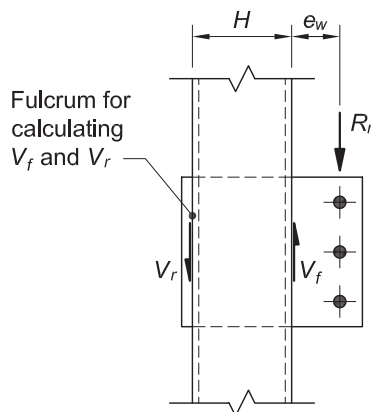


Fig. 5-8. Shear forces in a through-plate connection.

In these cases, one might evaluate the following options:

- Detail the beam to frame over the column and transfer the high shear to the column load through bearing. Bracing the column should also be considered.
- Utilize double-angle connections if the column dimensions and adjacent framing allows.
- Utilize a through-plate connection.
- Look for ways to spread out the load and deliver it to the HSS column sidewalls, thus precluding the shear yielding (punching shear) limit state.
- Reinforce the column face with a plate thicker than the HSS wall. Extend the plate a minimum of the full width of the flat of the HSS. Do not consider the plate to act in conjunction with the HSS wall.

5.4 HSS BEAM-TO-HSS COLUMN CONNECTIONS

Although most frequently seen as axial load-carrying members, HSS are occasionally used as beams when the application warrants it. A perimeter beam may be an HSS to solve issues with torsional loading due to the exterior wall. A beam may be an HSS for architectural reasons, such as being exposed to view or the facility being “clean,” like a food processing plant. When an HSS is used as a beam, the two vertical sidewalls act as the web elements that carry the shear forces. Because these two “webs” are on the outside of the section, as opposed to a wide-flange web centered on the section, many of the commonly used beam connections are not applicable. Instead, options for connecting HSS beams to supports include end plates, double angles, unstiffened seats, stiffened seats, double-plate connections, and a plate in a slot at the end of the HSS beam. Double-angle and double-plate connections can be field welded or use through bolts or threaded rods that pass through the cavity of the tube (see discussion of through bolts in Chapter 4).

For stiffened seat connections, it must be determined if a single stiffener centered under the HSS beam is acceptable or if two stiffeners aligned with the walls of the HSS are required.

It is common when using HSS beams to need to transfer forces other than, or in addition to, vertical shear. These additional forces may include horizontal shear, torsion, axial forces, and sometimes moments. Connection designers should be vigilant in recognizing situations where the connection material required to transfer all of the forces causes the connection to have more fixity than was assumed for combined end reactions and then requires the beam and support designs to be reevaluated.

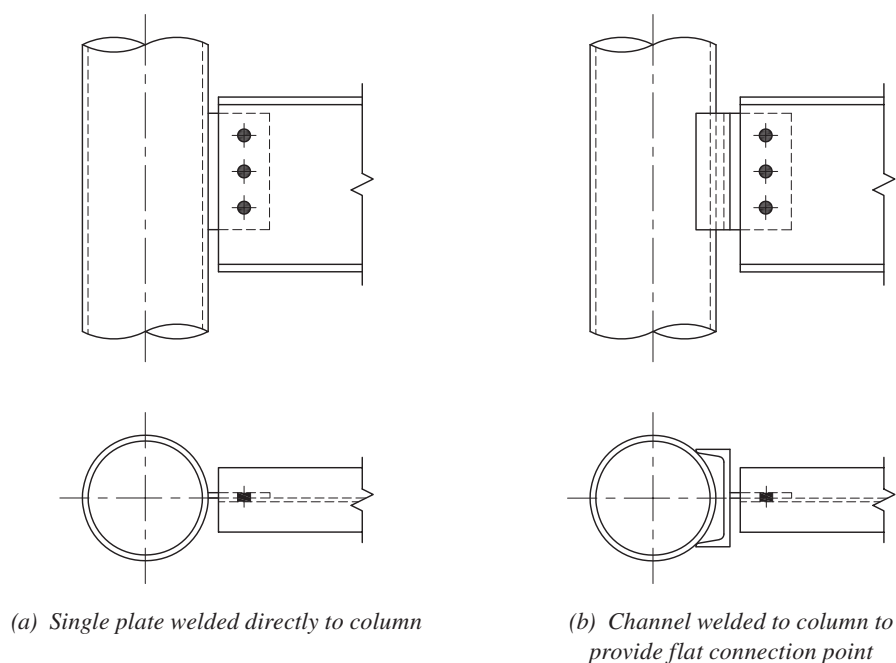


Fig. 5-9. Shear connections to round HSS.

5.5 SHEAR CONNECTION DESIGN EXAMPLES

Example 5.1—Single Plate-to-HSS Connection

Given:

Design a single-plate shear connection for the geometry and loading as shown in Figure 5-10. Use $\frac{3}{4}$ -in.-diameter Group 120 bolts with threads not excluded from the shear plane (thread condition N). Use standard (STD) holes in the beam web and horizontal short-slotted (SSLT) holes in the plate. The HSS column is ASTM A500/A500M Grade C, the beam is ASTM A992/A992M, and the plate is ASTM A572/A572M Grade 50. Use 70 ksi weld electrodes.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

Beam

ASTM A992/A992M

$F_y = 50$ ksi

$F_u = 65$ ksi

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50$ ksi

$F_{up} = 65$ ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

Beam

W21 $\times$ 44

$t_w = 0.350$ in.

$T = 18\frac{3}{8}$ in.

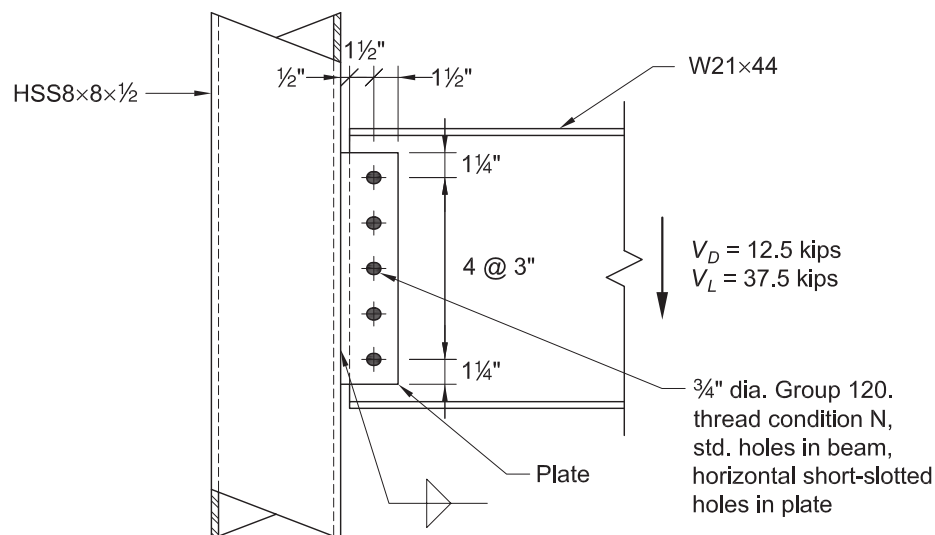


Fig. 5-10. Single-plate connection for Example 5.1.

Column
HSS8×8×½
 $B = 8.00$ in.
 $t = 0.465$ in.
 $b/t = 14.2$

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$V_u = 1.2V_D + 1.6V_L$ $= 1.2(12.5 \text{ kips}) + 1.6(37.5 \text{ kips})$ $= 75.0 \text{ kips}$	$V_a = V_D + V_L$ $= 12.5 \text{ kips} + 37.5 \text{ kips}$ $= 50.0 \text{ kips}$

Consider rotational ductility

To ensure rotational ductility in a single-plate connection, AISC *Specification* Section J1.2 requires the geometry and thickness of the plate to be configured such that the plate will yield, the bolt group will rotate, or the bolt holes will elongate at failure prior to the failure of the welds or bolts. Limitations are outlined in AISC *Manual* Part 10.

1. Only one vertical row of bolts is permitted **o.k.**
2. $a \leq 3\frac{1}{2}$ in. **o.k.**
3. STD or SSLT holes transverse to reaction **o.k.**
4. $l_{ev} = 1\frac{1}{4}$ in. > 1 in. per AISC *Specification* Table J3.4 **o.k.**
 $l_{eh} = 1\frac{1}{2}$ in. $\geq 2d_b = 2(\frac{3}{4} \text{ in.}) = 1\frac{1}{2}$ in. **o.k.**
5. Maximum thickness requirement will be checked at the end of the calculation.

Therefore, this connection meets the limitations for a conventional configuration single-plate.

Bolt strength

The available bolt shear strength is checked in accordance with AISC *Specification* Section J3.7 assuming the eccentricity, e , shown in AISC *Manual* Table 10-9 and the effective number of bolts from AISC *Manual* Table 7-6.

The available shear strength of a $\frac{3}{4}$ -in.-diameter Group 120 bolt (thread condition N) is determined from AISC *Manual* Table 7-1:

LRFD	ASD
$\phi r_n = 17.9 \text{ kips/bolt}$ Number of bolts $n = \frac{V_u}{\phi r_n}$ $= \frac{75.0 \text{ kips}}{17.9 \text{ kips/bolt}}$ $= 4.19 \text{ bolts}$	$r_n/\Omega = 11.9 \text{ kips/bolt}$ Number of bolts $n = \frac{V_a}{r_n/\Omega}$ $= \frac{50.0 \text{ kips}}{11.9 \text{ kips/bolt}}$ $= 4.20 \text{ bolts}$

Try five bolts.

From AISC *Manual* Table 10-9, with $n = 5$ and SSLT holes:

$$\begin{aligned}
 e &= a/2 \\
 &= 2 \text{ in.}/2 \\
 &= 1.00 \text{ in.}
 \end{aligned}$$

From AISC *Manual* Table 7-6 (Angle = 0°), with $s = 3$ in., $e_x = 1.00$ in., and $n = 5$ bolts:

$$C = 4.77$$

LRFD	ASD
$\phi R_n = C \phi r_n$ $= 4.77(17.9 \text{ kips/bolt})$ $= 85.4 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$R_n/\Omega = C(r_n/\Omega)$ $= 4.77(11.9 \text{ kips/bolt})$ $= 56.8 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Bolt bearing and tearout of plate

Because the connection geometry meets the requirements of a conventional configuration, plate bearing and tearout checks are calculated in accordance with AISC *Specification* Section J3.11 assuming the reaction is applied concentrically.

Try a $\frac{3}{8}$ in. single plate.

Using AISC *Manual* Tables 7-4 and 7-5, the available bearing and tearout strength of the plate is determined as follows:

LRFD	ASD
For the interior bolts $s = 3 \text{ in.}$ $\phi r_{n,int} = 87.8 \text{ kips/bolt/in.}$ For the edge bolts $l_{ev} = 1\frac{1}{4} \text{ in.}$ $\phi r_{n,edge} = 49.4 \text{ kips/bolt/in.}$ For the 5 bolts $\phi R_n = \left[(n-1)\phi r_{n,int} + \phi r_{n,edge} \right] t_p$ $= \left[(4 \text{ bolts})(87.8 \text{ kips/bolt/in.}) + (1 \text{ bolt})(49.4 \text{ kips/bolt/in.}) \right] \left(\frac{3}{8} \text{ in.} \right)$ $= 150 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	For the interior bolts $s = 3 \text{ in.}$ $r_{n,int}/\Omega = 58.5 \text{ kips/bolt/in.}$ For the edge bolts $l_{ev} = 1\frac{1}{4} \text{ in.}$ $r_{n,edge}/\Omega = 32.9 \text{ kips/bolt/in.}$ For the 5 bolts $\frac{R_n}{\Omega} = \left[(n-1)\frac{r_{n,int}}{\Omega} + \frac{r_{n,edge}}{\Omega} \right] t_p$ $= \left[(4 \text{ bolts})(58.5 \text{ kips/bolt/in.}) + (1 \text{ bolt})(32.9 \text{ kips/bolt/in.}) \right] \left(\frac{3}{8} \text{ in.} \right)$ $= 100 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Bolt bearing and tearout of beam web

Using AISC *Manual* Table 7-4, the available bearing strength is determined as follows:

LRFD	ASD
$\phi R_n = n \phi r_{n,int} t_w$ $= (5 \text{ bolts})(87.8 \text{ kips/bolt/in.})(0.350 \text{ in.})$ $= 154 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$\frac{R_n}{\Omega} = n \left(\frac{r_{n,int}}{\Omega} \right) t_w$ $= (5 \text{ bolts})(58.5 \text{ kips/bolt/in.})(0.350 \text{ in.})$ $= 102 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Shear yielding of the plate

The nominal shear yielding strength of the plate is determined using AISC *Specification* Section J4.2(a).

$$\begin{aligned}l &= 2l_{ev} + (n-1)s \\ &= 2(1\frac{1}{4} \text{ in.}) + (5-1)(3 \text{ in.}) \\ &= 14.5 \text{ in.}\end{aligned}$$

$$\begin{aligned}A_{gv} &= lt_p \\ &= (14.5 \text{ in.})(\frac{3}{8} \text{ in.}) \\ &= 5.44 \text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n &= 0.60F_yA_{gv} && (\text{Spec. Eq. J4-3}) \\ &= 0.60(50 \text{ ksi})(5.44 \text{ in.}^2) \\ &= 163 \text{ kips}\end{aligned}$$

The available shear yielding strength of the plate is as follows:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(163 \text{ kips})$ $= 163 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{163 \text{ kips}}{1.50}$ $= 109 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Shear rupture of the plate

The nominal shear rupture strength of the plate is determined using AISC *Specification* Section J4.2(b). From AISC *Specification* Table J3.3, the nominal hole diameter for a $\frac{3}{4}$ in. bolt is $d_h = \frac{13}{16}$ in. From AISC *Specification* Section B4.3b, an additional $\frac{1}{16}$ in. is added to account for damage around the hole caused by drilling, punching, or thermal cutting.

$$\begin{aligned}l_{nv} &= l - n(d_h + \frac{1}{16} \text{ in.}) \\ &= 14.5 \text{ in.} - 5(\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.}) \\ &= 10.1 \text{ in.}\end{aligned}$$

$$\begin{aligned}A_{nv} &= l_{nv}t_p \\ &= (10.1 \text{ in.})(\frac{3}{8} \text{ in.}) \\ &= 3.79 \text{ in.}\end{aligned}$$

$$\begin{aligned}R_n &= 0.60F_uA_{nv} && (\text{Spec. Eq. J4-4}) \\ &= 0.60(65 \text{ ksi})(3.79 \text{ in.}^2) \\ &= 148 \text{ kips}\end{aligned}$$

The available shear rupture strength of the plate is as follows:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(148 \text{ kips})$ $= 111 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{148 \text{ kips}}{2.00}$ $= 74.0 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Block shear in plate

From AISC *Specification* Section J4.3, the nominal strength of the plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= [l_{ev} + (n-1)s]t_p \\ &= [1\frac{1}{4} \text{ in.} + 4(3 \text{ in.})](\frac{3}{8} \text{ in.}) \\ &= 4.97 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= [l_{eh} - 0.5(d_h + \frac{1}{16} \text{ in.})]t_p \\ &= [1\frac{1}{2} \text{ in.} - 0.5(\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})](\frac{3}{8} \text{ in.}) \\ &= 0.398 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (n-0.5)(d_h + \frac{1}{16} \text{ in.})t_p \\ &= 4.97 \text{ in.}^2 - 4.5(\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.}) (\frac{3}{8} \text{ in.}) \\ &= 3.49 \text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0 \text{ because tension is uniform}$$

Evaluating AISC *Specification* Equation J4-5:

$$\begin{aligned} R_n &= 0.60F_{up}A_{nv} + U_{bs}F_{up}A_{nt} \leq 0.60F_{yp}A_{gv} + U_{bs}F_{up}A_{nt} \\ &= 0.60(65 \text{ ksi})(3.49 \text{ in.}^2) + 1.0(65 \text{ ksi})(0.398 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(4.97 \text{ in.}^2) + 1.0(65 \text{ ksi})(0.398 \text{ in.}^2) \\ &= 162 \text{ kips} < 175 \text{ kips} \\ &= 162 \text{ kips} \end{aligned}$$

The available strength of the plate for the limit state of block shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(162 \text{ kips})$ $= 122 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{162 \text{ kips}}{2.00}$ $= 81.0 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Weld to HSS

To develop the strength of the plate, size the welds between the HSS column and the plate as $\frac{5}{8}t_p$ (AISC *Manual* Part 10, Single-Plate Connections)

$$\begin{aligned} w &= \frac{5}{8}t_p \\ &= \frac{5}{8}(\frac{3}{8} \text{ in.}) \\ &= 0.234 \text{ in.} \end{aligned}$$

Use $\frac{1}{4}$ in. fillet welds on each side of the plate.

From Table 3-3, the HSS minimum thickness for a $\frac{1}{4}$ in. fillet weld is 0.200 in. **o.k.**

Column slenderness

Per AISC *Manual* Part 10, single-plate connections may be used as long as the HSS column is nonslender under axial load.

$$b/t = 14.2$$

From AISC *Specification* Table B4.1a, Case 6:

$$\begin{aligned}\lambda_r &= 1.40 \sqrt{\frac{E}{F_y}} \\ &= 1.40 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 33.7 > 14.2 \quad \mathbf{o.k.}\end{aligned}$$

Shear yielding (punching shear) in HSS column

Per AISC *Manual* Part 10, punching shear is evaluated by:

LRFD	ASD
$R_r e \leq \frac{\phi F_u t l^2}{5} \quad (\text{from } \textit{Manual} \text{ Eq. 10-10a})$ $e = 2 \text{ in.}$ $\phi = 0.75$ $R_u (2 \text{ in.}) \leq \frac{0.75(62 \text{ ksi})(0.465 \text{ in.})(14.5 \text{ in.})^2}{5}$ $= 455 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$R_r e \leq \frac{F_u t l^2}{5\Omega} \quad (\text{from } \textit{Manual} \text{ Eq. 10-10b})$ $e = 2 \text{ in.}$ $\Omega = 2.00$ $R_u (2 \text{ in.}) \leq \frac{(62 \text{ ksi})(0.465 \text{ in.})(14.5 \text{ in.})^2}{5(2.00)}$ $= 303 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Recommended plate length

From AISC *Manual* Part 10, the minimum recommended connection length is one-half of the T -dimension of the beam:

$$\begin{aligned}l_{min} &= \frac{T}{2} \\ &= \frac{18\frac{3}{8} \text{ in.}}{2} \\ &= 9.19 \text{ in.} < 14.5 \text{ in.} \quad \mathbf{o.k.}\end{aligned}$$
$$\begin{aligned}l_{max} &= T \\ &= 18\frac{3}{8} \text{ in.} > 14.5 \text{ in.} \quad \mathbf{o.k.}\end{aligned}$$

Maximum thickness requirement

From AISC *Manual* Table 10-9, for a conventional single-plate connection with five bolts using short-slotted bolts, there is no maximum t_p or t_w . Therefore, this connection is acceptable.

Example 5.2—Double Angle-to-Narrow HSS Connection

Given:

Design a double-angle shear connection for the geometry and loading as shown in Figure 5-11. Use $\frac{7}{8}$ -in.-diameter Group 120 bolts with threads not excluded from the shear plane (thread condition N). Use standard holes in the beam web and horizontal short-slotted holes in the angles. The HSS column is ASTM A500/A500M Grade C, the beam is ASTM A992/A992M, and the angles are ASTM A572/A572M Grade 50. Use 70 ksi weld electrodes.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

Beam

ASTM A992/A992M

$F_y = 50$ ksi

$F_u = 65$ ksi

Angles

ASTM A572/A572M Grade 50

$F_y = 50$ ksi

$F_u = 65$ ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

Beam

W24×55

$t_w = 0.395$ in.

$T = 20\frac{3}{4}$ in.

Column

HSS6×6× $\frac{3}{8}$

$t = 0.349$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2V_D + 1.6V_L$ $= 1.2(20 \text{ kips}) + 1.6(60 \text{ kips})$ $= 120 \text{ kips}$	$R_a = V_D + V_L$ $= 20 \text{ kips} + 60 \text{ kips}$ $= 80.0 \text{ kips}$

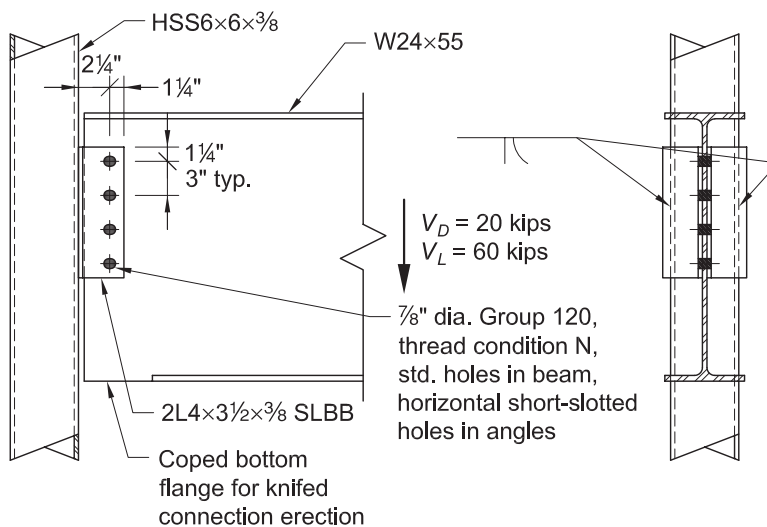


Fig. 5-11. Double-angle connection for Example 5.2.

Per AISC *Manual* Part 10, the eccentricity on the beam side of the connection may be neglected because it is anticipated that the connection will have a single row of bolts, and the dimension from the face of the column to the centerline of the row of bolts, a , is less than 3 in.

Bolt strength

The available bolt shear strength is checked in accordance with AISC *Specification* Section J3.7.

From AISC *Manual* Table 7-1, for a $\frac{7}{8}$ -in.-diameter bolt (thread condition N) in double shear, the number of required bolts in double shear is:

LRFD	ASD
$\phi r_n = 48.7 \text{ kips/bolt}$ Number of bolts $n = \frac{R_u}{\phi r_n}$ $= \frac{120 \text{ kips}}{48.7 \text{ kips/bolt}}$ $= 2.46 \text{ bolts}$	$r_n/\Omega = 32.5 \text{ kips/bolt}$ Number of bolts $n = \frac{R_a}{r_n/\Omega}$ $= \frac{80.0 \text{ kips}}{32.5 \text{ kips/bolt}}$ $= 2.46 \text{ bolts}$

Use four bolts to meet the minimum recommended connection length requirements ($T/2$).

Bolt bearing and tearout on angle

The available bearing and tearout strength of the angles are determined in accordance with AISC *Specification* Section J3.11.

Try $\frac{3}{8}$ in. angles.

Using AISC *Manual* Tables 7-4 and 7-5, the available bearing and tearout strength is determined as follows:

LRFD	ASD
For the interior bolts $s = 3 \text{ in.}$ $\phi r_{n,int} = 102 \text{ kips/bolt/in.}$ For the edge bolt $l_{ev} = 1\frac{1}{4} \text{ in.}$ $\phi r_{n,edge} = 45.7 \text{ kips/bolt/in.}$ For the 4 bolts $\phi R_n = \left[(n-1)\phi r_{n,int} + \phi r_{n,edge} \right] 2t$ $= \left[(3 \text{ bolts})(102 \text{ kips/bolt/in.}) + (1 \text{ bolt})(45.7 \text{ kips/bolt/in.}) \right] (2 \text{ angles})(\frac{3}{8} \text{ in.})$ $= 264 \text{ kips} > 120 \text{ kips} \quad \mathbf{o.k.}$	For the interior bolts $s = 3 \text{ in.}$ $r_{n,int}/\Omega = 68.3 \text{ kips/bolt/in.}$ For the edge bolt $l_{ev} = 1\frac{1}{4} \text{ in.}$ $r_{n,edge}/\Omega = 30.5 \text{ kips/bolt/in.}$ For the 4 bolts $\frac{R_n}{\Omega} = \left[(n-1)\frac{r_{n,int}}{\Omega} + \frac{r_{n,edge}}{\Omega} \right] 2t$ $= \left[(3 \text{ bolts})(68.3 \text{ kips/bolt/in.}) + (1 \text{ bolt})(30.5 \text{ kips/bolt/in.}) \right] (2 \text{ angles})(\frac{3}{8} \text{ in.})$ $= 177 \text{ kips} > 80.0 \text{ kips} \quad \mathbf{o.k.}$

Bolt bearing and tearout of beam web

Using AISC *Manual* Table 7-4, the available bearing strength of the beam web is determined as follows:

LRFD	ASD
For the interior bolts $s = 3 \text{ in.}$ $\phi r_{n,int} = 102 \text{ kips/bolt/in.}$ For the 4 bolts $\phi R_n = n \phi r_{n,int} t_w$ $= (4 \text{ bolts})(102 \text{ kips/bolt/in.})(0.395 \text{ in.})$ $= 161 \text{ kips} > 120 \text{ kips} \quad \text{o.k.}$	For the interior bolts $s = 3 \text{ in.}$ $r_{n,int}/\Omega = 68.3 \text{ kips/bolt/in.}$ For the 4 bolts $\frac{R_n}{\Omega} = n \left(\frac{r_{n,int}}{\Omega} \right) t_w$ $= (4 \text{ bolts})(68.3 \text{ kips/bolt/in.})(0.395 \text{ in.})$ $= 108 \text{ kips} > 80.0 \text{ kips} \quad \text{o.k.}$

Shear yielding of the angles

The nominal shear yielding strength of the angles is determined using AISC *Specification* Section J4.2(a).

$$\begin{aligned}
 l &= 2l_{ev} + (n-1)s \\
 &= 2(1\frac{1}{4} \text{ in.}) + (4-1)(3 \text{ in.}) \\
 &= 11.5 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 A_{gv} &= lt \\
 &= (11.5 \text{ in.})(\frac{3}{8} \text{ in.}) \\
 &= 4.31 \text{ in.}^2
 \end{aligned}$$

$$\begin{aligned}
 R_n &= 0.60F_y A_{gv} && (\text{Spec. Eq. J4-3}) \\
 &= 0.60(50 \text{ ksi})(4.31 \text{ in.}^2) \\
 &= 129 \text{ kips}
 \end{aligned}$$

The available shear yielding strength of the angles is determined as follows:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(129 \text{ kips})(2 \text{ angles})$ $= 258 \text{ kips} > 120 \text{ kips} \quad \text{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{129 \text{ kips}}{1.50} (2 \text{ angles})$ $= 172 \text{ kips} > 80.0 \text{ kips} \quad \text{o.k.}$

Shear rupture of the angles

The nominal shear rupture strength of the angles is determined using AISC *Specification* Section J4.2(b). From AISC *Specification* Table J3.3, the nominal hole diameter for a $\frac{7}{8}$ in. bolt is $d_h = \frac{15}{16}$ in. From AISC *Specification* Section B4.3b, an additional $\frac{1}{16}$ in. is added to account for damage around the hole caused by drilling, punching, or thermal cutting.

$$\begin{aligned}
 l_{nv} &= l - n(d_h + \frac{1}{16} \text{ in.}) \\
 &= 11.5 \text{ in.} - 4(\frac{15}{16} \text{ in.} + \frac{1}{16} \text{ in.}) \\
 &= 7.50 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 A_{nv} &= l_{nv} t \\
 &= (7.50 \text{ in.})(\frac{3}{8} \text{ in.}) \\
 &= 2.81 \text{ in.}^2
 \end{aligned}$$

$$\begin{aligned}
 R_n &= 0.60F_u A_{nv} & (\text{Spec Eq. J4-4}) \\
 &= 0.60(65 \text{ ksi})(2.81 \text{ in.}^2) \\
 &= 110 \text{ kips}
 \end{aligned}$$

The available shear rupture strength of the angles is determined as follows:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(110 \text{ kips})(2 \text{ angles})$ $= 165 \text{ kips} > 120 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{110 \text{ kips}}{2.00} (2 \text{ angles})$ $110 \text{ kips} > 80.0 \text{ kips} \quad \mathbf{o.k.}$

Block shear in angles

From AISC *Specification* Section J4.3, the nominal strength of the angles based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned}
 A_{gv} &= [l_{ev} + (n-1)s]t \\
 &= [1\frac{1}{4} \text{ in.} + 3(3 \text{ in.})](\frac{3}{8} \text{ in.}) \\
 &= 3.84 \text{ in.}^2
 \end{aligned}$$

$$\begin{aligned}
 A_{nt} &= [l_{eh} - 0.5(d_h + \frac{1}{16} \text{ in.})]t \\
 &= [1\frac{1}{4} \text{ in.} - 0.5(1\frac{5}{16} \text{ in.} + \frac{1}{16} \text{ in.})](\frac{3}{8} \text{ in.}) \\
 &= 0.281 \text{ in.}^2
 \end{aligned}$$

$$\begin{aligned}
 A_{nv} &= A_{gv} - (n-0.5)(d_h + \frac{1}{16} \text{ in.})t \\
 &= 3.84 \text{ in.}^2 - 3.5(1\frac{5}{16} \text{ in.} + \frac{1}{16} \text{ in.})(\frac{3}{8} \text{ in.}) \\
 &= 2.53 \text{ in.}^2
 \end{aligned}$$

$$U_{bs} = 1.0 \text{ because tension is uniform}$$

Evaluating AISC *Specification* Equation J4-5:

$$\begin{aligned}
 R_n &= 0.60F_{up}A_{nv} + U_{bs}F_{up}A_{nt} \leq 0.60F_{yp}A_{gv} + U_{bs}F_{up}A_{nt} \\
 &= 0.60(65 \text{ ksi})(2.53 \text{ in.}^2) + 1.0(65 \text{ ksi})(0.281 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(3.84 \text{ in.}^2) + 1.0(65 \text{ ksi})(0.281 \text{ in.}^2) \\
 &= 117 \text{ kips} < 133 \text{ kips} \\
 &= 117 \text{ kips}
 \end{aligned}$$

The available strength of the angles for the limit state of block shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(117 \text{ kips})(2 \text{ angles})$ $= 176 \text{ kips} > 120 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{117 \text{ kips}}{2.00} (2 \text{ angles})$ $= 117 \text{ kips} > 80.0 \text{ kips} \quad \mathbf{o.k.}$

Weld to HSS

Because the width of the double angles is larger than the width of the HSS, a flare-bevel groove weld is required at the HSS corners. The outside radius is:

$$\begin{aligned} R &= 2t \\ &= 2(0.349 \text{ in.}) \\ &= 0.698 \text{ in.} \end{aligned}$$

From AISC *Specification* Table J2.2, conservatively assuming a FCAW-S welding process, the effective throat thickness of the flare-bevel groove weld is:

$$\begin{aligned} E &= \frac{5}{16}R \\ &= \frac{5}{16}(0.698 \text{ in.}) \\ &= 0.218 \text{ in.} \end{aligned}$$

This effective throat thickness will be used for subsequent calculations; however, for detailing, a ¼ in. weld should be specified.

Using AISC *Specification* Table J2.3, the minimum effective throat thickness of the flare-bevel groove weld based on the thickness of the HSS column is ⅜ in, which is less than the effective throat discussed previously.

The equivalent fillet weld with the same throat dimensions is:

$$\begin{aligned} \left(\frac{D}{16}\right)\left(\frac{1}{\sqrt{2}}\right) &= 0.218 \text{ in.} \\ D &= 16\sqrt{2}(0.218 \text{ in.}) \\ &= 4.93 \text{ sixteenths of an inch} \end{aligned}$$

Nominal weld shear strength

$$\begin{aligned} e_x &= al \\ a &= \frac{e_x}{l} \\ &= \frac{2\frac{1}{4} \text{ in.}}{11.5 \text{ in.}} \\ &= 0.196 \end{aligned}$$

Interpolating from AISC *Manual* Table 8-4, with $a = 0.196$ and $k = 0$:

$$C = 3.52$$

From AISC *Manual* Table 8-3, the electrode strength coefficient, $C_1 = 1.00$, for E70XX weld electrodes.

$$\begin{aligned} R_n &= CC_1Dl \\ &= 3.52(1.00)(4.93 \text{ sixteenths})(11.5 \text{ in.}) \\ &= 200 \text{ kips} \end{aligned}$$

The available weld shear strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(200 \text{ kips})$ $= 150 \text{ kips} > 120 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{200 \text{ kips}}{2.00}$ $= 100 \text{ kips} > 80.0 \text{ kips} \quad \text{o.k.}$

Shear rupture of the HSS at the weld

$$\begin{aligned}
 t_{min} &= \frac{3.09D}{F_u} && \text{(Manual Eq. 9-6)} \\
 &= \frac{3.09(4.93 \text{ sixteenths})}{62 \text{ ksi}} \\
 &= 0.246 \text{ in.} < 0.349 \text{ in.} \quad \mathbf{o.k.}
 \end{aligned}$$

Therefore, shear rupture will not control the weld design.

Example 5.3—Through Plate-to-HSS Connection

Given:

Design a through-plate shear connection for the geometry and loading as shown in Figure 5-12. Use 3/4-in.-diameter Group 120 bolts with threads not excluded from the shear plane (thread condition N). Use standard holes in the beam web and horizontal short-slotted holes in the plate. The HSS column is ASTM A500/A500M Grade C, the beam is ASTM A992/A992M, and the plate is ASTM A572/A572M Grade 50. Use 70 ksi weld electrodes.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$F_y = 50 \text{ ksi}$

$F_u = 62 \text{ ksi}$

Beam

ASTM A992/A992M

$F_y = 50 \text{ ksi}$

$F_u = 65 \text{ ksi}$

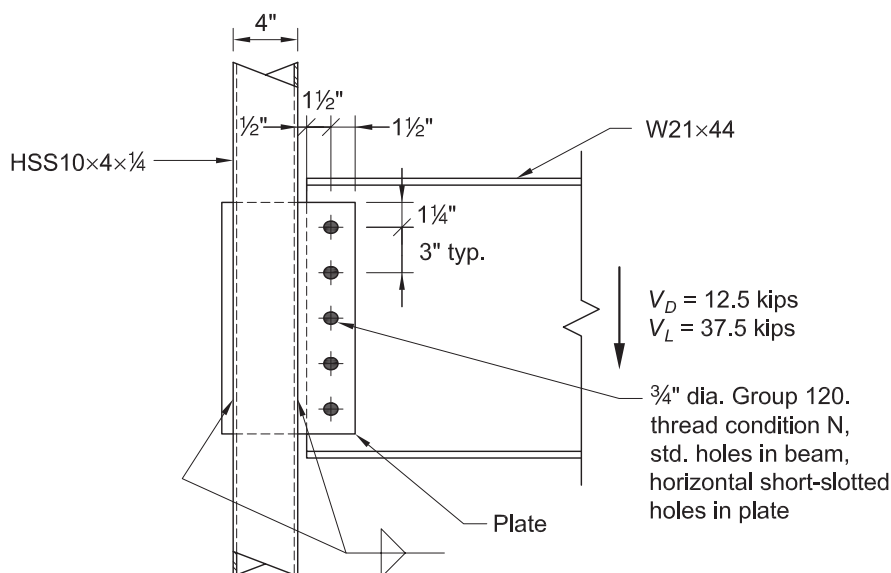


Fig. 5-12. Through-plate connection for Example 5.3.

Plate
 ASTM A572/A572M
 $F_{yp} = 50 \text{ ksi}$
 $F_{up} = 65 \text{ ksi}$

From AISC *Manual* Tables 1-1 and 1-11, the geometric properties are as follows:

Beam
 W21×44
 $t_w = 0.350 \text{ in.}$
 $T = 18\frac{3}{8} \text{ in.}$

Column
 HSS10×4×¼
 $B = 4.00 \text{ in.}$
 $H = 10.0 \text{ in.}$
 $t = 0.233 \text{ in.}$
 $h/t = 39.9$

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$V_u = 1.2V_D + 1.6V_L$ $= 1.2(12.5 \text{ kips}) + 1.6(37.5 \text{ kips})$ $= 75.0 \text{ kips}$	$V_a = V_D + V_L$ $= 12.5 \text{ kips} + 37.5 \text{ kips}$ $= 50.0 \text{ kips}$

A through-plate connection is very expensive and often not necessary. There are two main reasons a through plate may be required for a simple shear connection. First, from AISC *Manual* Part 10, a single-plate connection is not permitted if a member is slender for compression. Second, if the member wall thickness is not sufficient under shear yielding (punching shear) to weld a single plate to the face of the HSS, an alternate connection must be investigated. In either case, other types of connections, such as a double-angle connection, have an economic advantage over a through-plate connection.

Investigate a single-plate connection

Column slenderness

Per AISC *Manual* Part 10, single-plate connections may be used if the HSS column is nonslender under axial load.

$$h/t = 39.9$$

Per AISC *Specification* Table B4.1a, Case 6:

$$\begin{aligned}
 \lambda_r &= 1.40 \sqrt{\frac{E}{F_y}} \\
 &= 1.40 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\
 &= 31.5 < 39.9 \quad \text{n.g.}
 \end{aligned}$$

Therefore, an alternate connection is required. A through plate will be considered.

The through plate-to-beam connection is treated in the same manner as a single-plate connection.

Single-plate capacity

Try five rows of bolts,

$$\begin{aligned}l &= 2l_{ev} + (n-1)s \\&= 2(1\frac{1}{4} \text{ in.}) + (5-1)(3 \text{ in.}) \\&= 14.5 \text{ in.}\end{aligned}$$

Try a $\frac{3}{8}$ in. plate thickness.

Recommended plate length

From AISC *Manual* Part 10, the minimum recommended connection length is one-half of the T -dimension of the beam:

$$\begin{aligned}l_{min} &= \frac{T}{2} \\&= \frac{18\frac{3}{8} \text{ in.}}{2} \\&= 9.19 \text{ in.} < 14.5 \text{ in.} \quad \mathbf{o.k.}\end{aligned}$$
$$\begin{aligned}l_{max} &= T \\&= 18\frac{3}{8} \text{ in.} > 14.5 \text{ in.} \quad \mathbf{o.k.}\end{aligned}$$

Maximum thickness requirement

From AISC *Manual* Table 10-9, for a conventional single-plate connection with five bolts using short-slotted bolts, there is no maximum t_p or t_w .

Obtain the available single-plate connection strength from AISC *Manual* Table 10-10a:

LRFD	ASD
$\phi R_n = 111 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$R_n/\Omega = 74.0 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Weld to HSS

The strength of the welds in this connection are evaluated at the location of the maximum reaction along the weld line closest to the bolt line. The reaction at this weld line is determined by taking a moment about the weld line farthest from the bolt line (see Figure 5-8).

$$e_x = 2 \text{ in.}$$

LRFD	ASD
$\begin{aligned}V_{fu} &= \frac{R_u(B + e_x)}{B} \\&= \frac{(75.0 \text{ kips})(4.00 \text{ in.} + 2 \text{ in.})}{4.00 \text{ in.}} \\&= 113 \text{ kips}\end{aligned}$	$\begin{aligned}V_{fa} &= \frac{R_a(B + e_x)}{B} \\&= \frac{(50.0 \text{ kips})(4.00 \text{ in.} + 2 \text{ in.})}{4.00 \text{ in.}} \\&= 75.0 \text{ kips}\end{aligned}$

From AISC *Manual* Part 8, the required weld size is:

LRFD	ASD
$D_{req} = \frac{V_{fu}}{1.392l} \quad (\text{from Manual Eq. 8-2a})$ $= \frac{113 \text{ kips}}{1.392(14.5 \text{ in.})(2 \text{ lines})}$ $= 2.80 \text{ sixteenths}$	$D_{req} = \frac{V_{fa}}{0.928l} \quad (\text{from Manual Eq. 8-2b})$ $= \frac{75.0 \text{ kips}}{0.928(14.5 \text{ in.})(2 \text{ lines})}$ $= 2.79 \text{ sixteenths}$

Use double-sided $\frac{3}{16}$ in. fillet welds.

Shear yielding of HSS column

The available strength of the HSS for the limit state of shear yielding is given by AISC *Specification* Section J4.2(a).

$$\begin{aligned}
 R_n &= 0.60F_yA_{gv} && (\text{Spec Eq. J4-3}) \\
 &= 0.60F_y t l (2 \text{ planes}) \\
 &= 0.60(50 \text{ ksi})(0.233 \text{ in.})(14.5 \text{ in.})(2 \text{ planes}) \\
 &= 203 \text{ kips}
 \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(203 \text{ kips})$ $= 203 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{203 \text{ kips}}{1.50}$ $= 135 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Shear rupture of HSS column

The available shear rupture strength of the HSS is determined from AISC *Specification* Section J4.2(b).

$$\begin{aligned}
 R_n &= 0.60F_uA_{nv} && (\text{Spec Eq. J4-4}) \\
 &= 0.60F_u t l (2 \text{ planes}) \\
 &= 0.60(62 \text{ ksi})(0.233 \text{ in.})(14.5 \text{ in.})(2 \text{ planes}) \\
 &= 251 \text{ kips}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(251 \text{ kips})$ $= 188 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{251 \text{ kips}}{2.00}$ $= 126 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Beam web strength

The only applicable limit state for the beam web, as confirmed in Example 5.1, is the bolt strength.

Bolt strength

AISC *Specification* Section J3.7 gives the shear strength of a bolt as:

$$r_n = F_{nv} A_b \quad (\text{from Spec. Eq. J3-1})$$

From AISC *Specification* Table J3.2, for a Group 120 bolt with threads not excluded from shear planes, $F_{nv} = 54$ ksi. The nominal area of a $\frac{3}{4}$ -in.-diameter bolt is $A_b = 0.442$ in.². Therefore:

$$\begin{aligned} r_n &= (54 \text{ ksi})(0.442 \text{ in.}^2) \\ &= 23.9 \text{ kips/bolt} \end{aligned}$$

From AISC *Manual* Table 7-6, with $s = 3$ in., $e_x = 2$ in., and $n = 5$ bolts:

$$C = 4.39$$

The available strength of the bolts in shear is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = C \phi r_n$ $= 4.39(0.75)(23.9 \text{ kips/bolt})$ $= 78.7 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = C \left(\frac{r_n}{\Omega} \right)$ $= 4.39 \left(\frac{23.9 \text{ kips/bolt}}{2.00} \right)$ $= 52.5 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Bolt bearing and tearout in the beam web

From AISC *Specification* Table J3.3, the nominal hole diameter for a $\frac{3}{4}$ in. bolt in a standard hole is $d_h = \frac{13}{16}$ in. The clear distance between bolt holes is:

$$\begin{aligned} l_c &= s - d_h \\ &= 3 \text{ in.} - \frac{13}{16} \text{ in.} \\ &= 2.19 \text{ in.} \end{aligned}$$

From AISC *Specification* Section J3.11a, the nominal bearing strength of a bolt in a standard hole when deformation at the bolt hole at service load is a design consideration is:

$$\begin{aligned} r_n &= 2.4 d_b t_w F_u \quad (\text{from Spec. Eq. J3-6a}) \\ &= 2.4 \left(\frac{3}{4} \text{ in.} \right) (0.350 \text{ in.}) (65 \text{ ksi}) \\ &= 41.0 \text{ kips/bolt} \end{aligned}$$

From AISC *Specification* Section J3.11a, the nominal tearout strength of a bolt in a standard hole when deformation at the bolt hole is a design consideration is:

$$\begin{aligned} r_n &= 1.2 l_c t_w F_u \quad (\text{from Spec. Eq. J3-6c}) \\ &= 1.2 (2.19 \text{ in.}) (0.350 \text{ in.}) (65 \text{ ksi}) \\ &= 59.8 \text{ kips/bolt} \end{aligned}$$

The bolt shear strength controls over bearing and tearout in the beam web.

Bolt bearing and tearout in the plate

Because the plate thickness is greater than the beam web thickness, neither the bearing strength nor the tearout strength for interior bolts in the plate will control. Check the tearout strength of the edge bolt in the plate.

From AISC *Specification* Table J3.3, the nominal vertical hole dimension for a $\frac{3}{4}$ -in.-diameter bolt in a short-slotted hole is $d_h = \frac{13}{16}$ in. From Figure 5-12, the edge distance, $l_e = 1\frac{1}{4}$ in. Therefore, the clear distance is:

$$\begin{aligned} l_c &= l_e - \frac{d_h}{2} \\ &= 1\frac{1}{4} \text{ in.} - \frac{\frac{13}{16} \text{ in.}}{2} \\ &= 0.844 \text{ in.} \end{aligned}$$

From AISC *Specification* Section J3.11a, the nominal tearout strength of the edge bolt in a short-slotted hole when deformation at the bolt hole at service load is a design consideration is:

$$\begin{aligned} r_n &= 1.2l_c t_p F_u && \text{(from Spec. Eq. J3-6c)} \\ &= 1.2(0.844 \text{ in.})(\frac{3}{8} \text{ in.})(65 \text{ ksi}) \\ &= 24.7 \text{ kips/bolt} \end{aligned}$$

From previous calculations, the bolt shear strength controls over bearing and tearout in the plate for both interior and edge holes.

Chapter 6

Moment Connections

Moment connections are defined as connections where the transfer of moment forces is expected to occur and are classified as fully restrained (FR) and partially restrained (PR) in the *AISC Manual*. FR moment connections are sufficiently rigid to maintain the angle between the members at the joint when load is applied. A PR moment connection's behavior lies between the full rigidity of a FR moment connection and that of a simple shear connection. A PR connection can transfer moment to the supporting member, but the rotation is not negligible. The discussion in this chapter assumes the supporting HSS member is stiff enough to classify the connection as FR, as defined in *AISC Specification* Section B3.4b.

When considering moment connections, the most common beam shape connected to an HSS column is a wide flange and, thus, that is what this chapter will focus on. Information for HSS-to-HSS moment connections can be found in Chapter 9 of this Design Guide. While the information in Chapter 9 is focused on truss-type connections, it may be adapted for a conventional beam-to-column connection easily. As with this entire Design Guide, the scope is restricted to nonseismic applications.

Moment connections to unstiffened HSS columns with thin walls may behave in a semi-rigid manner. Design moments are limited to the yield moment produced by the plastification of the column wall, which is limited to a commonly accepted ultimate load deformation limit of 3% (Lu et al., 1994). Regardless of the shape of the column, developing full moment capacity of a beam often requires transverse column stiffeners to transfer axial loads in the beam flanges. For HSS column connections, these stiffeners can go through the HSS, internal to the HSS, or external to (or around) the HSS.

Some of the same connections that are used for wide-flange members can also be used for HSS members with occasional variations to account for the HSS closed section shape. These connections can be bolted or welded using the same bolt types and similar welding techniques as with other structural steel connections (AISC, 1997b).

AISC Manual Part 11 discusses six types of wide-flange beam-to-HSS column moment connections:

- HSS through-plate flange-plated
- HSS external diaphragm plate (cut-out plate) flange-plated
- HSS directly welded
- HSS above and below continuous beams
- HSS welded tee flange

- HSS internal diaphragm plate

These key items should be considered when designing connections to HSS members:

1. Due to the flexibility of HSS walls, the local strength of an HSS member at the connection may control the capacity of the connection. This is an important consideration when sizing the wall thickness of an HSS member prior to connection design.
2. Reinforcing HSS walls to locally strengthen the member is typically not an economical solution. This emphasizes the previous point that connections should be evaluated at the time the member size is determined.
3. The fillet weld size to HSS members that can be considered in design may be controlled by the strength of the base metal. See Chapter 3.
4. For elements connected transversely to the HSS face, the force transmitted through a fillet weld is not uniform due to the variation of flexural stiffness of the HSS wall across its section width.
5. Additional limit states to consider for bolts in tension fastened to HSS members are pullout and HSS wall distortion. Both limit states are discussed in Chapter 4.

The best configuration for a particular application depends on three factors:

- The magnitude of the moment that must be transferred to or through the HSS
- The magnitude of the axial force in the HSS
- The requirements of the orthogonal framing

The detail of the connection must accommodate the combined effects of mill, fabrication, and erection tolerances of the wide-flange beam. These include, but are not limited to, variations in the depth and tilt of the flanges outlined in *AISC Manual* Table 1-22. Shimming in the field with conventional shims or finger shims is the most used method to account for these variations. Accounting for tolerances during detailing is essential.

Costs associated with these connections can vary significantly. A continuous beam over an HSS column connection is the most economical connection of those listed, while connections that require the column to be split and welded back together, like the through-plate flange-plated connection and internal diaphragm plate connection, are significantly more expensive to fabricate.

6.1 CONTINUOUS BEAM OVER HSS COLUMN

A continuous beam over an HSS column, as shown in Figure 6-1, is a very efficient connection for single-story construction. It is made by bolting the bottom flange of a wide-flange beam to a cap plate welded to the top of an HSS column. Although some moment may be induced in the column due to the beam loading, this connection largely avoids the task of transferring moment to, or through, the column by running the beam through the joint.

There is an important stability issue that must be considered if this connection is employed. In this type of construction, the beam is braced only at the top flange. Therefore, out-of-plane motion at the top of the HSS is restrained only by the relatively weak flexural stiffness of the beam web, greatly increasing the effective length of the column.

The restraint to this motion must be increased by adding stiffeners to the web, as shown in Figure 6-1(a), or by using a bottom chord extension to the joist supported by the beam at the connection, as shown in Figure 6-1(b), or both, as illustrated in Figure 6-1(c). The stiffener in Figure 6-1(a) can also be used as the orthogonal framing connection. See *AISC Manual* Part 2 for additional information and details for beams framing continuously over columns. Stiffeners may also be provided to align with the walls of the HSS for a direct load path in heavily loaded connections. If these stiffeners are too close together to allow an orthogonal beam to frame in between them, an extended single-plate shear connection may be employed to allow enough clearance for bolt installation.

This connection may also be useful in multi-story projects because it can be the column splice location as well; see Figure 6-2. If the load from the column above is not significant, stiffener plates may be used to transfer the load down

through the wide-flange beam, as shown in Figure 6-2(a). If the column axial load is higher, column continuity may be provided by splitting the column section and welding it between the beam flanges, as shown in Figure 6-2(b). The beam flange should at least be as wide as the HSS column, or plates will have to be added to extend the width of the flanges locally. Alternatively, a rectangular column with its long dimension aligning with the beam length may be used.

The limit states specific to HSS that are applicable to this connection are HSS sidewall local yielding and HSS sidewall local crippling. Sidewall local yielding may occur when the member sidewalls, rather than the connecting face, are in compression or tension. Sidewall local crippling (or buckling) may occur when the sidewalls are in compression. These HSS limit states have been adapted (for two webs instead of one) from provisions for concentrated loading to a wide-flange beam web and can be found in *AISC Specification* Section J10.2 (yielding) and Section J10.3 (crippling). Additionally, prying action due to the moment force couple may be the controlling limit state and require the cap plate to be upsized.

Additional limit states for this connection include:

- Bending with prying on the flange of the beam
- Bending with prying on the cap plate of the HSS
- Strength of the beam web relative to yielding, crippling, and buckling
- Strength based on bolting to the cap plate
- Strength of the weld connecting the cap plate to the HSS
- Strength of the HSS wall relative to yielding and crippling

Example 6.1 in Section 6.7 illustrates the limit states applicable to this connection.

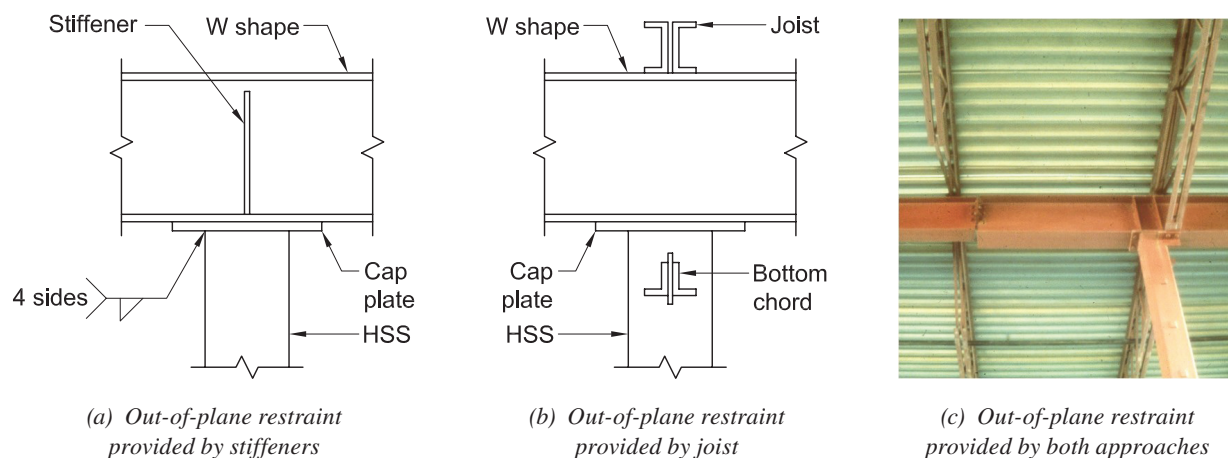


Fig. 6-1. Beam over HSS column.

6.2 EXTERNAL DIAPHRAGM PLATE (CUT-OUT PLATE) FLANGE-PLATED

An external diaphragm, or cut-out, plate connection employs top and bottom flange plates wider than the HSS column with a cut-out that wraps around the HSS. An external diaphragm plate connection can be two-sided, as shown in Figure 6-3, and used to transfer the moment from the beam on one side to the other or one-sided and used to transfer the moment directly to the column. A single plate welded to the face of the HSS is often used to support the beam shear load and allows for erection from the side. Other shear connections may be used at the expense of more difficult erection. If the external diaphragm plates are both shop-installed, the beams must be erected by sliding them in between the diaphragm plates. Beam mill tolerances including flange tilt and beam depth should be accounted for in detailing and will require field shims. Alternatively, the top flange plates may be field welded in place, allowing for a simpler erection. However, this comes at the expense of the field welding. In either case, it is recommended that the engineer of record (EOR), fabricator, and erector discuss the connection if specified on a project.

Attention should be paid to the framing orthogonal to the moment connection. Because the top and bottom flange plates extend beyond the face of the HSS column, they may create interference with orthogonal connections. If a moment connection is required in the orthogonal direction, and the beams are of identical depth, the same plates may be employed for that connection as well. If a simple shear connection is required, it is advantageous for the perpendicular beams to be shallower and their top flanges coped (or same depth with both flanges coped) to allow fit up between the

top and bottom flange plates. If the column and beams are adjacent to the exterior wall, care should be taken to detail the width of the plate to not extend into the exterior wall system.

For both the cut-out plate and the through-plate flange-plate connection described in Section 6.5, supplemental deck supports may be required when the beams connected in this manner are also supporting floor or roof deck. Deck support may be provided by a thin plate between the top flange and the flange plate that projects 3 in. for deck bearing or light angles welded to the edge of the flange plate.

For this connection, there are no limit states specific to HSS that are applicable. The load path is the force couple that is transferred from each beam flange, into the flange plates, through the flange plates around the column, and then into the beam on the other side of the column. If the connection is single sided, or if the magnitude of the moments are not equal on opposing sides of the joint, the moment will be transferred in the same manner, with the exception that the force couple will be delivered to the HSS column sidewalls.

Example 6.2 in Section 6.7 illustrates the limit states applicable to this connection.

6.3 WELDED TEE FLANGE

The welded tee flange connection uses a WT section at each of the top and bottom flanges to transfer the moment directly to the HSS column (see Figure 6-4). The tees should be long enough such that a flare-bevel groove weld can be made between the corner of the HSS column section and the back of the WT. This delivers the load directly to the two resisting sidewalls rather than requiring one face of the HSS column to deliver the load to the sidewalls. However, care should be

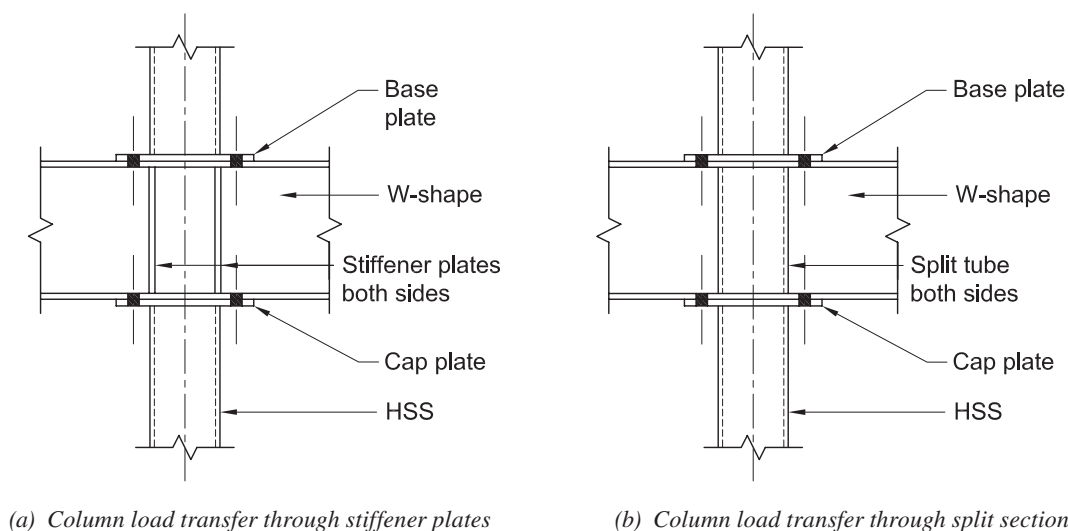


Fig. 6-2. HSS column-to-continuous beam in multilevel construction.

taken if a thick HSS column is specified because its corner radius will be large, resulting in a large flare-bevel groove weld.

The beam shear may be resisted by a single-plate connection at the beam web welded to the face of the HSS or by using the bottom WT as a seated connection. The bottom WT may require stiffeners to resist the shear load, but this should be avoided if possible.

The limit states specific to HSS that are applicable to this connection are HSS sidewall local yielding, HSS sidewall local crippling, and rupture strength of HSS at the weld. Sidewall local yielding may occur when the member sidewalls, rather than the connecting face, are in compression or tension. Sidewall local crippling (or buckling) may occur when the sidewalls are in compression. These HSS limit states have been adapted (for two webs instead of one) from provisions for concentrated loading to a wide-flange beam web and can be found in AISC *Specification* Section J10.2

(yielding) and Section J10.3 (crippling). Additionally, as discussed in Chapter 2, the HSS base metal strength must be considered.

Example 6.5 in Section 6.7 illustrates the limit states applicable to this connection.

6.4 DIRECTLY WELDED

Some moment transfer can be achieved by welding the flanges of the W-shape directly to the wall of the HSS column as shown in Figure 6-5. The single-plate connection transfers all the shear force and facilitates the erection of the beam. These connections may be capable of developing the full flexural strength of the HSS (depending primarily on the beam width relative to the HSS column width). Developing the full flexural strength of the W-shape, however, is seldom achievable.

The strength of the flanges in tension and compression is calculated by considering them as transverse plates

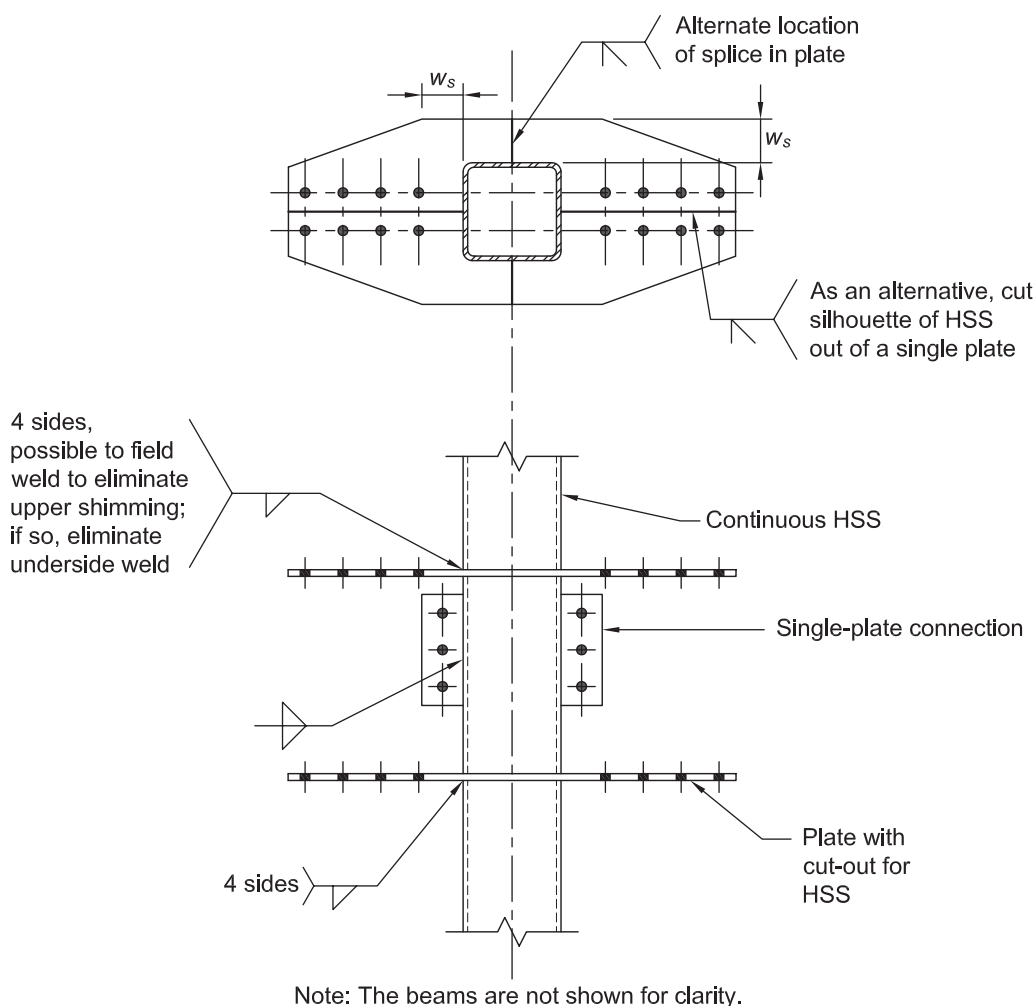


Fig. 6-3. Cut-out plate moment connection.

connected to the face of the HSS column. To obtain the maximum efficiency, the HSS should be thick, and the beam flange width should be close to the flat width of the HSS, estimated as $B - 3t$.

Due to the flexibility of the HSS wall, care should be taken when detailing this connection for a joint assumed to have negligible rotation during design. An unstiffened HSS face is not as inherently stiff and resistant to rotations as a W-shape column flange. It may be difficult to satisfy the negligible angle change and rotation requirements that constitute a FR moment connection.

If a directly welded connection is being used to support a cantilever, it may be prudent to specify a small camber of the cantilevered beam, as well as standard holes in the web connection to limit rotation. The camber must be coordinated with the EOR, fabricator and connection designer, and detailer.

The limit states that must be considered from the AISC *Specification* are local yielding of beam flanges due to uneven load distribution, HSS shear yielding (punching), sidewall tension, and sidewall compression failure. However, shear yielding only needs to be checked for flange widths less than the HSS width, and the sidewall limit states only apply when the flange and HSS widths are approximately the same.

Example 6.3 in Section 6.7 illustrates the limit states applicable to this connection.

6.5 THROUGH-PLATE FLANGE-PLATED

Through-plate moment connections are made by terminating the HSS column below the joint, providing top and bottom flange plates at the wide-flange beam flanges continuous from one side of the joint to the other, adding a short column section between the flange plates, and then continuing

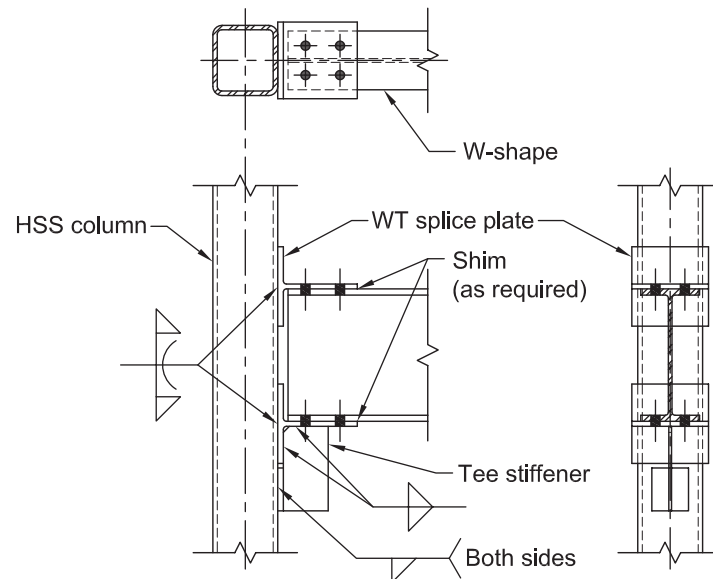


Fig. 6-4. Welded tee flange connection.

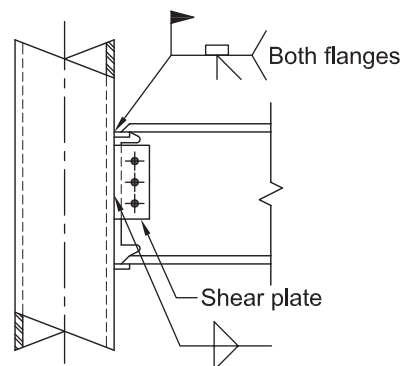


Fig. 6-5. Directly welded connection.

the column above the joint (see Figure 6-6). Single-plate or single-angle web connections are typically used to resist beam shear and allow for ease in erection. Because of the many pieces involved, this connection can be difficult and very costly to fabricate. However, the direct moment transfer from beam-to-beam across or to the column means the capacity of a through-plate connection is typically very high. An alternate fabrication technique would be to shop assemble the joint and stub beams and then provide a field-bolted splice at the beam inflection point. As shown in Figure 6-6(b), this connection may serve a dual purpose as a column splice as well.

Attention should be paid to the framing orthogonal to the moment connection. Because the top and bottom flange plates extend beyond the face of the HSS column, they may create interference with orthogonal connections. If a moment connection is required in the orthogonal direction, and the beams are of identical depth, the same plates may be employed for that connection as well. If a simple shear

connection is required, it is advantageous for the perpendicular beams to be shallower, and their top flanges coped (or same depth with both flanges coped) to allow fit up between the top and bottom flange plates. If the column and beams are adjacent to the exterior wall, care should be taken to detail the width of the plate to not extend into the exterior wall system.

Example 6.4 in Section 6.7 illustrates the limit states applicable to this connection.

6.6 INTERNAL DIAPHRAGM PLATE

A through-plated flange-plate connection interrupts the column twice at each connection. Instead, if an internal diaphragm is used, the column must be cut only once at each connection, which considerably reduces the fabrication costs. However, welding the diaphragm plates inside the cavity of the tube requires a one-sided weld and a tube large enough for proper weld and inspection access.

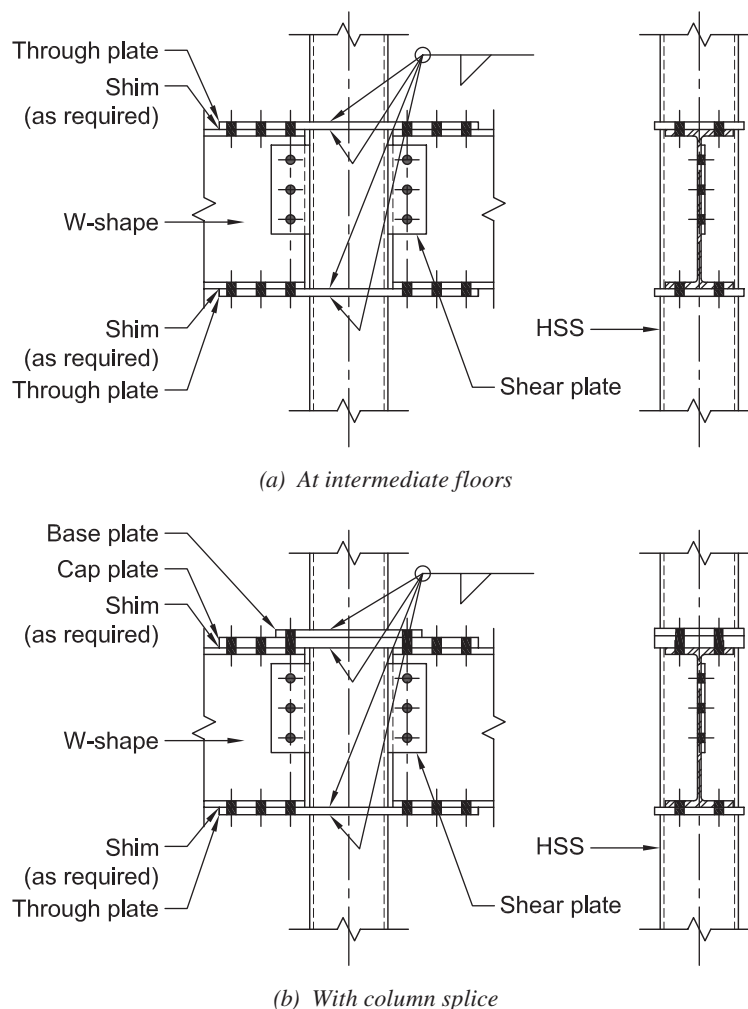


Fig. 6-6. Through-plate moment connection.

6.7 MOMENT CONNECTION DESIGN EXAMPLES

EXAMPLE 6.1—Beam over HSS Column Connection

Given:

Verify the adequacy of the connection shown in Figure 6-7, between a two-span ASTM A992/A992M W18×40 beam and an ASTM A500/A500M Grade C HSS8×8×¼ column. An ASTM A572/A572M Grade 50 cap plate is welded to the column with 70 ksi filler metal. The cap plate is bolted to the beam flange using four ¾-in.-diameter Group 120 bolts with threads not excluded from the shear plane (thread condition N). The column axial dead and live loads are $P_D = 7.5$ kips and $P_L = 22.5$ kips, respectively. The moment transferred across the beam-to-column interface consists of dead and live load moments of $M_D = 4.5$ kip-ft and $M_L = 13.5$ kip-ft, respectively.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

Beam

ASTM A992/A992M

$F_y = 50$ ksi

$F_u = 65$ ksi

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50$ ksi

$F_{up} = 65$ ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

Column

HSS8×8×¼

$B = 8.00$ in.

$t = 0.233$ in.

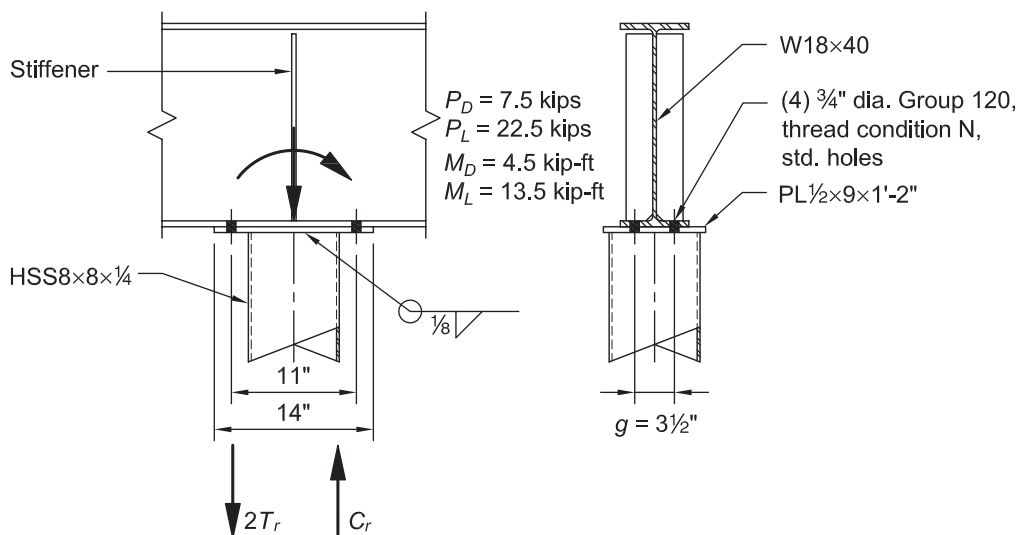


Fig. 6-7. Beam over HSS column connection for Example 6.1.

Beam

W18×40

$d = 17.9$ in.

$t_w = 0.315$ in.

$t_f = 0.525$ in.

$k_{des} = 0.927$ in.

$k_1 = 1\frac{3}{16}$ in.

$g = 3\frac{1}{2}$ in. (workable gage)

Solution:

From ASCE/SEI 7, Chapter 2, the required strengths are:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(7.5 \text{ kips}) + 1.6(22.5 \text{ kips})$ $= 45.0 \text{ kips}$ $M_u = 1.2M_D + 1.6M_L$ $= 1.2(4.5 \text{ kip-ft}) + 1.6(13.5 \text{ kip-ft})$ $= 27.0 \text{ kip-ft}$	$P_a = P_D + P_L$ $= 7.5 \text{ kips} + 22.5 \text{ kips}$ $= 30.0 \text{ kips}$ $M_a = M_D + M_L$ $= 4.5 \text{ kip-ft} + 13.5 \text{ kip-ft}$ $= 18.0 \text{ kip-ft}$

Prying Force

Calculate the bolt tension without prying forces, T_r , assuming the compression force, C_r , acts at the face of the column as shown in Figure 6-7. From the summation of forces and moments, the required bolt tension is determined as follows:

LRFD	ASD
$2T_u = \frac{M_u - P_u \left(\frac{B}{2} \right)}{\frac{B}{2} + \frac{s}{2}}$ $= \frac{(27.0 \text{ kip-ft})(12 \text{ in./ft}) - (45.0 \text{ kips}) \left(\frac{8.00 \text{ in.}}{2} \right)}{\frac{8.00 \text{ in.}}{2} + \frac{11 \text{ in.}}{2}}$ $= 15.2 \text{ kips}$ $T_u = 7.60 \text{ kips}$	$2T_a = \frac{M_a - P_a \left(\frac{B}{2} \right)}{\frac{B}{2} + \frac{s}{2}}$ $= \frac{(18.0 \text{ kip-ft})(12 \text{ in./ft}) - (30.0 \text{ kips}) \left(\frac{8.00 \text{ in.}}{2} \right)}{\frac{8.00 \text{ in.}}{2} + \frac{11 \text{ in.}}{2}}$ $= 10.1 \text{ kips}$ $T_a = 5.05 \text{ kips}$

Effect of prying action—Beam flange

Check the flange thickness for prying action using “Solution Method 1” from AISC *Manual* Part 9.

The distance from the bolt centerline to the face of the tee stem, b , and the distance from the face of the bolt to the face of the tee stem, b' , are:

$$b = \frac{g - t_w}{2}$$
$$= \frac{3\frac{1}{2} \text{ in.} - 0.315 \text{ in.}}{2}$$
$$= 1.59 \text{ in.}$$

$$\begin{aligned}
 b' &= b - \frac{d_b}{2} && \text{(from Manual Eq. 9-24)} \\
 &= 1.59 \text{ in.} - \frac{3/4 \text{ in.}}{2} \\
 &= 1.22 \text{ in.}
 \end{aligned}$$

The tributary length per bolt is calculated as shown in AISC *Manual* Figure 9-4(a). The bolt hole spacing, $s = 11$ in.

$$\begin{aligned}
 p &= 1.75b + \min\left(\frac{s}{2}, 1.75b\right) \\
 &= 1.75(1.59 \text{ in.}) + \min\left[\frac{11 \text{ in.}}{2}, 1.75(1.59 \text{ in.})\right] \\
 &= 2.78 \text{ in.} + \min(5.50 \text{ in.}, 2.78 \text{ in.}) \\
 &= 5.56 \text{ in.}
 \end{aligned}$$

AISC *Manual* Equations 9-34a (LRFD) and 9-34b (ASD) calculate the minimum element thickness that eliminates prying action, t_{np} :

LRFD	ASD
$\phi_b = 0.90$ (from <i>Manual</i> Eq. 9-34a) $t_{np} = \sqrt{\frac{4T_u b'}{\phi_b p F_u}}$ $= \sqrt{\frac{4(7.60 \text{ kips})(1.22 \text{ in.})}{0.90(5.56 \text{ in.})(65 \text{ ksi})}}$ $= 0.338 \text{ in.}$ $t_f = 0.525 \text{ in.} > 0.338 \text{ in.} \quad \mathbf{o.k.}$	$\Omega_b = 1.67$ (from <i>Manual</i> Eq. 9-34b) $t_{np} = \sqrt{\frac{4\Omega_b T_u b'}{p F_u}}$ $= \sqrt{\frac{4(1.67)(5.05 \text{ kips})(1.22 \text{ in.})}{(5.56 \text{ in.})(65 \text{ ksi})}}$ $= 0.337 \text{ in.}$ $t_f = 0.525 \text{ in.} > 0.337 \text{ in.} \quad \mathbf{o.k.}$

Because $t_{np} < t_f$, there is no prying action in the beam flange.

Effect of prying action—Cap plate

Check the cap-plate thickness for prying action using “Solution Method 1” from AISC *Manual* Part 9. The longitudinal bolt spacing is $s = 11$ in.

$$\begin{aligned}
 b &= \frac{s - B}{2} \\
 &= \frac{11 \text{ in.} - 8 \text{ in.}}{2} \\
 &= 1.50 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 b' &= b - \frac{d_b}{2} && \text{(from Manual Eq. 9-24)} \\
 &= 1.50 \text{ in.} - \frac{3/4 \text{ in.}}{2} \\
 &= 1.13 \text{ in.}
 \end{aligned}$$

The tributary length per bolt is calculated as shown in AISC *Manual* Figure 9-4(b). The transverse bolt hole spacing is equal to the workable gage of the beam, $g = 3\frac{1}{2}$ in.

$$\begin{aligned}
 l_e &= \frac{b_p - g}{2} \\
 &= \frac{9 \text{ in.} - 3\frac{1}{2} \text{ in.}}{2} \\
 &= 2.75 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 p &= \min(l_e, 1.75b) + \min\left(\frac{g}{2}, 1.75b\right) \\
 &= \min[2.75 \text{ in.}, 1.75(1.50 \text{ in.})] + \min\left[\frac{3\frac{1}{2} \text{ in.}}{2}, 1.75(1.50 \text{ in.})\right] \\
 &= \min(2.75 \text{ in.}, 2.63 \text{ in.}) + \min(1.75 \text{ in.}, 2.63 \text{ in.}) \\
 &= 4.38 \text{ in.}
 \end{aligned}$$

AISC *Manual* Equations 9-34a (LRFD) and 9-34b (ASD) calculate the minimum element thickness that eliminates prying action, t_{np} :

LRFD	ASD
$\phi_b = 0.90$ (from <i>Manual</i> Eq. 9-34a) $t_{np} = \sqrt{\frac{4T_u b'}{\phi_b p F_{up}}}$ $= \sqrt{\frac{4(7.60 \text{ kips})(1.13 \text{ in.})}{0.90(4.38 \text{ in.})(65 \text{ ksi})}}$ $= 0.366 \text{ in.}$ $t_p = \frac{1}{2} \text{ in.} > 0.366 \text{ in.} \quad \text{o.k.}$	$\Omega_b = 1.67$ (from <i>Manual</i> Eq. 9-34b) $t_{np} = \sqrt{\frac{4\Omega_b T_a b'}{p F_{up}}}$ $= \sqrt{\frac{4(1.67)(5.05 \text{ kips})(1.13 \text{ in.})}{(4.38 \text{ in.})(65 \text{ ksi})}}$ $= 0.366 \text{ in.}$ $t_p = \frac{1}{2} \text{ in.} > 0.366 \text{ in.} \quad \text{o.k.}$

Because $t_{np} < t_p$, there is no prying action in the cap plate.

Bolt available tensile strength

Because there is no prying action in either the beam flange or the cap plate, the required strength of the bolt is T_r (T_u for LRFD and T_a for ASD). The available tensile strength, ϕr_n and r_n/Ω of a $\frac{3}{4}$ -in.-diameter bolt is given in AISC *Manual* Table 7-2.

LRFD	ASD
$\phi r_n = 29.8 \text{ kips/bolt} > T_u = 7.60 \text{ kips/bolt} \quad \text{o.k.}$	$\frac{r_n}{\Omega} = 19.9 \text{ kips/bolt} > T_a = 5.05 \text{ kips/bolt} \quad \text{o.k.}$

Beam web local yielding

The limit state of beam web local yielding is checked as follows.

Determine the compression force, C_r (C_u for LRFD and C_a for ASD).

LRFD	ASD
$C_u = 2T_u + P_u$ $= 15.2 \text{ kips} + 45.0 \text{ kips}$ $= 60.2 \text{ kips}$	$C_a = 2T_a + P_a$ $= 10.1 \text{ kips} + 30.0 \text{ kips}$ $= 40.1 \text{ kips}$

Using the same load dispersion width of 2.5:1 as shown in AISC *Specification* Commentary Figure C-K2.2, the bearing length is:

$$\begin{aligned} l_b &= 5t_p + t \\ &= 5\left(\frac{1}{2} \text{ in.}\right) + 0.233 \text{ in.} \\ &= 2.73 \text{ in.} \end{aligned}$$

The nominal web local yielding strength is determined using AISC *Specification* Section J10.2. The concentrated force to be resisted is applied at a distance from the member end that is greater than the full nominal depth of the member. Therefore, AISC *Specification* Equation J10-2 applies:

$$\begin{aligned} R_n &= F_y t_w (5k + l_b) && (\text{Spec. Eq. J10-2}) \\ &= (50 \text{ ksi})(0.315 \text{ in.})[5(0.927 \text{ in.}) + 2.73 \text{ in.}] \\ &= 116 \text{ kips} \end{aligned}$$

The available strength of the beam web for the limit state of local yielding is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(116 \text{ kips})$ $= 116 \text{ kips} > 60.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{116 \text{ kips}}{1.50}$ $= 77.3 \text{ kips} > 40.1 \text{ kips} \quad \mathbf{o.k.}$

Beam web crippling

The nominal web local crippling strength is determined using AISC *Specification* Section J10.3. The concentrated compressive force to be resisted is applied at a distance from the member end greater than $d/2$; therefore, AISC *Specification* Equation J10-4 applies.

$$R_n = 0.80t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f \quad (\text{Spec Eq. J10-4})$$

where

$$Q_f = 1.0 \text{ for wide-flange sections}$$

$$l_b = 2.73 \text{ in.}$$

The nominal compressive strength due to web local crippling is:

$$\begin{aligned} R_n &= 0.80(0.315 \text{ in.})^2 \left[1 + 3 \left(\frac{2.73 \text{ in.}}{17.9 \text{ in.}} \right) \left(\frac{0.315 \text{ in.}}{0.525 \text{ in.}} \right)^{1.5} \right] \sqrt{\frac{(29,000 \text{ ksi})(50 \text{ ksi})(0.525 \text{ in.})}{0.315 \text{ in.}}} (1.0) \\ &= 150 \text{ kips} \end{aligned}$$

The available compressive strength of the HSS for the limit state of web local crippling is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(150 \text{ kips})$ $= 113 \text{ kips} > 60.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{150 \text{ kips}}{2.00}$ $= 75.0 \text{ kips} > 40.1 \text{ kips} \quad \mathbf{o.k.}$

Alternatively, AISC *Specification* Equation J10-4 can be simplified using AISC *Manual* Table 9-4 as follows:

LRFD	ASD
Because the concentrated compressive force to be resisted is applied at a distance from the member end greater than $d/2$, use <i>Manual</i> Equation 9-64a: $\phi R_n = 2(\phi R_3 + l_b \phi R_4)$ From AISC <i>Manual</i> Table 9-4 for a W18×40: $\phi R_3 = 46.3 \text{ kips}$ $\phi R_4 = 3.60 \text{ kip/in.}$ $\phi R_n = 2[46.3 \text{ kips} + (2.73 \text{ in.})(3.60 \text{ kip/in.})]$ $= 112 \text{ kips} > 60.2 \text{ kips}$	Because the concentrated compressive force to be resisted is applied at a distance from the member end greater than $d/2$, use <i>Manual</i> Equation 9-64b: $R_n/\Omega = 2(R_3/\Omega + l_b R_4/\Omega)$ From AISC <i>Manual</i> Table 9-4 for a W18×40: $R_3/\Omega = 30.9 \text{ kips}$ $R_4/\Omega = 2.40 \text{ kips/in.}$ $R_n/\Omega = 2[30.9 \text{ kips} + (2.73 \text{ in.})(2.40 \text{ kips/in.})]$ $= 74.9 \text{ kips} > 40.1 \text{ kips}$

Beam web compression buckling

Note that if the total axial force was applied at the beam top flange over the connection, web compression buckling (AISC *Specification* Section J10.5) would also be checked to determine if a stiffener is required over the HSS wall.

HSS wall strength

The compression force in the HSS wall may not be uniformly distributed over the entire HSS wall width due to shear lag. This phenomenon is also discussed in Section 7.1 of this Design Guide. Assume that the force in the beam web disperses at a slope of 2.5:1 through the beam flange.

$$\begin{aligned}
 N &= 2k_1 + 5t_f \\
 &= 2(1\frac{13}{16} \text{ in.}) + 5(0.525 \text{ in.}) \\
 &= 4.25 \text{ in.}
 \end{aligned}$$

Using the same load dispersion width of 2.5:1 through the cap plate as shown in AISC *Specification* Commentary Figure C-K2.2, the width is:

$$\begin{aligned}
 5k + l_b &= 5t_p + N \\
 &= 5(\frac{1}{2} \text{ in.}) + 4.25 \text{ in.} \\
 &= 6.75 \text{ in.} < 8.00 \text{ in.}
 \end{aligned}$$

Only one wall of the HSS is subjected to the compression load. Therefore, only part of one HSS wall contributes to the load resistance.

Local yielding of the HSS wall

The nominal strength per AISC *Specification* Section J4.1 is:

$$\begin{aligned}
 R_n &= F_y t (5k + l_b) && \text{(from Spec. Eq. J10-2)} \\
 &= (50 \text{ ksi})(0.233 \text{ in.})(6.75 \text{ in.}) \\
 &= 78.6 \text{ kips}
 \end{aligned}$$

The available strength of the HSS wall for the limit state of local yielding is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(78.6 \text{ kips})$ $= 78.6 \text{ kips} > 60.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{78.6 \text{ kips}}{1.50}$ $= 52.4 \text{ kips} > 40.1 \text{ kips} \quad \mathbf{o.k.}$

HSS wall local crippling

From AISC *Specification* Section J10.3, the nominal compressive strength based on the limit state of wall local crippling, for one wall, is:

$$R_n = 0.80t^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t}{t_p} \right)^{1.5} \right] \sqrt{\frac{EF_y t_p}{t}} Q_f \quad (\text{from Spec. Eq. J10-4})$$

The transverse plate thickness, l_b , is the beam web in this case and is taken as:

$$\begin{aligned} l_b &= 2k_1 \\ &= 2(1\frac{3}{16} \text{ in.}) \\ &= 1.63 \text{ in.} \end{aligned}$$

$$\begin{aligned} d &= B/2 \\ &= (8.00 \text{ in.})/2 \\ &= 4.00 \text{ in.} \end{aligned}$$

The chord-stress interaction parameter, Q_f , has been omitted because it is intended for connections where forces are normal to the compressive stress in the HSS wall.

In the cap plate connection, the compressive force is parallel to the compressive stress in the HSS wall. The nominal compressive strength due to wall local crippling is:

$$\begin{aligned} R_n &= 0.80(0.233 \text{ in.})^2 \left[1 + 3 \left(\frac{1.63 \text{ in.}}{4.00 \text{ in.}} \right) \left(\frac{0.233 \text{ in.}}{1/2 \text{ in.}} \right)^{1.5} \right] \sqrt{\frac{(29,000 \text{ ksi})(50 \text{ ksi})(1/2 \text{ in.})}{0.233 \text{ in.}}} \\ &= 106 \text{ kips} \end{aligned}$$

The available compressive strength of the HSS for the limit state of wall local crippling is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(106 \text{ kips})$ $= 79.5 \text{ kips} > 60.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{106 \text{ kips}}{2.00}$ $= 53.0 \text{ kips} > 40.1 \text{ kips} \quad \mathbf{o.k.}$

Weld size between the HSS and cap plate

Using a 45° maximum load dispersion angle, the weld of the cap plate to the HSS can be checked by evaluating a load per inch on the lesser of the two distances: g or $d_b + 2b$, where b is the distance from the bolt center to the face of the HSS. If $2g$ or $2(d_b + 2b)$ exceeds the width of the HSS, B , the weld should be checked for $2T_r/B$.

$$g = 3\frac{1}{2} \text{ in.}$$

$$\begin{aligned} d_b + 2b &= \frac{3}{4} \text{ in.} + 2(1.50 \text{ in.}) \\ &= 3\frac{3}{4} \text{ in.} \end{aligned}$$

$$g < d_b + 2b$$

$$\begin{aligned} 2g &= 2(3\frac{1}{2} \text{ in.}) \\ &= 7.00 \text{ in.} < 8.00 \text{ in.} \quad \mathbf{o.k.} \end{aligned}$$

$$\begin{aligned} 2(d_b + 2b) &= 2\left[\frac{3}{4} \text{ in.} + 2\left(\frac{11.0 \text{ in.} - 8.00 \text{ in.}}{2}\right)\right] \\ &= 7.50 \text{ in.} < 8.00 \text{ in.} \quad \mathbf{o.k.} \end{aligned}$$

Because $g < d_b + 2b$ and neither $2g$ nor $2(d_b + 2b)$ exceed the width of the HSS, use T_r/g as the load per inch.

Using the procedure given in AISC *Manual* Part 8, determine the weld size required.

LRFD	ASD
The required strength per inch of weld is: $\frac{T_u}{g} = \frac{7.60 \text{ kips}}{3\frac{1}{2} \text{ in.}} = 2.17 \text{ kips/in.}$ The required weld size is determined from AISC <i>Manual</i> Equation 8-2a, neglecting the l term to determine a weld strength per inch: $(1.392 \text{ kip/in.})D \geq 2.17 \text{ kips/in.}$ $D \geq 1.56 \text{ sixteenths}$ Use an $\frac{1}{8}$ in. fillet weld.	The required strength per inch of weld is: $\frac{T_a}{g} = \frac{5.05 \text{ kips}}{3\frac{1}{2} \text{ in.}} = 1.44 \text{ kips/in.}$ The required weld size is determined from AISC <i>Manual</i> Equation 8-2b, neglecting the l term to determine a weld strength per inch: $(0.928 \text{ kip/in.})D \geq 1.44 \text{ kips/in.}$ $D \geq 1.55 \text{ sixteenths}$ Use an $\frac{1}{8}$ in. fillet weld.

From AISC *Specification* Table J2.4, the minimum fillet weld size for a $\frac{1}{4}$ -in.-thick HSS wall is $\frac{1}{8}$ in. Therefore, an $\frac{1}{8}$ in. fillet weld is acceptable.

EXAMPLE 6.2—External Diaphragm Plate (Cut-Out Plate) Flange-Plated Connection

Given:

Determine the required size of the cut-out plate for an external diaphragm plate connection between two ASTM A992/A992M W18×35 beams and an ASTM A500/A500M Grade C HSS10×10× $\frac{1}{2}$ column, as shown in Figure 6-8. The full beam plastic moment is required to be transferred across the HSS. Use $\frac{3}{4}$ -in.-diameter Group 120 bolts with threads not excluded from the shear plane (thread condition N). Use standard holes in both the beam and plate. The plate material is ASTM A572/A572M Grade 50.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$F_y = 50 \text{ ksi}$

$F_u = 62 \text{ ksi}$

Beam
 ASTM A992/A992M
 $F_y = 50$ ksi
 $F_u = 65$ ksi

Plate
 ASTM A572/A572M Grade 50
 $F_{yp} = 50$ ksi
 $F_{up} = 65$ ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

Column
 HSS10×10× $\frac{1}{2}$
 $t = 0.465$ in.

Beams
 W18×35
 $d = 17.7$ in.
 $b_f = 6.00$ in.
 $t_f = 0.425$ in.
 $g = 3\frac{1}{2}$ in. (workable gage)
 $S_x = 57.6$ in.³
 $Z_x = 66.5$ in.³

From AISC *Specification* Table J3.3, the hole diameter for a $\frac{3}{4}$ -in.-diameter bolt with standard holes is:

$$d_h = \frac{13}{16} \text{ in.}$$

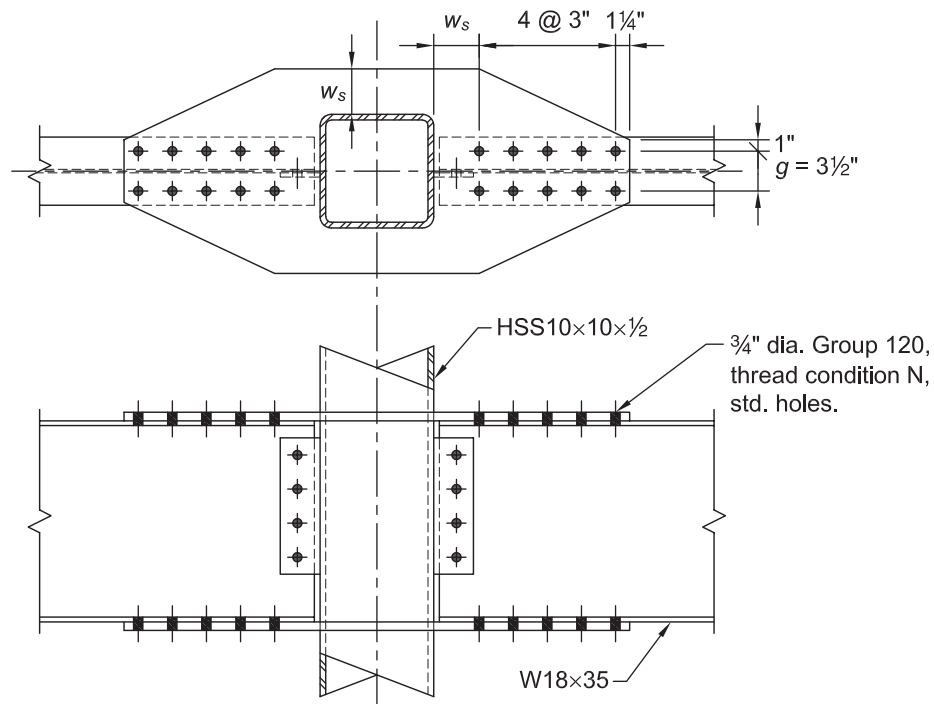


Fig. 6-8. Cut-out plate connection for Example 6.2.

Solution:*Flexural strength of the beam*

The nominal flexural strength of the beam based on the limit state of yielding is:

$$\begin{aligned}
 M_n &= F_y Z_x && \text{(from Spec. Eq. F2-1)} \\
 &= (50 \text{ ksi})(66.5 \text{ in.}^3) \\
 &= 3,330 \text{ kip-in.}
 \end{aligned}$$

Because there are bolt holes in the tension flange, the limit state of tension rupture may apply. The tension flange is checked in accordance with AISC *Specification* Section F13.1.

The gross flange area, A_{fg} , is calculated in accordance with AISC *Specification* Section B4.3a:

$$\begin{aligned}
 A_{fg} &= b_f t_f \\
 &= (6.00 \text{ in.})(0.425 \text{ in.}) \\
 &= 2.55 \text{ in.}^2
 \end{aligned}$$

The net flange area, A_{fn} , is calculated in accordance with AISC *Specification* Section B4.3b:

$$\begin{aligned}
 A_{fn} &= [b_f - 2(d_h + 1/16 \text{ in.})] t_f \\
 &= [6.00 \text{ in.} - 2(1 3/16 \text{ in.} + 1/16 \text{ in.})](0.425 \text{ in.}) \\
 &= 1.81 \text{ in.}^2
 \end{aligned}$$

$$\begin{aligned}
 \frac{F_y}{F_u} &= \frac{50 \text{ ksi}}{65 \text{ ksi}} \\
 &= 0.769 < 0.8; \text{ therefore, } Y_t = 1.0
 \end{aligned}$$

$$\begin{aligned}
 F_u A_{fn} &= (65 \text{ ksi})(1.81 \text{ in.}^2) \\
 &= 118 \text{ kips}
 \end{aligned}$$

$$\begin{aligned}
 Y_t F_y A_{fg} &= 1.0(50 \text{ ksi})(2.55 \text{ in.}^2) \\
 &= 128 \text{ kips} > 118 \text{ kips}
 \end{aligned}$$

Therefore, the limit state of tension rupture does apply. The nominal flexural strength at the location of the holes in the tension flange will be no greater than:

$$\begin{aligned}
 M_n &= \frac{F_u A_{fn}}{A_{fg}} S_x && \text{(Spec. Eq. F13-1)} \\
 &= \frac{(65 \text{ ksi})(1.81 \text{ in.}^2)}{2.55 \text{ in.}^2} (57.6 \text{ in.}^3) \\
 &= 2,660 \text{ kip-in.}
 \end{aligned}$$

The nominal flexural strength at the bolt holes due to tension rupture controls. The available flexural strength is calculated as:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(2,660 \text{ kip-in.})$ $= 2,390 \text{ kip-in.}$ $= 199 \text{ kip-ft}$	$\Omega_b = 1.67$ $\frac{M_n}{\Omega_b} = \frac{2,660 \text{ kip-in.}}{1.67}$ $= 1,590 \text{ kip-in.}$ $= 133 \text{ kip-ft}$

Approximate plate axial forces

Use a plate thickness equal to the flange thickness to determine an approximate required axial force in the plate, R_u or R_a .

LRFD	ASD
$M_u \approx R_u(d + t_f)$ $R_u \approx \frac{M_u}{d + t_f}$ $\approx \frac{2,660 \text{ kip-in.}}{17.7 \text{ in.} + 0.425 \text{ in.}}$ $\approx 147 \text{ kips}$	$M_a \approx R_a(d + t_f)$ $R_a \approx \frac{M_a}{d + t_f}$ $\approx \frac{1,590 \text{ kip-in.}}{17.7 \text{ in.} + 0.425 \text{ in.}}$ $\approx 87.7 \text{ kips}$

Compressive strength in the plate at the cut-out

The critical section of the cut-out plate occurs along the sides of the HSS, where the plate projects beyond the flat of the HSS. Therefore, the required plate projection, w_s , on each side can be determined for the maximum compression force.

Try $w_s = 4 \text{ in.}$ and cut-out plate thickness, $t_p = \frac{3}{4} \text{ in.}$

Check the width-to-thickness ratio of the plate projection for compression.

$$\frac{w_s}{t_p} = \frac{4 \text{ in.}}{\frac{3}{4} \text{ in.}}$$

$$= 5.33$$

From AISC *Specification* Table B4.1a, Case 3 for unstiffened elements:

$$\lambda_r = 0.45 \sqrt{\frac{E}{F_y}}$$

$$= 0.45 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}}$$

$$= 10.8 > 5.33 \quad \mathbf{o.k.}$$

Therefore, the plate is nonslender. The critical buckling stress is taken as equal to the yield stress because the effective length of the member is zero.

$$P_n = F_y A_g \quad (\text{Spec. Eq. J4-6})$$

$$= F_y t_p w_s$$

$$= (50 \text{ ksi})(\frac{3}{4} \text{ in.})(4 \text{ in.})$$

$$= 150 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi P_n = 0.90(150 \text{ kips})$ $= 135 \text{ kips}$	$\Omega = 1.67$ $\frac{P_n}{\Omega} = \frac{150 \text{ kips}}{1.67}$ $= 89.8 \text{ kips}$

The required axial force will be divided between the plates on each of the two sides of the HSS.

LRFD	ASD
$P_u = \frac{147 \text{ kips}}{2}$ $= 73.5 \text{ kips} < 135 \text{ kips} \quad \text{o.k.}$	$P_a = \frac{110 \text{ kips}}{2}$ $= 43.9 \text{ kips} < 89.8 \text{ kips} \quad \text{o.k.}$

Check the maximum force in cut-out plate using the proposed plate thickness:

LRFD	ASD
$M_u = R_u(d + t_p)$ $R_u = \frac{M_u}{d + t_p}$ $= \frac{2,660 \text{ kip-in.}}{17.7 \text{ in.} + \frac{3}{4} \text{ in.}}$ $= 144 \text{ kips}$	$M_a = R_a(d + t_p)$ $R_a = \frac{M_a}{d + t_p}$ $= \frac{1,590 \text{ kip-in.}}{17.7 \text{ in.} + \frac{3}{4} \text{ in.}}$ $= 86.2 \text{ kips}$

Strength of the bolted connection

The strength of the bolt group is taken as the sum of the strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.7, the bearing strength of the plate or beam flange at the bolt hole per AISC *Specification* Section J3.11, or the tearout strength of the plate or beam flange at the bolt hole per AISC *Specification* Section J3.11.

From AISC Manual Table 7-1, the available shear strength for a $\frac{3}{4}$ -in.-diameter, Group 120 bolt with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 17.9 \text{ kips/bolt}$	$\frac{r_n}{\Omega} = 11.9 \text{ kips/bolt}$

The available bearing strength of the plate per bolt is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration:

$$\begin{aligned}
 r_n &= 2.4 d t_p F_u && \text{(from Spec. Eq. J3-6a)} \\
 &= 2.4 \left(\frac{3}{4} \text{ in.}\right) \left(\frac{3}{4} \text{ in.}\right) (65 \text{ ksi}) \\
 &= 87.8 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(87.8 \text{ kips/bolt})$ $= 65.9 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{87.8 \text{ kips}}{2.00}$ $= 43.9 \text{ kips/bolt}$

The available tearout strength of the plate at the interior bolts is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration.

$$\begin{aligned}
 l_c &= s - d_h \\
 &= 3 \text{ in.} - 13/16 \text{ in.} \\
 &= 2.19 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 r_n &= 1.2l_c t_p F_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(2.19 \text{ in.})(3/4 \text{ in.})(65 \text{ ksi}) \\
 &= 128 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(128 \text{ kips/bolt})$ $= 96.0 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{128 \text{ kips/bolt}}{2.00}$ $= 64.0 \text{ kips/bolt}$

The available tearout strength of the plate at the edge bolts is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration.

$$\begin{aligned}
 l_c &= l_e - d_h/2 \\
 &= 1\frac{1}{4} \text{ in.} - (13/16 \text{ in.})/2 \\
 &= 0.844 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 r_n &= 1.2l_c t_p F_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(0.844 \text{ in.})(3/4 \text{ in.})(65 \text{ ksi}) \\
 &= 49.4 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(49.4 \text{ kips/bolt})$ $= 37.1 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{49.4 \text{ kips/bolt}}{2.00}$ $= 24.7 \text{ kips/bolt}$

The available bearing strength of the beam flange per bolt is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration:

$$\begin{aligned}
 r_n &= 2.4 d t_f F_u && \text{(from Spec. Eq. J3-6a)} \\
 &= 2.4(3/4 \text{ in.})(0.425 \text{ in.})(65 \text{ ksi}) \\
 &= 49.7 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(49.7 \text{ kips/bolt})$ $= 37.3 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{49.7 \text{ kips/bolt}}{2.00}$ $= 24.9 \text{ kips/bolt}$

The available tearout strength of the beam flange at the interior bolts is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration.

$$\begin{aligned}
 l_c &= s - d_h \\
 &= 3 \text{ in.} - \frac{13}{16} \text{ in.} \\
 &= 2.19 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 r_n &= 1.2l_c t_f F_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(2.19 \text{ in.})(0.425 \text{ in.})(65 \text{ ksi}) \\
 &= 72.6 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(72.6 \text{ kips/bolt})$ $= 54.5 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{72.6 \text{ kips/bolt}}{2.00}$ $= 36.3 \text{ kips/bolt}$

The available tearout strength of the beam flange at the edge bolt is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration. Assuming a $\frac{1}{2}$ in. beam setback from the face of the column, the edge distance is $l_e = 3\frac{1}{2}$ in.

$$\begin{aligned}
 l_c &= l_e - \frac{d_h}{2} \\
 &= 3\frac{1}{2} \text{ in.} - \frac{\frac{13}{16} \text{ in.}}{2} \\
 &= 2.69 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 r_n &= 1.2l_c t_f F_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(2.69 \text{ in.})(0.425 \text{ in.})(65 \text{ ksi}) \\
 &= 89.2 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(89.2 \text{ kips/bolt})$ $= 66.9 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{89.2 \text{ kips}}{2.00}$ $= 44.6 \text{ kips/bolt}$

Therefore, the available bolt shear strength controls over bearing or tearout of the plate or beam flange.

The available shear transfer strength of the bolt group in the cut-out plate is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = \phi r_n n$ $= (17.9 \text{ kips/bolt})(10 \text{ bolts})$ $= 179 \text{ kips} > 144 \text{ kips} \quad \mathbf{o.k.}$	$\frac{R_n}{\Omega} = \frac{r_n}{\Omega} n$ $= (11.9 \text{ kips/bolt})(10 \text{ bolts})$ $= 119 \text{ kips} > 86.2 \text{ kips} \quad \mathbf{o.k.}$

Tensile yielding of cut-out plate

The critical location for tensile yielding will occur at the location of the minimum gross area; either the first bolt line closest to the column or at the cut-out. Based on the width of the Whitmore section at the first row of bolts, the usable plate width is:

$$l_w = 2[4(3 \text{ in.})\tan 30^\circ] + 3\frac{1}{2} \text{ in.}$$

$$= 17.4 \text{ in.}$$

The width of the plate at the cut-out, $w_s = 4 \text{ in.}$; $2w_s = 8.00 \text{ in.}$ Therefore, the critical location for tensile yielding is at the cut out.

Because the cut out is controlling the design of the plates, the following check is not required and will not control; however, it has been shown for illustration.

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the cut-out plate is determined as follows:

$$A_g = t_p(2w_s)$$

$$= (\frac{3}{4} \text{ in.})(2)(4 \text{ in.})$$

$$= 6.00 \text{ in.}^2$$

$$R_n = F_y A_g \quad (\text{Spec Eq. J4-1})$$

$$= (50 \text{ ksi})(6.00 \text{ in.}^2)$$

$$= 300 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(300 \text{ kips})$ $= 270 \text{ kips} > 144 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{300 \text{ kips}}{1.67}$ $= 180 \text{ kips} > 86.2 \text{ kips} \quad \mathbf{o.k.}$

Tensile rupture of cut-out plate

Based on the width of the Whitmore section previously calculated, check the limit state of tensile rupture of the cut-out plate. From AISC *Specification* Section J4.1(b), the nominal tensile rupture strength of the cut-out plate is:

$$R_n = F_u A_e \quad (\text{Spec. Eq. J4-2})$$

The effective area, A_e , is determined from AISC *Specification* Section D3 as follows:

$$A_e = A_n U \quad (\text{Spec. Eq. D3-1})$$

$$U = 1.0 \text{ from AISC } \textit{Specification} \text{ Table D3.1 (no unconnected elements)}$$

The net tensile area is:

$$A_n = A_g - 2(d_h + \frac{1}{16} \text{ in.})t_p$$

At the first row of bolts

$$\begin{aligned}
 A_g &= t_p l_w \\
 &= \left(\frac{3}{4} \text{ in.}\right)(17.4 \text{ in.}) \\
 &= 13.1 \text{ in.}^2 \\
 A_n &= A_g - 2(d_h + \frac{1}{16} \text{ in.})t_p \\
 &= 13.1 \text{ in.}^2 - 2\left(\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.}\right)\left(\frac{3}{4} \text{ in.}\right) \\
 &= 11.8 \text{ in.}^2
 \end{aligned}$$

and the effective net area is:

$$\begin{aligned}
 A_e &= (11.8 \text{ in.}^2)(1.0) \\
 &= 11.8 \text{ in.}^2
 \end{aligned}$$

At the plate cut out

$$\begin{aligned}
 A_g &= t_p (2w_s) \\
 &= \left(\frac{3}{4} \text{ in.}\right)(2)(4 \text{ in.}) \\
 &= 6.00 \text{ in.}^2 \\
 A_n &= A_g \\
 &= 6.00 \text{ in.}^2
 \end{aligned}$$

and the effective net area is:

$$\begin{aligned}
 A_e &= (6.00 \text{ in.}^2)(1.0) \\
 &= 6.00 \text{ in.}^2
 \end{aligned}$$

The net area at the bolts is greater than the net area at the cut out, therefore, the critical location for tensile rupture is at the cut out.

The nominal tensile rupture strength of the cut-out plate is:

$$\begin{aligned}
 R_n &= (65 \text{ ksi})(6.00 \text{ in.}^2) \\
 &= 390 \text{ kips}
 \end{aligned}$$

The available tensile rupture strength of the cut-out plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(390 \text{ kips})$ $= 293 \text{ kips} > 144 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{390 \text{ kips}}{2.00}$ $= 195 \text{ kips} > 86.2 \text{ kips} \quad \text{o.k.}$

Block shear rupture of cut-out plate

There are three cases for which block shear should be checked (see Figure 6-9).

From AISC *Specification* Section J4.3, the nominal strength of the cut-out plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

Case 1

By inspection, Case 2 will control over Case 1 because the bolt gage is smaller than the width of the plate outside the bolts.

Case 2

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= (2)[(n-1)s + l_{ev}]t_p \\ &= (2)[(5-1)(3 \text{ in.}) + 1\frac{1}{4} \text{ in.}](\frac{3}{4} \text{ in.}) \\ &= 19.9 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (n-0.5)(d_h + \frac{1}{16} \text{ in.})t_p \\ &= 19.9 \text{ in.}^2 - (2)(5-0.5)(\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})(\frac{3}{4} \text{ in.}) \\ &= 14.0 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= [g - (d_h + \frac{1}{16} \text{ in.})]t_p \\ &= [3\frac{1}{2} \text{ in.} - (\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})](\frac{3}{4} \text{ in.}) \\ &= 1.97 \text{ in.}^2 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

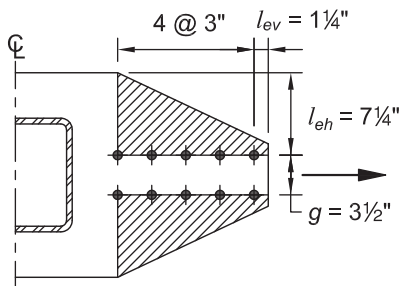
$$\begin{aligned} R_n &= 0.60(65 \text{ ksi})(14.0 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.97 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(19.9 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.97 \text{ in.}^2) \\ &= 674 \text{ kips} < 725 \text{ kips} \\ &= 674 \text{ kips} \end{aligned}$$

Therefore:

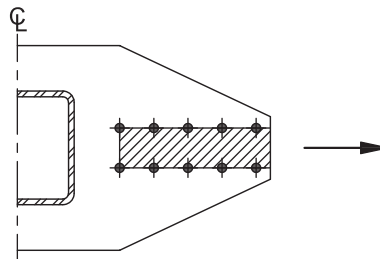
LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(674 \text{ kips})$ $= 506 \text{ kips} > 144 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{674 \text{ kips}}{2.00}$ $= 337 \text{ kips} > 86.2 \text{ kips} \quad \text{o.k.}$

Case 3

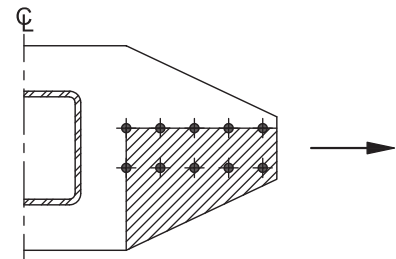
$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$



(a) Case 1



(b) Case 2



(c) Case 3

Fig. 6-9. Block shear cases.

where

$$\begin{aligned} A_{gv} &= [(n-1)s + l_{ev}]t_p \\ &= [(5-1)(3 \text{ in.}) + 1\frac{1}{4} \text{ in.}](\frac{3}{4} \text{ in.}) \\ &= 9.94 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (n-0.5)(d_h + \frac{1}{16} \text{ in.})t_p \\ &= 9.94 \text{ in.}^2 - (5-0.5)(\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})(\frac{3}{4} \text{ in.}) \\ &= 6.99 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= [g + l_{eh} - 1.5(d_h + \frac{1}{16} \text{ in.})]t_p \\ &= [3\frac{1}{2} \text{ in.} + 7.25 \text{ in.} - 1.5(\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})](\frac{3}{4} \text{ in.}) \\ &= 7.08 \text{ in.}^2 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned} R_n &= 0.60(65 \text{ ksi})(6.99 \text{ in.}^2) + 1.0(65 \text{ ksi})(7.08 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(9.94 \text{ in.}^2) + 1.0(65 \text{ ksi})(7.08 \text{ in.}^2) \\ &= 733 \text{ kips} < 758 \text{ kips} \\ &= 733 \text{ kips} \end{aligned}$$

Therefore:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(733 \text{ kips})$ $= 550 \text{ kips} > 144 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{733 \text{ kips}}{2.00}$ $= 367 \text{ kips} > 86.2 \text{ kips} \quad \text{o.k.}$

Block shear rupture of the beam flange

Block shear rupture of the beam flange will follow the pattern shown in Figure 6-10. From AISC *Specification* Section J4.3, the nominal strength of the beam flange based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= (2)[(n-1)s + l_e]t_f \\ &= (2)[(5-1)(3 \text{ in.}) + 3\frac{1}{2} \text{ in.}](0.425 \text{ in.}) \\ &= 13.2 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2)(n-0.5)(d_h + \frac{1}{16} \text{ in.})t_f \\ &= 13.2 \text{ in.}^2 - 2(5-0.5)(\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})(0.425 \text{ in.}) \\ &= 9.85 \text{ in.}^2 \end{aligned}$$

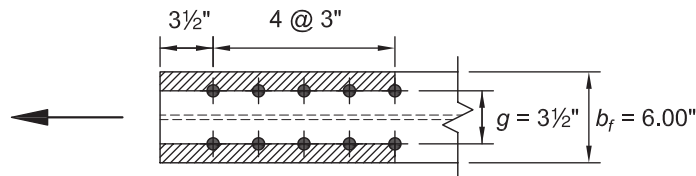


Fig. 6-10. Block shear in beam flange.

$$\begin{aligned}
 A_{nt} &= [b_f - g - (d_h + \frac{1}{16} \text{ in.})] t_f \\
 &= [6.00 \text{ in.} - 3\frac{1}{2} \text{ in.} - (\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})] (0.425 \text{ in.}) \\
 &= 0.691 \text{ in.}^2
 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned}
 R_n &= 0.60(65 \text{ ksi})(9.85 \text{ in.}^2) + 1.0(65 \text{ ksi})(0.691 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(13.2 \text{ in.}^2) + 1.0(65 \text{ ksi})(0.691 \text{ in.}^2) \\
 &= 429 \text{ kips} < 441 \text{ kips} \\
 &= 429 \text{ kips}
 \end{aligned}$$

Therefore:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(429 \text{ kips})$ $= 322 \text{ kips} > 144 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{429 \text{ kips}}{2.00}$ $= 215 \text{ kips} > 86.2 \text{ kips} \quad \text{o.k.}$

EXAMPLE 6.3—Directly Welded Connection

Given:

Determine the available flexural strength of the directly welded connection shown in Figure 6-11. Use an ASTM A992/A992M W16×57 beam and an ASTM A500/A500M Grade C HSS10×10×½ column.

From AISC *Manual* Table 2-4, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$F_y = 50 \text{ ksi}$

$F_u = 62 \text{ ksi}$

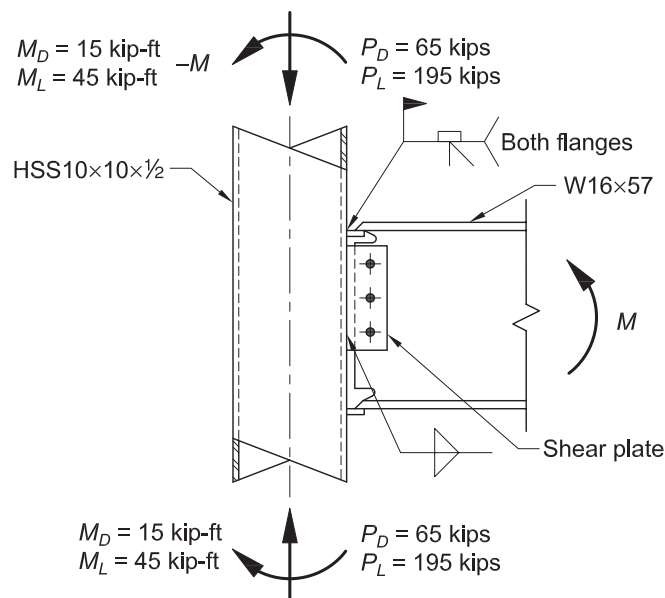


Fig. 6-11. Directly welded moment connection for Example 6.3.

Beam
 ASTM A992/A992M
 $F_y = 50$ ksi
 $F_u = 65$ ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

Column
 HSS10×10×½
 $H = 10.0$ in.
 $B = 10.0$ in.
 $t = 0.465$ in.
 $A = 17.2$ in.²
 $S = 51.2$ in.³
 Workable flat = 7¾ in.

Beam
 W16×57
 $d = 16.4$ in.
 $b_f = 7.12$ in.
 $t_f = 0.715$ in.

Solution:

Compatibility between the beam flange width and the width of the HSS

It is recommended when directly welding a beam flange to an HSS column that the beam flange width be as close to the flat dimension of the HSS as possible. If the beam flange width exceeds the width of the HSS, the beam flange does not need to be trimmed to the HSS width; however, the maximum flange width that may be used for the following calculations is the width of the HSS—that is, $\beta = 1$.

Flat of HSS

Compare the workable flat of the column to the width of the beam flange:

$$7\frac{3}{4} \text{ in.} > b_f = 7.12 \text{ in.}$$

Therefore, the flange will be welded only to the flat of the HSS.

Local yielding of the beam flange

From AISC *Specification* Section J4.1(a), the nominal yield strength of the beam flange is:

$$R_n = F_y A_g \quad (\text{Spec. Eq. J4-1})$$

where

$$A_g = B_e t_p$$

$$B_e = \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b \quad (\text{Spec. Eq. K1-1})$$

In this case, F_{yb} is the minimum yield strength of the beam, t_b is the thickness of the beam flange, and B_b is the width of the beam flange.

$$B_e = \left[\frac{10}{(10 \text{ in.}/0.465 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.465 \text{ in.})}{(50 \text{ ksi})(0.715 \text{ in.})} \right] (7.12 \text{ in.}) < 7.12 \text{ in.}$$

$$= 2.15 \text{ in.} < 7.12 \text{ in.}$$

$$= 2.15 \text{ in.}$$

Therefore

$$\begin{aligned}
 R_n &= F_y B_e t_f \\
 &= (50 \text{ ksi})(2.15 \text{ in.})(0.715 \text{ in.}) \\
 &= 76.9 \text{ kips}
 \end{aligned}$$

The available yielding strength of the beam flange is:

LRFD	ASD
$\phi = 0.95$ $\phi R_n = 0.95(76.9 \text{ kips})$ $= 73.1 \text{ kips}$	$\Omega = 1.58$ $\frac{R_n}{\Omega} = \frac{76.9 \text{ kips}}{1.58}$ $= 48.7 \text{ kips}$

Local yielding of HSS sidewalls

From AISC *Specification* Section J10.2, the nominal local yielding strength of the sidewalls (webs) is:

$$R_n = F_y t_w (5k + l_b) \quad (\text{Spec. Eq. J10-2})$$

where

$$\begin{aligned}
 t_w &= 2t \\
 l_b &= t_f \\
 k &= 1.5t
 \end{aligned}$$

Therefore,

$$\begin{aligned}
 R_n &= 2F_y t [5(1.5t) + t_f] \\
 &= (2 \text{ walls})(50 \text{ ksi})(0.465 \text{ in.})[5(1.5)(0.465 \text{ in.}) + 0.715 \text{ in.}] \\
 &= 195 \text{ kips}
 \end{aligned}$$

The available local yielding strength of the HSS sidewalls is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(195 \text{ kips})$ $= 195 \text{ kips}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{195 \text{ kips}}{1.50}$ $= 130 \text{ kips}$

Plastification of connecting face

From AISC *Specification* Sections J10.10, J4.5, and AISC *Manual* Part 9, the flexural strength of the connecting face is:

$$R_n = \frac{t^2 F_y}{2} \left[\frac{(a+b) \left(4 \sqrt{\frac{wab}{a+b}} + l \right)}{ab} \right] Q_f \quad (\text{Manual Eq. 9-44})$$

Referring to AISC *Manual* Figure 9-5:

$$\begin{aligned}
 a &= b \\
 &= \frac{B - b_f}{2} \\
 &= \frac{10.0 \text{ in.} - 7.12 \text{ in.}}{2} \\
 &= 1.44 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 l &= t_f \\
 &= 0.715 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 w &= B \\
 &= 10.0 \text{ in.}
 \end{aligned}$$

and

$$Q_f = 1.3 - 0.4 \left(\frac{U}{\beta} \right) \leq 1.0 \quad (\text{Spec. Eq. K1-4})$$

where

$$\begin{aligned}
 \beta &= \frac{B_b}{B} \\
 &= \frac{b_f}{B} \\
 &= \frac{7.12 \text{ in.}}{10.0 \text{ in.}} \\
 &= 0.712
 \end{aligned}$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0$$

LRFD	ASD
$ \begin{aligned} P_{ro} &= P_u \\ &= 1.2P_D + 1.6P_L \\ &= 1.2(65 \text{ kips}) + 1.6(195 \text{ kips}) \\ &= 390 \text{ kips} \\ M_{ro} &= M_u \\ &= 1.2M_D + 1.6M_L \\ &= 1.2(15 \text{ kip-ft}) + 1.6(45 \text{ kip-ft}) \\ &= 90.0 \text{ kip-ft} \\ F_c &= F_y \\ &= 50 \text{ ksi} \end{aligned} $	$ \begin{aligned} P_{ro} &= P_a \\ &= P_D + P_L \\ &= 65 \text{ kips} + 195 \text{ kips} \\ &= 260 \text{ kips} \\ M_{ro} &= M_a \\ &= M_D + M_L \\ &= 15 \text{ kip-ft} + 45 \text{ kip-ft} \\ &= 60.0 \text{ kip-ft} \\ F_c &= 0.6F_y \\ &= 0.6(50 \text{ ksi}) \\ &= 30.0 \text{ ksi} \end{aligned} $
$ \begin{aligned} U &= \left \frac{390 \text{ kips}}{(50 \text{ ksi})(17.2 \text{ in.}^2)} + \frac{(90.0 \text{ kip-ft})(12 \text{ in./ft})}{(50 \text{ ksi})(51.2 \text{ in.}^3)} \right \\ &= 0.875 < 1.0 \\ Q_f &= 1.3 - 0.4 \left(\frac{0.875}{0.712} \right) \\ &= 0.808 < 1.0 \end{aligned} $	$ \begin{aligned} U &= \left \frac{260 \text{ kips}}{(30.0 \text{ ksi})(17.2 \text{ in.}^2)} + \frac{(60.0 \text{ kip-ft})(12 \text{ in./ft})}{(30.0 \text{ ksi})(51.2 \text{ in.}^3)} \right \\ &= 0.973 < 1.0 \\ Q_f &= 1.3 - 0.4 \left(\frac{0.973}{0.712} \right) \\ &= 0.753 < 1.0 \end{aligned} $

Therefore,

LRFD	ASD
$R_n = \frac{t^2 F_y}{2} \left[\frac{(a+b) \left(4 \sqrt{\frac{wab}{a+b}} + l \right)}{ab} \right] Q_f$ $= \frac{(0.465 \text{ in.})^2 (50 \text{ ksi})}{2}$ $\times \left\{ \frac{(1.44 \text{ in.} + 1.44 \text{ in.})}{(1.44 \text{ in.})(1.44 \text{ in.})} \right.$ $\times \left. \left[\frac{4 \sqrt{\frac{(10.0 \text{ in.})(1.44 \text{ in.})^2}{2(1.44 \text{ in.})}} + 0.715 \text{ in.}}{(1.44 \text{ in.})(1.44 \text{ in.})} \right] \right\} (0.808)$ $= 69.4 \text{ kips}$	$R_n = \frac{t^2 F_y}{2} \left[\frac{(a+b) \left(4 \sqrt{\frac{wab}{a+b}} + l \right)}{ab} \right] Q_f$ $= \frac{(0.465 \text{ in.})^2 (50 \text{ ksi})}{2}$ $\times \left\{ \frac{(1.44 \text{ in.} + 1.44 \text{ in.})}{(1.44 \text{ in.})(1.44 \text{ in.})} \right.$ $\times \left. \left[\frac{4 \sqrt{\frac{(10.0 \text{ in.})(1.44 \text{ in.})^2}{2(1.44 \text{ in.})}} + 0.715 \text{ in.}}{(1.44 \text{ in.})(1.44 \text{ in.})} \right] \right\} (0.753)$ $= 64.7 \text{ kips}$

The flexural strength of the HSS connecting face is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(69.4 \text{ kips})$ $= 69.4 \text{ kips}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{64.7 \text{ kips}}{1.50}$ $= 43.1 \text{ kips}$

Shear yielding (punching shear) of connecting face

From AISC Specification Section J10.10 and Manual Part 9, the punching shear strength of the connecting face is:

$$R_n = 0.6 F_y t_p (2c_{eff} + 2l) \quad (\text{Manual Eq. 9-43})$$

where

$$c_{eff} = B_{ep} \quad (\text{from Spec. Eq. K1-2})$$

$$= \left(\frac{10}{B/t} \right) B_b \leq B_b$$

$$= \left[\frac{10}{(10 \text{ in.})/0.465 \text{ in.}} \right] (7.12 \text{ in.}) < 7.12 \text{ in.}$$

$$= 3.31 \text{ in.}$$

$$l = t_f$$

$$= 0.715 \text{ in.}$$

$$t_p = t$$

$$= 0.465 \text{ in.}$$

Therefore,

$$R_n = 0.6 F_y t (2c_{eff} + 2t_f)$$

$$= 0.6(50 \text{ ksi})(0.465 \text{ in.})[2(3.31 \text{ in.}) + 2(0.715 \text{ in.})]$$

$$= 112 \text{ kips}$$

The available punching shear strength of the HSS connecting face is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(112 \text{ kips})$ $= 112 \text{ kips}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{112 \text{ kips}}{1.50}$ $= 74.7 \text{ kips}$

HSS wall local crippling

From AISC *Specification* Section J10.3, the nominal compressive strength based on the limit state of wall local crippling, for one wall, is:

$$R_n = \frac{1.6t^2 \left(1 + \frac{3l_b}{H} \right) \sqrt{EF_y} Q_f}{\sin \theta} \quad (\text{from Eq. 2-19})$$

where

$$l_b = t_f$$

$$= 0.715 \text{ in.}$$

$$\theta = 90^\circ$$

The nominal compressive strength due to wall local crippling is:

LRFD	ASD
$R_n = \frac{\left\{ 1.6(0.465 \text{ in.})^2 \left[1 + \frac{3(0.715 \text{ in.})}{10.0 \text{ in.}} \right] \right\} \times \sqrt{(29,000 \text{ ksi})(50 \text{ ksi})(0.808)}}}{\sin 90^\circ}$ $= 409 \text{ kips}$	$R_n = \frac{\left\{ 1.6(0.465 \text{ in.})^2 \left[1 + \frac{3(0.715 \text{ in.})}{10.0 \text{ in.}} \right] \right\} \times \sqrt{(29,000 \text{ ksi})(50 \text{ ksi})(0.753)}}}{\sin 90^\circ}$ $= 381 \text{ kips}$

The available compressive strength of the HSS for the limit state of wall local crippling is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(409 \text{ kips})$ $= 307 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{381 \text{ kips}}{2.00}$ $= 191 \text{ kips}$

The available strength is controlled by the flexural strength of the HSS connecting face.

LRFD	ASD
$\phi R_n = 69.4 \text{ kips}$	$\frac{R_n}{\Omega} = 43.1 \text{ kips}$

Available flexural strength

Determine the flexural strength as the flange force, F_f , multiplied by the distance between the flange centers:

$$\begin{aligned}
 M &= F_f(d - t_f) \\
 &= F_f(16.4 \text{ in.} - 0.715 \text{ in.}) \\
 &= F_f(15.7 \text{ in.})
 \end{aligned}$$

The available flexural strength is determined as follows:

LRFD	ASD
$F_f = \phi R_n$ $= 69.4 \text{ kips}$ $\phi M_n = (69.4 \text{ kips})(15.7 \text{ in.}) \left(\frac{1 \text{ ft}}{12 \text{ in.}} \right)$ $= 90.8 \text{ kip-ft}$	$F_f = \frac{R_n}{\Omega}$ $= 43.1 \text{ kips}$ $\frac{M_n}{\Omega} = (43.1 \text{ kips})(15.7 \text{ in.}) \left(\frac{1 \text{ ft}}{12 \text{ in.}} \right)$ $= 56.4 \text{ kip-ft}$

EXAMPLE 6.4—Through-Plate Flange-Plated Connection

Given:

Design a through-plate flange-plated connection between ASTM A992/A992M W18×71 beams and an ASTM A500/A500M Grade C HSS20×12×½ column, as shown in Figure 6-12. Use 1-in.-diameter Group 120 bolts with threads not excluded from the shear plane (thread condition N) in standard holes and 70 ksi filler metal. The applied dead and live loads are shown in Figure 6-12. The loads on the two beams can be reversed; therefore, the connection should be symmetric.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$F_y = 50 \text{ ksi}$

$F_u = 62 \text{ ksi}$

Beam

ASTM A992/A992M

$F_y = 50 \text{ ksi}$

$F_u = 65 \text{ ksi}$

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50 \text{ ksi}$

$F_{up} = 65 \text{ ksi}$

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

Column

HSS20×12×½

$H = 20.0 \text{ in.}$

$B = 12.0 \text{ in.}$

$t = 0.465 \text{ in.}$

Beam

W18×71

$d = 18.5 \text{ in.}$

$b_f = 7.64 \text{ in.}$

$t_f = 0.810 \text{ in.}$

$g = 3\frac{1}{2} \text{ in. (workable gage)}$

Solution:

From Chapter 2 of ASCE/SEI 7, the required strengths are:

LRFD	ASD
$V_{u\ left} = 1.2(8\ \text{kips})$ $= 9.60\ \text{kips}$	$V_{a\ left} = 8\ \text{kips}$
$V_{u\ right} = 1.2(8\ \text{kips}) + 1.6(24\ \text{kips})$ $= 48.0\ \text{kips}$	$V_{a\ right} = 8\ \text{kips} + 24\ \text{kips}$ $= 32.0\ \text{kips}$
$M_{u\ left} = 1.2(60\ \text{kip-ft})$ $= 72.0\ \text{kip-ft}$	$M_{a\ left} = 60.0\ \text{kip-ft}$
$M_{u\ right} = 1.2(60\ \text{kip-ft}) + 1.6(180\ \text{kip-ft})$ $= 360\ \text{kip-ft}$	$M_{a\ right} = 60\ \text{kip-ft} + 180\ \text{kip-ft}$ $= 240\ \text{kip-ft}$
$P_{u\ top} = 1.2(48\ \text{kips}) + 1.6(144\ \text{kips})$ $= 288\ \text{kips}$	$P_{a\ top} = 48\ \text{kips} + 144\ \text{kips}$ $= 192\ \text{kips}$

The required axial and flexural strength for connection equilibrium:

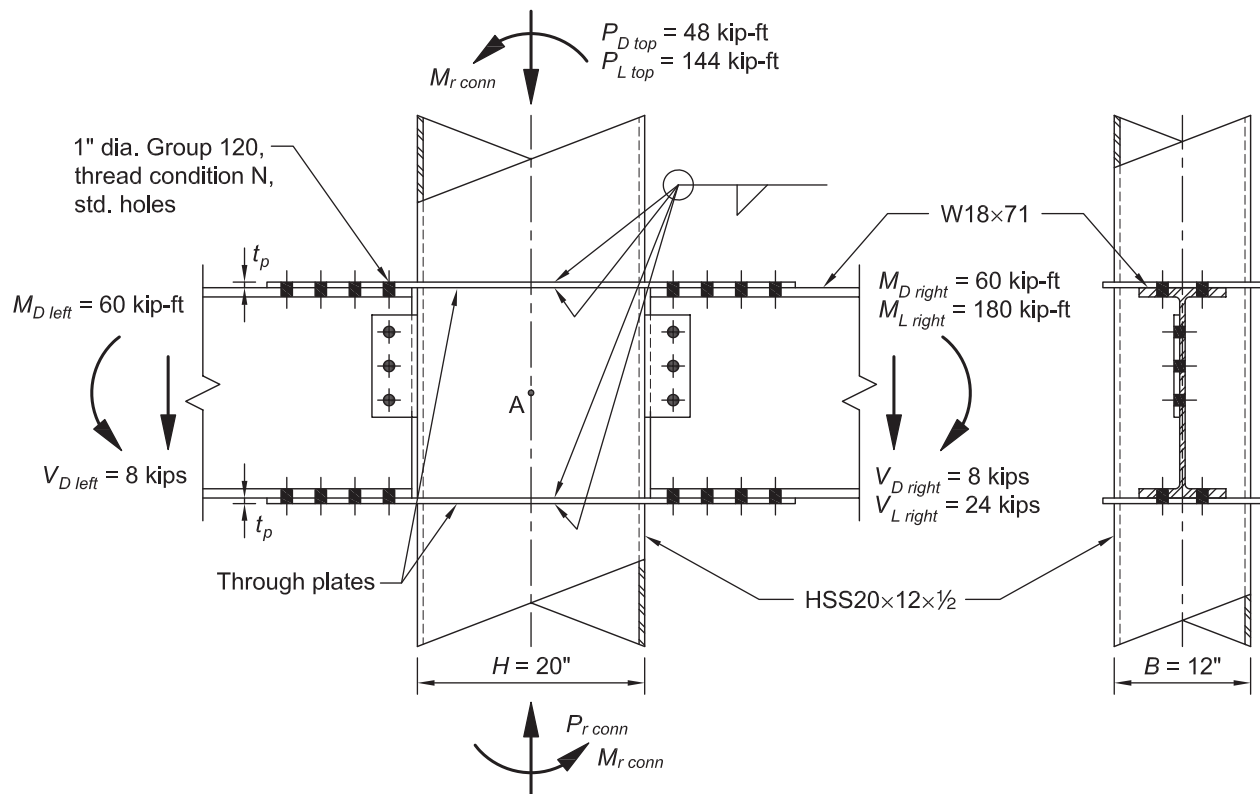


Fig. 6-12. Through-plate flange-plated connection for Example 6.4.

LRFD	ASD
$\begin{aligned}\sum F_v &= V_{u \text{ left}} + V_{u \text{ right}} + P_{u \text{ top}} - P_{u \text{ conn}} \\ &= 9.60 \text{ kips} + 48.0 \text{ kips} + 288 \text{ kips} - P_{u \text{ conn}} \\ &= 0 \\ P_{u \text{ conn}} &= 346 \text{ kips} \\ \sum M_A &= M_{u \text{ right}} - M_{u \text{ left}} - 2M_{u \text{ conn}} \\ &\quad + \frac{H}{2}(V_{u \text{ right}} - V_{u \text{ left}}) \\ &= 360 \text{ kip-ft} - 72.0 \text{ kip-ft} - 2M_{u \text{ conn}} \\ &\quad + \left(\frac{20.0 \text{ in.}}{2}\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)(48.0 \text{ kips} - 9.60 \text{ kips}) \\ &= 0 \\ M_{u \text{ conn}} &= 160 \text{ kip-ft}\end{aligned}$	$\begin{aligned}\sum F_v &= V_{a \text{ left}} + V_{a \text{ right}} + P_{a \text{ top}} - P_{a \text{ conn}} \\ &= 8.00 \text{ kips} + 32.0 \text{ kips} + 192 \text{ kips} - P_{a \text{ conn}} \\ &= 0 \\ P_{a \text{ conn}} &= 232 \text{ kips} \\ \sum M_A &= M_{a \text{ right}} - M_{a \text{ left}} - 2M_{a \text{ conn}} \\ &\quad + \frac{H}{2}(V_{a \text{ right}} - V_{a \text{ left}}) \\ &= 240 \text{ kip-ft} - 60.0 \text{ kip-ft} - 2M_{a \text{ conn}} \\ &\quad + \left(\frac{20.0 \text{ in.}}{2}\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)(32.0 \text{ kips} - 8.00 \text{ kips}) \\ &= 0 \\ M_{a \text{ conn}} &= 100 \text{ kip-ft}\end{aligned}$

Use a plate width of 14 in. to permit the use of a fillet weld to the HSS.

To begin, use a plate thickness equal to the flange thickness to determine the approximate required force in the plate, R_u and R_a .

LRFD	ASD
$\begin{aligned}M_{u \text{ right}} &\approx R_u(d + t_f) \\ R_u &\approx \frac{M_{u \text{ right}}}{d + t_f} \\ &\approx \frac{(360 \text{ kip-ft})(12 \text{ in./ft})}{18.5 \text{ in.} + 0.810 \text{ in.}} \\ &\approx 224 \text{ kips}\end{aligned}$	$\begin{aligned}M_{a \text{ right}} &\approx R_a(d + t_f) \\ R_a &\approx \frac{M_{a \text{ right}}}{d + t_f} \\ &\approx \frac{(240 \text{ kip-ft})(12 \text{ in./ft})}{18.5 \text{ in.} + 0.810 \text{ in.}} \\ &\approx 149 \text{ kips}\end{aligned}$

Tensile yielding of through plate

Using AISC *Specification* Section J4.1(a), determine the through-plate thickness, t_p , based on tensile yielding. The force in the plate, R_u and R_a , can then be determined.

LRFD	ASD
$\begin{aligned}\phi &= 0.90 \\ A_g &= \frac{R_u}{\phi F_y} \\ &= \frac{224 \text{ kips}}{0.90(50 \text{ ksi})} \\ &= 4.98 \text{ in.}^2\end{aligned}$	$\begin{aligned}\Omega &= 1.67 \\ A_g &= \frac{R_a}{F_y/\Omega} \\ &= \frac{149 \text{ kips}}{(50 \text{ ksi})/1.67} \\ &= 4.98 \text{ in.}^2\end{aligned}$

$$\begin{aligned}t_p &\approx \frac{A_g}{b_p} \\ &\approx \frac{4.98 \text{ in.}^2}{14 \text{ in.}} \\ &\approx 0.356 \text{ in.}\end{aligned}$$

Use $t_p = 1/2 \text{ in.}$

The required force is recalculated using the actual through-plate thickness:

LRFD	ASD
$M_{u \text{ right}} = R_u(d + t_p)$ $R_u = \frac{M_{u \text{ right}}}{d + t_p}$ $= \frac{(360 \text{ kip-ft})(12 \text{ in./ft})}{18.5 \text{ in.} + \frac{1}{2} \text{ in.}}$ $= 227 \text{ kips}$	$M_{a \text{ right}} = R_a(d + t_p)$ $R_a = \frac{M_{a \text{ right}}}{d + t_p}$ $= \frac{(240 \text{ kip-ft})(12 \text{ in./ft})}{18.5 \text{ in.} + \frac{1}{2} \text{ in.}}$ $= 152 \text{ kips}$

Tensile rupture of through plate

Using the through-plate thickness determined previously, check the limit state of tensile rupture of the through plate. Note that the plate is treated as a splice plate. From AISC *Specification* Section J4.1(b), the nominal tensile rupture strength of the through plate is:

$$R_n = F_u A_e \quad (\text{Spec. Eq. J4-2})$$

The effective area, A_e , is determined from AISC *Specification* Section D3.3 as follows:

$$A_e = A_n U \quad (\text{Spec. Eq. D3-1})$$

$$U = 1.0 \text{ from AISC } \textit{Specification} \text{ Table D3.1 (no unconnected elements)}$$

The net tensile area is:

$$A_n = A_g - 2(d_h + \frac{1}{16} \text{ in.})t_p$$

From AISC *Specification* Table J3.3, $d_h = 1\frac{1}{8} \text{ in.}$

$$\begin{aligned} A_g &= b_p t_p \\ &= (14 \text{ in.})(\frac{1}{2} \text{ in.}) \\ &= 7.00 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_n &= 7.00 \text{ in.}^2 - 2(1\frac{1}{8} \text{ in.} + \frac{1}{16} \text{ in.})(\frac{1}{2} \text{ in.}) \\ &= 5.81 \text{ in.}^2 \end{aligned}$$

and the effective net area is:

$$\begin{aligned} A_e &= (5.81 \text{ in.}^2)(1.0) \\ &= 5.81 \text{ in.}^2 \end{aligned}$$

The nominal tensile rupture strength of the through plate is:

$$\begin{aligned} R_n &= (65 \text{ ksi})(5.81 \text{ in.}^2) \\ &= 378 \text{ kips} \end{aligned}$$

The available tensile rupture strength of the through plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(378 \text{ kips})$ $= 284 \text{ kips} > 227 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{378 \text{ kips}}{2.00}$ $= 189 \text{ kips} > 152 \text{ kips} \quad \mathbf{o.k.}$

Compressive yielding and buckling of through plate

Calculate the radius of gyration of the plate:

$$\begin{aligned}
 r &= \frac{t_p}{\sqrt{12}} \\
 &= \frac{1/2 \text{ in.}}{\sqrt{12}} \\
 &= 0.144 \text{ in.}
 \end{aligned}$$

Calculate the nominal compressive strength, P_n , from AISC *Specification* Section J4.4. The distance from the first bolt to the face of the HSS is chosen as $L = 3 \text{ in.}$ Therefore:

$$\begin{aligned}
 \frac{KL}{r} &= \frac{1.0(3 \text{ in.})}{0.144 \text{ in.}} \\
 &= 20.8 < 25
 \end{aligned}$$

Because $KL/r \leq 25$, use $P_n = F_y A_g$. This check is the same as the tension yielding check done previously.

Use PL $\frac{1}{2}$ ×14 for the through plates.

Strength of through plate bolted connection

Try two rows of four bolts on each side of the HSS with 3 in. spacing, 3 in. from the HSS to the first bolt, and 1 $\frac{3}{4}$ in. edge distance, which is greater than the minimum edge distance from AISC *Specification* Section J3.5.

The strength of the bolt group is taken as the sum of the strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.7, the bearing strength of the plate or flange at the bolt hole per AISC *Specification* Section J3.11, or the tearout strength of the plate or flange at the bolt hole per AISC *Specification* Section J3.11.

From AISC *Manual* Table 7-1, the available shear strength per bolt for 1-in.-diameter, Group 120 bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 31.8 \text{ kips/bolt}$	$\frac{r_n}{\Omega} = 21.2 \text{ kips/bolt}$

The available bearing strength of the plate per bolt is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration:

$$\begin{aligned}
 r_n &= 2.4 d t_p F_u && \text{(from Spec. Eq. J3-6a)} \\
 &= 2.4(1 \text{ in.})(1/2 \text{ in.})(65 \text{ ksi}) \\
 &= 78.0 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(78.0 \text{ kips/bolt})$ $= 58.5 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{78.0 \text{ kips/bolt}}{2.00}$ $= 39.0 \text{ kips/bolt}$

The available tearout strength of the plate at the interior bolts is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration.

$$\begin{aligned}
 l_c &= s - d_h \\
 &= 3 \text{ in.} - 1\frac{1}{8} \text{ in.} \\
 &= 1.88 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 r_n &= 1.2l_c t_p F_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(1.88 \text{ in.})(\frac{1}{2} \text{ in.})(65 \text{ ksi}) \\
 &= 73.3 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(73.3 \text{ kips/bolt})$ $= 55.0 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{73.3 \text{ kips/bolt}}{2.00}$ $= 36.7 \text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout of the plate at the interior bolts.

The available tearout strength of the plate at the edge bolt is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration.

$$\begin{aligned}
 l_c &= l_e - d_h/2 \\
 &= 1\frac{3}{4} \text{ in.} - (1\frac{1}{8} \text{ in.})/2 \\
 &= 1.19 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 r_n &= 1.2l_c t F_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(1.19 \text{ in.})(\frac{1}{2} \text{ in.})(65 \text{ ksi}) \\
 &= 46.4 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(46.4 \text{ kips/bolt})$ $= 34.8 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{46.4 \text{ kips/bolt}}{2.00}$ $= 23.2 \text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout of the plate at the edge bolts.

The available bearing strength of the flange per bolt is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration:

$$\begin{aligned}
 r_n &= 2.4d_t F_u && \text{(from Spec. Eq. J3-6a)} \\
 &= 2.4(1 \text{ in.})(0.810 \text{ in.})(65 \text{ ksi}) \\
 &= 126 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(126 \text{ kips/bolt})$ $= 94.5 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{126 \text{ kips/bolt}}{2.00}$ $= 63.0 \text{ kips/bolt}$

The available tearout strength of the flange at the interior bolts is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration.

$$\begin{aligned}
 l_c &= s - d_h \\
 &= 3 \text{ in.} - 1\frac{1}{8} \text{ in.} \\
 &= 1.88 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 r_n &= 1.2l_c t F_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(1.88 \text{ in.})(0.810 \text{ in.})(65 \text{ ksi}) \\
 &= 119 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(119 \text{ kips/bolt})$ $= 89.3 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{119 \text{ kips/bolt}}{2.00}$ $= 59.5 \text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout of the flange at interior bolts.

The available tearout strength of the flange at the edge bolt is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration. Assuming a 1 in. beam setback from the face of the column, the edge distance is $l_e = 2$ in.

$$\begin{aligned}
 l_c &= l_e - \frac{d_h}{2} \\
 &= 2 \text{ in.} - \frac{1\frac{1}{8} \text{ in.}}{2} \\
 &= 1.44 \text{ in.}
 \end{aligned}$$

$$\begin{aligned}
 r_n &= 1.2l_c t F_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(1.44 \text{ in.})(0.810 \text{ in.})(65 \text{ ksi}) \\
 &= 91.0 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(91.0 \text{ kips/bolt})$ $= 68.3 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{91.0 \text{ kips/bolt}}{2.00}$ $= 45.5 \text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout of the flange at edge bolts.

The strength of the bolt group is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = \phi r_n n$ $= (31.8 \text{ kips/bolt})(8 \text{ bolts})$ $= 254 \text{ kips} > 227 \text{ kips} \quad \text{o.k.}$	$\frac{R_n}{\Omega} = \frac{r_n}{\Omega} n$ $= (21.2 \text{ kips/bolt})(8 \text{ bolts})$ $= 170 \text{ kips} > 152 \text{ kips} \quad \text{o.k.}$

Block shear rupture of the through plate

The controlling failure surface for the block shear rupture limit state is the shear area through the two lines of bolts and the tensile area between the two bolts closest to the HSS. From AISC *Specification* Section J4.3, the nominal strength of the through plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$A_{gv} = 2[l_e + (n-1)s]t_p$$

$$= 2[1\frac{3}{4} \text{ in.} + (4-1)(3 \text{ in.})](\frac{1}{2} \text{ in.})$$

$$= 10.8 \text{ in.}^2$$

$$A_{nv} = A_{gv} - 2(n-0.5)(d_h + \frac{1}{16} \text{ in.})t_p$$

$$= 10.8 \text{ in.}^2 - 2(4-0.5)(1\frac{1}{8} \text{ in.} + \frac{1}{16} \text{ in.})(\frac{1}{2} \text{ in.})$$

$$= 6.64 \text{ in.}^2$$

$$A_{nt} = [g - (d_h + \frac{1}{16} \text{ in.})]t_p$$

$$= [3\frac{1}{2} \text{ in.} - (1\frac{1}{8} \text{ in.} + \frac{1}{16} \text{ in.})](\frac{1}{2} \text{ in.})$$

$$= 1.16 \text{ in.}^2$$

$$U_{bs} = 1.0 \text{ because tension is uniform}$$

$$R_n = 0.60(65 \text{ ksi})(6.64 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.16 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(10.8 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.16 \text{ in.}^2)$$

$$= 334 \text{ kips} < 399 \text{ kips}$$

$$= 334 \text{ kips}$$

The available strength of the gusset plate for the limit state of block shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(334 \text{ kips})$ $= 251 \text{ kips} > 227 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{334 \text{ kips}}{2.00}$ $= 167 \text{ kips} > 152 \text{ kips} \quad \text{o.k.}$

Block shear rupture of the beam flange

The block shear pattern is shear through the lines of bolts and tension from the last bolt to the tips of the flange on both sides. From AISC *Specification* Section J4.3, the nominal strength of the through plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= 2[l_e + (n-1)s]t_f \\ &= 2[2 \text{ in.} + (4-1)(3 \text{ in.})](0.810 \text{ in.}) \\ &= 17.8 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - 2(n-0.5)(d_h + 1/16 \text{ in.})t_f \\ &= 17.8 \text{ in.}^2 - 2(4-0.5)(1 1/8 \text{ in.} + 1/16 \text{ in.})(0.810 \text{ in.}) \\ &= 11.1 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= [b_f - g - (d_h + 1/16 \text{ in.})]t_f \\ &= [7.64 \text{ in.} - 3 1/2 \text{ in.} - (1 1/8 \text{ in.} + 1/16 \text{ in.})](0.810 \text{ in.}) \\ &= 2.39 \text{ in.}^2 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned} R_n &= 0.60(65 \text{ ksi})(11.1 \text{ in.}^2) + 1.0(65 \text{ ksi})(2.39 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(17.8 \text{ in.}^2) + 1.0(65 \text{ ksi})(2.39 \text{ in.}^2) \\ &= 588 \text{ kips} < 689 \text{ kips} \\ &= 588 \text{ kips} \end{aligned}$$

The available strength of the gusset plate for the limit state of block shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(588 \text{ kips})$ $= 441 \text{ kips} > 227 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{588 \text{ kips}}{2.00}$ $= 294 \text{ kips} > 152 \text{ kips} \quad \mathbf{o.k.}$

Weld size for plate-to-HSS connection

Design the plate-to-HSS weld for combined axial force and moment on the compression side where they are additive.

$$\begin{aligned} A_w &= 2(B+H)t_{weld} \\ &= 2(12.0 \text{ in.} + 20.0 \text{ in.})t_{weld} \\ &= 64.0t_{weld} \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} I &= 2\left(\frac{H^3 t_{weld}}{12}\right) + 2Bt_{weld}\left(\frac{H}{2}\right)^2 \\ &= 2\left[\frac{(20.0 \text{ in.})^3 t_{weld}}{12}\right] + 2(12.0 \text{ in.})t_{weld}\left(\frac{20.0 \text{ in.}}{2}\right)^2 \\ &= 3,730t_{weld} \text{ in.}^4 \end{aligned}$$

The throat thickness is neglected for simplicity in the squared portion $(H/2)^2$ of the second term in the calculation of I , as its effect would be negligible.

$$\text{Required resistance per unit length of weld} = \frac{P_{r \text{ conn}}}{A_w/t_{weld}} + \frac{M_{r \text{ conn}}(H/2)}{I/t_{weld}}$$

Using the procedure provided in Part 8 of the AISC *Manual*, determine the weld size required to connect the plate to the HSS.

LRFD	ASD
$\phi R_n = 1.392 D l$ (Manual Eq. 8-2a) $\frac{\phi R_n}{l} = 1.392 D$ $\geq \frac{P_{u \text{ conn}}}{A_w/t_{\text{weld}}} + \frac{M_{u \text{ conn}}(H/2)}{I/t_{\text{weld}}}$ $\geq \frac{346 \text{ kips}}{64.0 \text{ in.}} + \frac{(160 \text{ kip-ft})(12 \text{ in./ft})(20.0 \text{ in./2})}{3,730 \text{ in.}^3}$ $\geq 10.6 \text{ kips/in.}$ $D \geq 7.58 \text{ sixteenths}$	$\frac{R_n}{\Omega} = 0.928 D l$ (Manual Eq. 8-2b) $\frac{R_n}{\Omega l} = 0.928 D$ $\geq \frac{P_{a \text{ conn}}}{A_w/t_{\text{weld}}} + \frac{M_{a \text{ conn}}(H/2)}{I/t_{\text{weld}}}$ $\geq \frac{232 \text{ kips}}{64.0 \text{ in.}} + \frac{(100 \text{ kip-ft})(12 \text{ in./ft})(20.0 \text{ in./2})}{3,730 \text{ in.}^3}$ $\geq 6.84 \text{ kips/in.}$ $D \geq 7.37 \text{ sixteenths}$

Use a 1/2 in. fillet weld.

Check shear rupture in HSS wall at welds

$$t_{\min} = \frac{3.09 D}{F_u} \quad (\text{Manual Eq. 9-6})$$

$$= \frac{3.09(8 \text{ sixteenths})}{62 \text{ ksi}}$$

$$= 0.399 \text{ in.} < 0.465 \text{ in.} \quad \text{o.k.}$$

EXAMPLE 6.5—Welded WT Flange Connection

Given:

Verify the capacity of the WT flange connection as shown in Figure 6-13 for the dead and live load moments of $M_D = 20$ kip-ft and $M_L = 60$ kip-ft. Use a 10-in.-long WT8×28.5 top and bottom to resist the applied moments. The HSS is ASTM A500/A500M Grade C, and the wide flange and WT sections are ASTM A992/A992M. Use 7/8-in.-diameter Group 120 bolts with threads not excluded from the shear plate (thread condition N) in standard holes and 70 ksi filler metal. The column carries dead and live axial loads of $P_D = 33.3$ kips and $P_L = 100$ kips.

From AISC Manual Table 2-4, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

Beam and WT

ASTM A992/A992M

$F_y = 50$ ksi

$F_u = 65$ ksi

From AISC Manual Tables 1-1, 1-8, and 1-12, the geometric properties are as follows:

Column

HSS12×8×1/2

$H = 12.0$ in.

$B = 8.00$ in.

$t = 0.465$ in.

$A = 17.2$ in.²

$S = 55.6$ in.³

Beam

W16×36

$d = 15.9$ in.

$b_f = 6.99$ in.

$t_f = 0.430$ in.

WT

WT8×28.5

$d = 8.22$ in.

$t_w = 0.430$ in.

$b_f = 7.12$ in.

$t_f = 0.715$ in.

$k_{des} = 1.12$ in.

$k_1 = 7/8$ in. (for a W16×57)

Solution:

From ASCE/SEI 7, Chapter 2, the required strengths are:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(33.3 \text{ kips}) + 1.6(100 \text{ kips})$ $= 200 \text{ kips}$ $M_u = 1.2M_D + 1.6M_L$ $= 1.2(20 \text{ kip-ft}) + 1.6(60 \text{ kip-ft})$ $= 120 \text{ kip-ft}$	$P_a = P_D + P_L$ $= 33.3 \text{ kips} + 100 \text{ kips}$ $= 133 \text{ kips}$ $M_a = M_D + M_L$ $= 20 \text{ kip-ft} + 60 \text{ kip-ft}$ $= 80.0 \text{ kip-ft}$

Determine the required tension and compression forces in the WT's.

LRFD	ASD
$M_u = R_u(d + t_w)$ $R_u = \frac{M_u}{d + t_w}$ $= \frac{(120 \text{ kip-ft})(12 \text{ in./ft})}{15.9 \text{ in.} + 0.430 \text{ in.}}$ $= 88.2 \text{ kips}$	$M_a = R_a(d + t_w)$ $R_a = \frac{M_a}{d + t_w}$ $= \frac{(80 \text{ kip-ft})(12 \text{ in./ft})}{15.9 \text{ in.} + 0.430 \text{ in.}}$ $= 58.8 \text{ kips}$

WT flange local bending

From AISC *Specification* Section J10.1, the nominal flange local bending strength is determined by:

$$R_n = 6.25F_{yf}t_f^2 \quad (\text{Spec. Eq. J10-1})$$

However, the AISC *Specification* further states that when the concentrated force to be resisted is applied less than $10t_f$ from the end of the member, R_n should be reduced by 50%, which is the case for these WT's.

$$\begin{aligned} R_n &= 6.25(50 \text{ ksi})(0.715 \text{ in.})^2 \\ &= 160 \text{ kips} \end{aligned}$$

Reduced by 50% results in $R_n = 80.0$ kips per web, or in this case, per HSS sidewall for a minimum WT length of:

$$\begin{aligned} 10t_f &= 10(0.715 \text{ in.}) \\ &= 7.15 \text{ in.} \end{aligned}$$

Utilizing both side walls results in $R_n = 160$ kips for a WT length of:

$$\begin{aligned} 2(10t_f) &= 2(7.15 \text{ in.}) \\ &= 14.3 \text{ in.} > 10 \text{ in.} \end{aligned}$$

Because the WT is only 10 in. long, reduce R_n accordingly (AISC, 1997b).

$$\begin{aligned} R_n &= (160 \text{ kips}) \left(\frac{10 \text{ in.}}{14.3 \text{ in.}} \right) \\ &= 112 \text{ kips} \end{aligned}$$

The available strength of the WT flange for the limit state of local bending is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(112 \text{ kips})$ $= 101 \text{ kips} > 88.2 \text{ kips} \quad \text{o.k.}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{112 \text{ kips}}{1.67}$ $= 67.1 \text{ kips} > 58.8 \text{ kips} \quad \text{o.k.}$

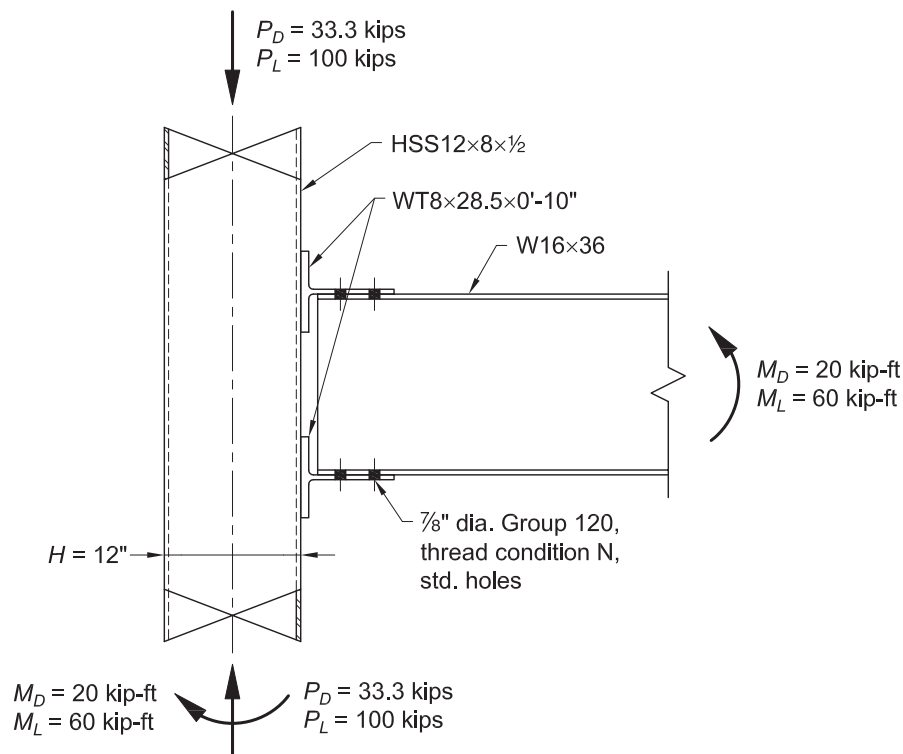


Fig. 6-13. WT connection for Example 6.5.

WT web local yielding

The load is applied within the beam depth, d , of the member end. Therefore, the nominal strength per AISC *Specification* Section J10.2 for the limit state of beam web local yielding is:

$$R_n = F_{yw}t_w(2.5k + l_b) \quad (\text{Spec. Eq. J10-3})$$

For flare-bevel groove welds between an HSS and a wide-flange section, l_b is taken as the flare-bevel groove weld size (the corner radius of the HSS) plus the leg length of the reinforcing fillet (if any).

$$\begin{aligned} l_b &= 2t \\ &= 2(0.465 \text{ in.}) \\ &= 0.930 \text{ in.} \end{aligned}$$

$$\begin{aligned} R_n &= (50 \text{ ksi})(0.430 \text{ in.})[2.5(1.12 \text{ in.}) + 0.930 \text{ in.}] \\ &= 80.2 \text{ kips/sidewall} \\ &= 160 \text{ kips for 2 sidewalls} \end{aligned}$$

The available strength of the WT web for the limit state of local yielding is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(160 \text{ kips})$ $= 160 \text{ kips} > 88.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{160 \text{ kips}}{1.50}$ $= 107 \text{ kips} > 58.8 \text{ kips} \quad \mathbf{o.k.}$

Local yielding of HSS sidewalls

From AISC *Specification* Section J10.2, the nominal tensile yield strength of the sidewalls (webs) is:

$$R_n = F_{yw}t_w(5k + l_b) \quad (\text{Spec. Eq. J10-2})$$

where

$$\begin{aligned} l_b &= 2k_1 \\ &= 2\left(\frac{7}{8} \text{ in.}\right) \\ &= 1.75 \text{ in.} \\ t_w &= t \\ &= 0.465 \text{ in.} \\ k &= 1.5t + t_f \\ &= 1.5(0.465 \text{ in.}) + 0.715 \text{ in.} \\ &= 1.41 \text{ in.} \end{aligned}$$

Therefore,

$$\begin{aligned} R_n &= F_{yw}t_w(5k + l_b) \\ &= (50 \text{ ksi})(0.465 \text{ in.})[5(1.41 \text{ in.}) + 1.75 \text{ in.}] \\ &= 205 \text{ kips/sidewall} \\ &= 410 \text{ kips for 2 sidewalls} \end{aligned}$$

The available local yielding strength of the two HSS sidewalls is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(410 \text{ kips})$ $= 410 \text{ kips} > 88.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{410 \text{ kips}}{1.50}$ $= 273 \text{ kips} > 58.8 \text{ kips} \quad \mathbf{o.k.}$

HSS wall local crippling

From AISC *Specification* Section J10.3, the nominal compressive strength based on the limit state of wall local crippling, for one wall, is given in *Specification* Eq. J10-4. As described in Chapter 2, for HSS it is given as:

$$P_n \sin \theta = 1.6t^2 \left(1 + \frac{3l_b}{H} \right) \sqrt{EF_y} Q_f \quad (2-19)$$

where

$$\begin{aligned}
 l_b &= 2k_1 \\
 &= 2(\% \text{ in.}) \\
 &= 1.75 \text{ in.}
 \end{aligned}$$

For a transverse plate connection:

$$0.4 \leq Q_f = 1.3 - 0.4 \left(\frac{U}{\beta} \right) \leq 1.0 \quad (\text{Spec. Eq. K1-4})$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{Spec. Eq. K1-6})$$

$$\begin{aligned}
 \beta &= B/H \\
 &= 8.00 \text{ in.}/12.0 \text{ in.} \\
 &= 0.667
 \end{aligned}$$

LRFD	ASD
$P_{ro} = P_u$ $= 200 \text{ kips}$ $M_{ro} = M_u$ $= 120 \text{ kip-ft}$ $F_c = F_y$ $= 50 \text{ ksi}$ $U = \left \frac{200 \text{ kips}}{(50 \text{ ksi})(17.2 \text{ in.}^2)} + \frac{(120 \text{ kip-ft})(12 \text{ in./ft})}{(50 \text{ ksi})(55.6 \text{ in.}^3)} \right \leq 1.0$ $= 0.751$ $Q_f = 1.3 - 1.4 \left(\frac{0.751}{0.667} \right) \leq 1.0$ $= 0.850$	$P_{ro} = P_a$ $= 133 \text{ kips}$ $M_{ro} = M_a$ $= 80.0 \text{ kip-ft}$ $F_c = 0.6F_y$ $= 0.6(50 \text{ ksi})$ $= 30.0 \text{ ksi}$ $U = \left \frac{133 \text{ kips}}{(30.0 \text{ ksi})(17.2 \text{ in.}^2)} + \frac{(80.0 \text{ kip-ft})(12 \text{ in./ft})}{(30.0 \text{ ksi})(55.6 \text{ in.}^3)} \right \leq 1.0$ $= 0.833$ $Q_f = 1.3 - 1.4 \left(\frac{0.833}{0.667} \right) \leq 1.0$ $= 0.800$

The nominal compressive strength due to wall local crippling is determined using Equation 2-19:

LRFD	ASD
$P_n = 1.6(0.465 \text{ in.})^2 \left[1 + \frac{3(1.75 \text{ in.})}{12 \text{ in.}} \right]$ $\times \sqrt{(29,000 \text{ ksi})(50 \text{ ksi})(0.850)/\sin 90^\circ}$ $= 509 \text{ kips/sidewall}$ $= 1,020 \text{ kips}$	$P_n = 1.6(0.465 \text{ in.})^2 \left[1 + \frac{3(1.75 \text{ in.})}{12 \text{ in.}} \right]$ $\times \sqrt{(29,000 \text{ ksi})(50 \text{ ksi})(0.800)/\sin 90^\circ}$ $= 479 \text{ kips/sidewall}$ $= 958 \text{ kips}$

The available compressive strength of the HSS for the limit state of wall local crippling is:

LRFD	ASD
$\phi = 0.75$ $\phi P_n = 0.75(1,020 \text{ kips})$ $= 765 \text{ kips} > 88.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{P_n}{\Omega} = \frac{958 \text{ kips}}{2.00}$ $= 479 \text{ kips} > 58.8 \text{ kips} \quad \mathbf{o.k.}$

Strength of WT-to-HSS weld

The flare-bevel groove weld provides a maximum effective throat of $\frac{5}{16}R$, where R is permitted to be taken as $2t$ per AISC *Specification* Table J2.2. The length of each weld line is taken as the flange width, $b_f = 7.12 \text{ in.}$

$$\begin{aligned}
 R_n &= F_{nw} A_{we} && (\text{Spec. Eq. J2-3}) \\
 &= 0.60 F_{EXX} \left[\frac{5}{16}(2t) \right] (b_f)(2 \text{ lines}) \\
 &= 0.60(70 \text{ ksi}) \left(\frac{5}{16} \right) (2)(0.465 \text{ in.})(7.12 \text{ in.})(2 \text{ lines}) \\
 &= 174 \text{ kips}
 \end{aligned}$$

The available strength of the weld between the WT and the HSS is determined using AISC *Specification* Table J2.5:

LRFD	ASD
$\phi = 0.80$ $\phi R_n = 0.80(174 \text{ kips})$ $= 139 \text{ kips} > 88.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.88$ $\frac{R_n}{\Omega} = \frac{174 \text{ kips}}{1.88}$ $= 92.6 \text{ kips} > 58.8 \text{ kips} \quad \mathbf{o.k.}$

Note that if the flare-bevel groove weld alone did not provide enough capacity, a reinforcing fillet weld can be added.

Bolt shear

Required flange force at the interface between the beam and WTs.

LRFD	ASD
$M_u = R_u d$ $R_u = \frac{M_u}{d}$ $= \frac{(120 \text{ kip-ft})(12 \text{ in./ft})}{15.9 \text{ in.}}$ $= 90.6 \text{ kips}$	$M_a = R_a d$ $R_a = \frac{M_a}{d}$ $= \frac{(80 \text{ kip-ft})(12 \text{ in./ft})}{15.9 \text{ in.}}$ $= 60.4 \text{ kips}$

The strength of the bolt group is taken as the sum of the strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.7, the bearing strength of the WT stem or beam flange at the bolt hole per AISC *Specification* Section J3.11, or the tearout strength of the WT stem or beam flange at the bolt hole per AISC *Specification* Section J3.11.

From AISC *Manual* Table 7-1, the available shear strength per bolt for 7/8-in.-diameter, Group 120 bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 24.3 \text{ kips/bolt}$ The required number of bolts, n, is: $n = \frac{R_u}{\phi r_n}$ $= \frac{90.6 \text{ kips}}{24.3 \text{ kips/bolt}}$ $= 3.73 \text{ bolts}$	$\frac{r_n}{\Omega} = 16.2 \text{ kips/bolt}$ The required number of bolts, n, is: $n = \frac{R_a}{r_n/\Omega}$ $= \frac{60.4 \text{ kips}}{16.2 \text{ kips/bolt}}$ $= 3.73 \text{ bolts}$

Try four 7/8-in.-diameter Group 120 bolts.

Bearing and tearout

The available bearing and tearout strength of the WT stem and beam flange is determined using AISC *Manual* Tables 7-4 and 7-5. Because the beam flange and WT stem are the same thickness, and the beam flange end edge distance is greater than the WT stem edge distance (see Figures 6-14 and 6-15), bolt bearing and tearout at the beam flange will not control.

LRFD	ASD
For the interior bolts $s = 3 \text{ in.}$ $\phi r_n = (102 \text{ kips/bolt})(0.430 \text{ in.})$ $= 43.9 \text{ kips/bolt}$ For the edge bolts $l_e = 2 \text{ in.}$ $\phi r_n = (89.6 \text{ kips/bolt})(0.430 \text{ in.})$ $= 38.5 \text{ kips/bolt}$	For the interior bolts $s = 3 \text{ in.}$ $\frac{r_n}{\Omega} = (68.3 \text{ kips/bolt})(0.430 \text{ in.})$ $= 29.4 \text{ kips/bolt}$ For the edge bolts $l_e = 2 \text{ in.}$ $\frac{r_n}{\Omega} = (59.7 \text{ kips/bolt})(0.430 \text{ in.})$ $= 25.7 \text{ kips/bolt}$

The available bolt shear strength will control over bearing or tearout of the WT stem.

LRFD	ASD
$R_n = n\phi r_n$ $= (4 \text{ bolts})(24.3 \text{ kips/bolt})$ $= 97.2 \text{ kips} > 90.6 \text{ kips} \quad \mathbf{o.k.}$	$R_n = n \frac{r_n}{\Omega}$ $= (4 \text{ bolts})(16.2 \text{ kips/bolt})$ $= 64.8 \text{ kips} > 60.4 \text{ kips} \quad \mathbf{o.k.}$

Four 7/8-in.-diameter Group 120 bolts are acceptable.

Tension yielding in WT stem

Using AISC *Specification* Section J4.1(a), determine the available tensile yielding strength of the WT stem.

$$R_n = F_y A_g \quad (\text{Spec Eq. J4-1})$$

Because the WT extends past the edges of the beam flange, an effective width $b_{eff} = b_f = 6.99$ in. is used.

$$A_g = b_{eff} t_w$$

$$= (6.99 \text{ in.})(0.430 \text{ in.})$$

$$= 3.01 \text{ in.}^2$$

$$R_n = (50 \text{ ksi})(3.01 \text{ in.}^2)$$

$$= 151 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(151 \text{ kips})$ $= 136 \text{ kips} > 88.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{151 \text{ kips}}{1.67}$ $= 90.4 \text{ kips} > 58.8 \text{ kips} \quad \mathbf{o.k.}$

Tensile rupture of WT stem

Using AISC *Specification* Section J4, determine the nominal tensile rupture strength of the WT stem.

$$R_n = F_u A_e \quad (\text{Spec. Eq. J4-2})$$

The effective area, A_e , is determined from AISC *Specification* Section D3.3 as follows:

$$A_e = A_n U$$

$$A_n = A_g - 2(d_h + 1/16 \text{ in.})t_w$$

$$= 3.01 \text{ in.}^2 - 2(15/16 \text{ in.} + 1/16 \text{ in.})(0.430 \text{ in.})$$

$$= 2.15 \text{ in.}^2$$

$U = 1.0$ from AISC *Specification* Table D3.1 (no unconnected elements)

$$A_e = (2.15 \text{ in.}^2)(1.0)$$

$$= 2.15 \text{ in.}^2$$

The nominal tensile rupture strength of the WT stem is:

$$\begin{aligned} R_n &= F_u A_e \\ &= (65 \text{ ksi})(2.15 \text{ in.}^2) \\ &= 140 \text{ kips} \end{aligned}$$

The available tensile rupture strength of the WT stem is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(140 \text{ kips})$ $= 105 \text{ kips} > 88.2 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{140 \text{ kips}}{2.00}$ $= 70.0 \text{ kips} > 58.8 \text{ kips} \quad \text{o.k.}$

Block shear rupture in WT stem

Three possible block shear rupture failure paths in the WT stem are identified in Figure 6-14.

$$\begin{aligned} l_{ev} &= \frac{10 \text{ in.} - 3\frac{1}{2} \text{ in.}}{2} \\ &= 3.25 \text{ in.} \end{aligned}$$

Case 1

Because $2l_{ev} > 3\frac{1}{2} \text{ in.}$, Case 2 will control over Case 1.

Case 2

From AISC *Specification* Section J4.3, the nominal strength of the through plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= 2(l_{eh} + s)t_w \\ &= 2(2 \text{ in.} + 3 \text{ in.})(0.430 \text{ in.}) \\ &= 4.30 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - 2(n - 0.5)(d_h + \frac{1}{16} \text{ in.})t_w \\ &= 4.30 \text{ in.}^2 - 2(2 - 0.5)(\frac{15}{16} \text{ in.} + \frac{1}{16} \text{ in.})(0.430 \text{ in.}) \\ &= 3.01 \text{ in.}^2 \end{aligned}$$

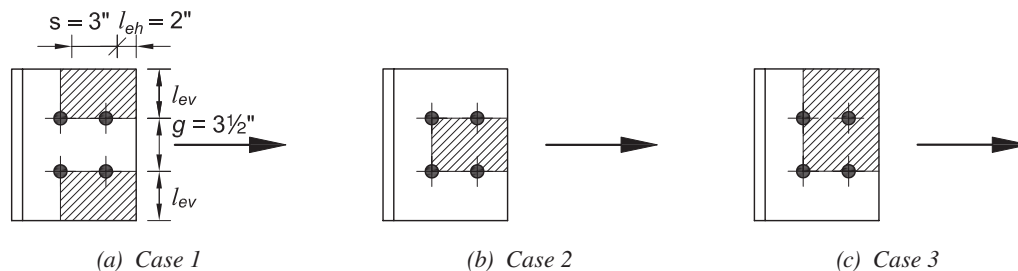


Fig. 6-14. Block shear rupture failure paths in WT stem.

$$\begin{aligned}
 A_{nt} &= [g - (d_h + 1/16 \text{ in.})]t_w \\
 &= [3\frac{1}{2} \text{ in.} - (1\frac{5}{16} \text{ in.} + 1/16 \text{ in.})](0.430 \text{ in.}) \\
 &= 1.08 \text{ in.}^2
 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned}
 R_n &= 0.60(65 \text{ ksi})(3.01 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.08 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(4.30 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.08 \text{ in.}^2) \\
 &= 188 \text{ kips} < 199 \text{ kips} \\
 &= 188 \text{ kips}
 \end{aligned}$$

The available strength of the WT stem for the limit state of block shear rupture, for Case 2, is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(188 \text{ kips})$ $= 141 \text{ kips} > 88.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{188 \text{ kips}}{2.00}$ $= 94.0 \text{ kips} > 58.8 \text{ kips} \quad \mathbf{o.k.}$

Case 3

From AISC *Specification* Section J4.3, the nominal strength of the through plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned}
 A_{gv} &= (l_{eh} + s)t_w \\
 &= (2 \text{ in.} + 3 \text{ in.})(0.430 \text{ in.}) \\
 &= 2.15 \text{ in.}^2
 \end{aligned}$$

$$\begin{aligned}
 A_{nv} &= A_{gv} - (n - 0.5)(d_h + 1/16 \text{ in.})t_w \\
 &= 2.15 \text{ in.}^2 - (2 - 0.5)(1\frac{5}{16} \text{ in.} + 1/16 \text{ in.})(0.430 \text{ in.}) \\
 &= 1.51 \text{ in.}^2
 \end{aligned}$$

$$\begin{aligned}
 A_{nt} &= [g + l_{ev} - (n - 0.5)(d_h + 1/16 \text{ in.})]t_w \\
 &= [3\frac{1}{2} \text{ in.} + 3.25 \text{ in.} - (2 - 0.5)(1\frac{5}{16} \text{ in.} + 1/16 \text{ in.})](0.430 \text{ in.}) \\
 &= 2.26 \text{ in.}^2
 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned}
 R_n &= 0.60(65 \text{ ksi})(1.51 \text{ in.}^2) + 1.0(65 \text{ ksi})(2.26 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(2.15 \text{ in.}^2) + 1.0(65 \text{ ksi})(2.26 \text{ in.}^2) \\
 &= 206 \text{ kips} < 211 \text{ kips} \\
 &= 206 \text{ kips}
 \end{aligned}$$

The available strength of the WT stem for the limit state of block shear rupture, for Case 3, is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(206 \text{ kips})$ $= 155 \text{ kips} > 88.2 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{206 \text{ kips}}{2.00}$ $= 103 \text{ kips} > 58.8 \text{ kips} \quad \mathbf{o.k.}$

Block shear rupture in the beam flange

The block shear rupture failure path in the beam flange is identified in Figure 6-15.

$$\begin{aligned} l_{ev} &= \frac{b_f - g}{2} \\ &= \frac{6.99 \text{ in.} - 3\frac{1}{2} \text{ in.}}{2} \\ &= 1.75 \text{ in.} \end{aligned}$$

From AISC *Specification* Section J4.3, determine the nominal block shear rupture strength of the beam flange:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= 2(l_{eh} + s)t_f \\ &= 2(2 \text{ in.} + 3 \text{ in.})(0.430 \text{ in.}) \\ &= 4.30 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - 2(n - 0.5)(d_h + \frac{1}{16} \text{ in.})t_f \\ &= 4.30 \text{ in.}^2 - 2(2 - 0.5)(\frac{15}{16} \text{ in.} + \frac{1}{16} \text{ in.})(0.430 \text{ in.}) \\ &= 3.01 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= [2l_{ev} - (d_h + \frac{1}{16} \text{ in.})]t_f \\ &= [2(1.75 \text{ in.}) - (\frac{15}{16} \text{ in.} + \frac{1}{16} \text{ in.})](0.430 \text{ in.}) \\ &= 1.08 \text{ in.}^2 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned} R_n &= 0.60(65 \text{ ksi})(3.01 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.08 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(4.30 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.08 \text{ in.}^2) \\ &= 188 \text{ kips} < 199 \text{ kips} \\ &= 188 \text{ kips} \end{aligned}$$

The available strength of the WT stem for the limit state of block shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(188 \text{ kips})$ $= 141 \text{ kips} > 88.2 \text{ kips} \quad \text{o.k.}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{188 \text{ kips}}{2.00}$ $= 94.0 \text{ kips} > 58.8 \text{ kips} \quad \text{o.k.}$

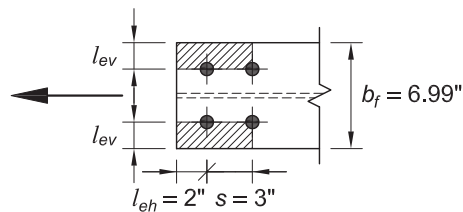


Fig. 6-15. Block shear failure path in beam flange.

The available connection strength is controlled by bolt shear.

The available flexural strength of the connection is:

LRFD	ASD
$\phi M_n = \phi R_n (d + t_w)$ $= (97.2 \text{ kips})(15.9 \text{ in.} + 0.430 \text{ in.}) \left(\frac{1 \text{ ft}}{12 \text{ in.}} \right)$ $= 132 \text{ kip-ft} > M_u = 120 \text{ kip-ft} \quad \mathbf{o.k.}$	$\frac{M_n}{\Omega} = \frac{R_n}{\Omega} (d + t_w)$ $= (64.8 \text{ kips})(15.9 \text{ in.} + 0.430 \text{ in.}) \left(\frac{1 \text{ ft}}{12 \text{ in.}} \right)$ $= 88.2 \text{ kip-ft} > M_a = 80.0 \text{ kip-ft} \quad \mathbf{o.k.}$

Chapter 7

Tension and Compression Connections

Round, rectangular, and square HSS are frequently used as bracing members in steel construction due to their high compressive strength-to-weight ratio. These members may be subjected to tension or compression loads, and the end connections to the supporting elements of the structure must be capable of resisting the required forces. In many applications, the forces in the member are much less than the yield capacity of the HSS. For example, the resistance of a slender compression brace is less than its yield strength. If the same force is developed in the reversed load condition, the tension force will also be considerably less than the yield capacity.

7.1 END-TEE CONNECTIONS

One type of connection for these members is the end-tee bolted connection shown in Figure 7-1. The tee can be either a WT section or a fitting built up with two plates. The flange is welded to the end of the HSS with a continuous weld around the perimeter naturally sealing the internal cavity to protect against corrosion. The stem is bolted to the supporting element. Single or multiple lines of bolts may be used. The transfer of force through the bolts to the connecting element is through shear. The length and width of the tee flange must extend beyond the HSS to provide a sufficient shelf dimension, s , to deposit the fillet weld around the perimeter of the HSS. For rectangular HSS members, it is ideal to orient the stem plate parallel to the long dimension, H , of the HSS section. This increases the likelihood that all four HSS walls are engaged by the stem plate and has resulted in increased capacities experimentally (Saidani, 2008).

For tension loads, the limit states for end-tee connections are:

1. Strength of the weld connecting the tee flange to the HSS or strength of the weld connecting the stem to the cap plate, for built-up tees
2. Strength of the HSS wall in local yielding
3. Strength of the tee flange for through-thickness shear adjacent to the stem
4. Strength of the tee flange in local yielding
5. Strength based on bolting to the tee stem
6. Strength of the stem in yielding, rupture, and block shear rupture

For compressive loading, two other limit states must be considered:

7. Strength of the HSS wall in crippling, for rectangular HSS
8. Strength based on buckling of the tee stem

The design method for end-tee or cap plate connections to the ends of HSS members, where the axial force in a plate is transferred to the HSS via a cap plate or via the flange of a tee-stub, recognizes that shear lag will be present in the HSS if some of the cross section is not loaded. A conservative slope of 2.5:1 for the load dispersion is assumed from each face of the tee-stub stem (Kitipornchai and Traves, 1989; Packer and Henderson, 1997; Wardenier et al., 2008a; Packer et al., 2009), as also adopted in AISC *Specification* Section J10.2. This produces a maximum dispersed load width of $(5t_p + l_b)$, where l_b is the bearing length [or alternatively the stem thickness, t_s , as shown in Figures 7-2(a) and 7-2(b)]. This load model is analogous to that used elsewhere in the AISC *Specification*, e.g., Equation J10-2.

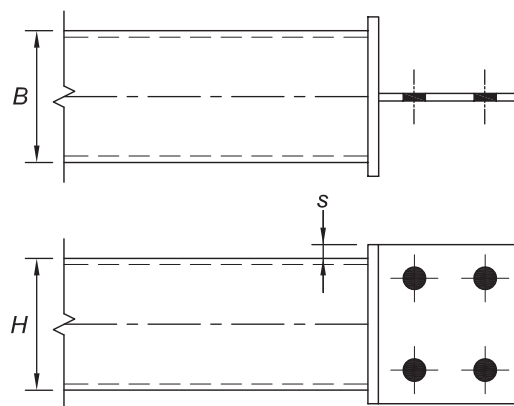


Fig. 7-1. End-tee bolted connection.

For the limit state of HSS local yielding, the nominal strength of a round or rectangular HSS, with the applied axial load in tension or compression, can be computed by summing the contributions of the parts of the HSS cross-sectional area into which the load is distributed, with an upper limit being the axial yield resistance of the gross area. Therefore,

$$R_n = 2F_y t(5t_p + t_s) \leq 0.9A_g F_y \quad (7-1)$$

For this limit state, $\phi = 1.00$ (LRFD) and $\Omega = 1.50$ (ASD). The ϕ and Ω values here are consistent with AISC *Specification* Section J10.2 for web local yielding, but it is logical to apply the resistance factor of 0.9 to $A_g F_y$ as an upper limit in Equation 7-1. It can be seen that the HSS cross-section capacity is fully utilized if $(5t_p + t_s) \geq D$ [in Figure 7-2(a)] or B [in Figure 7-2(b)], and no shear lag will be present. For this case $R_n = 0.9A_g F_y$. For a rectangular HSS it is assumed here that the line load is applied via a branch plate oriented parallel to the long wall dimension, H , as in Figure 7-2(b).

Another potential limit state to be checked, if the tee-stub flange or cap plate is in compression and the HSS is square or rectangular, is local crippling of the HSS sidewalls. AISC *Specification* Equation J10-4, which was derived for thin wide-flange section webs loaded by a concentrated force, has been applied historically to the case of a concentrated axial force on a capped rectangular HSS (AISC, 1997a). In doing so, the web depth has been taken as one-half the full width of the HSS, $B/2$. This assumes that the force spreads out at 45° in both directions from the center point of the bearing length, l_b , or t_s , as shown in Figure 7-3. The force

will therefore be across the full HSS width, B , at a depth no greater than $B/2$, below which crippling will not occur (AISC, 1997c). Thus, AISC *Specification* Equation J10-4 can be re-written for two HSS walls as:

$$R_n = 1.60t^2 \left[1 + 3 \left(\frac{t_s}{B/2} \right) \left(\frac{t}{t_p} \right)^{1.5} \right] \sqrt{\frac{E F_y t_p}{t}} \quad (7-2)$$

For this limit state, $\phi = 0.75$ (LRFD) and $\Omega = 2.00$ (ASD). The ϕ and Ω values here are consistent with AISC *Specification* Section J10.3 for web local crippling. The chord-stress interaction parameter, Q_f , in AISC *Specification* Equation J10-4 has been intentionally omitted from Equation 7-2 because this reduction factor is intended to address the influence of web compressive stress that is normal to the applied concentrated force; with the tee-stub or cap plate situation the web compressive stress is parallel to the applied concentrated force. This local crippling failure mode is avoided in prominent international design recommendations (Wardener et al., 2008a; Packer et al., 2009) by requiring the HSS cross section to be nonslender. It is interesting to examine the sidewall local crippling criterion relative to the sidewall local yielding criterion. For $(5t_p + t_s) < B$, $E = 29,000$ ksi, and $F_y = 50$ ksi, Equation 7-1 for local yielding would govern over Equation 7-2 for local crippling, for two walls, if (with application of the relevant resistance factors):

$$(5t_p + t_s) \leq 14.5\sqrt{t_p t} + 86.7 \left(\frac{t_s}{B} \right) \left(\frac{t^2}{t_p} \right) \quad (7-3)$$

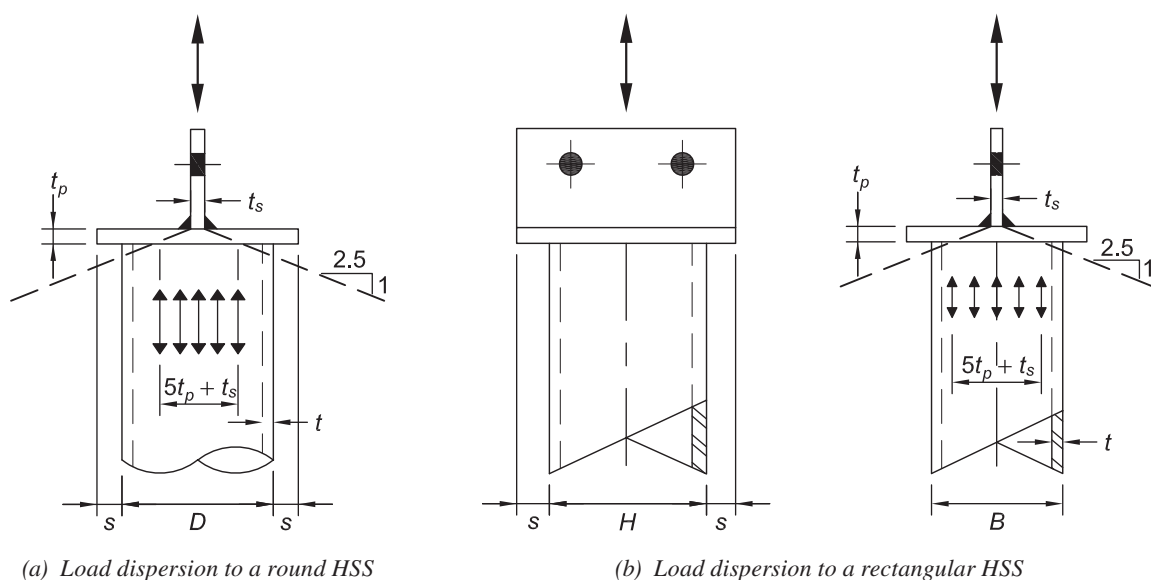


Fig. 7-2. Axial load dispersion through end-tee/cap plate connections.

Web crippling would only be critical over web yielding for very thin HSS walls, so one can further assume a punitive value of $B/t = 50$. Then,

$$(5t_p + t_s) \leq 14.5\sqrt{t_p t} + 1.73t_s \left(\frac{t}{t_p} \right) \quad (7-4)$$

It can be seen that this inequality holds (and therefore web crippling does not govern) for $0.05B \leq t_s \leq 0.2B$ over the range of $1.0 \leq t_s/t \leq 4.0$. Thus, for practical connections, Equation 7-2 is never likely to govern. Web crippling becomes critical over web yielding only for large bearing lengths, t_s , combined with thick cap plates (or tee-stub flanges), but for these combinations shear lag is negated, and all four walls of the HSS will participate, rendering Equation 7-2 insufficient.

Another limit state to be considered is local yielding of the stem of the tee-stub, for both axial compression and tension loading. In the same manner as the line load from the stem causes a load concentration in just part of the HSS cross section, the HSS causes a load concentration in the stem plate (and the adjoining welds) too, so that only the outer regions of the stem plate nearest the HSS wall are heavily loaded. Thus, the stem plate has an effective width that is less than its full width. In Figure 7-2(a), the diameter of the cap plate is $(D + 2s)$, and in Figure 7-2(b), the dimensions of the cap

plate are $(H + 2s) \times (B + 2s)$; the tee-stub stem is assumed to have the same width as the cap plate. Using a similar load dispersion as for local yielding of the HSS, the nominal strength of the tee-stub stem is:

$$R_n = 2F_{ys}t_s(t + 2.5t_p + s) \leq 2F_{ys}t_s(t + 5t_p) \quad (7-5)$$

For this limit state, $\phi = 1.00$ (LRFD) and $\Omega = 1.50$ (ASD). In Equation 7-5, F_{ys} and t_s pertain to the branch plate or stem. In Equations 7-1 and 7-5, the size of any weld legs has been conservatively ignored. If weld leg sizes are known, it is acceptable to assume load dispersion from the toes of the welds.

Tee-stub connections to the ends of HSS members usually have a single stem centered on the HSS member axis, but the stem is then frequently joined to a single gusset plate, usually by bolting. The resulting eccentricity causes a bending moment in the connection that must be considered. Therefore, a critical limit state for the connection design of compression braces is the buckling strength of the tee stem and connecting gusset plate. For this reason, it is advantageous to keep the buckling length as small as possible and the gusset plate connection compact. The procedure for determining the available buckling strength of the tee stem is based on an eccentrically loaded compression member as shown in Figure 7-4. Both ends are assumed fixed and can move

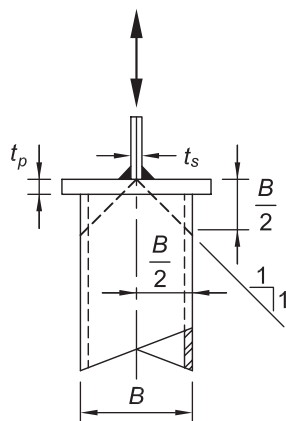


Fig. 7-3. Load dispersion for HSS local crippling.

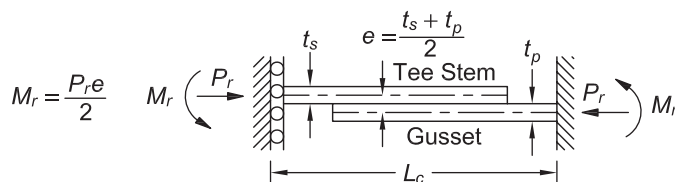


Fig. 7-4. Tee stem and gusset as an eccentrically loaded column.

laterally, resulting in the effective length factor $K = 1.2$. The geometric properties over the entire length, L_c , are taken as those of the thinner of the tee stem or supporting gusset. It is possible to add a stiffener to enhance the buckling strength; however, it is generally more economical to thicken the tee stem or gusset.

The plastic interaction equation developed by Dowswell (2015) can be applied for any load combination.

$$\left(\frac{P_r}{P_y}\right)^2 + \left(\frac{T_r}{T_p}\right)^2 + \left(\frac{V_r}{V_p}\right)^4 + \left[\left(\frac{M_{rx}}{M_{px}}\right)^{1.7} + \left(\frac{M_{rz}}{M_{pz}}\right)^{1.7}\right]^{0.59} \leq 1.0 \quad (7-6)$$

For the case of in-plane loading, and allowing for compression buckling, the interaction equation can be reduced to:

$$\left(\frac{P_r}{P_c}\right)^2 + \left(\frac{V_r}{V_p}\right)^4 + \frac{M_r}{M_p} \leq 1.0 \quad (7-7)$$

And for small shear load demand $[(V_r/V_p) \leq 0.40]$, the equation can be further reduced to:

$$\left(\frac{P_r}{P_c}\right)^2 + \frac{M_r}{M_p} \leq 1.0 \quad (7-8)$$

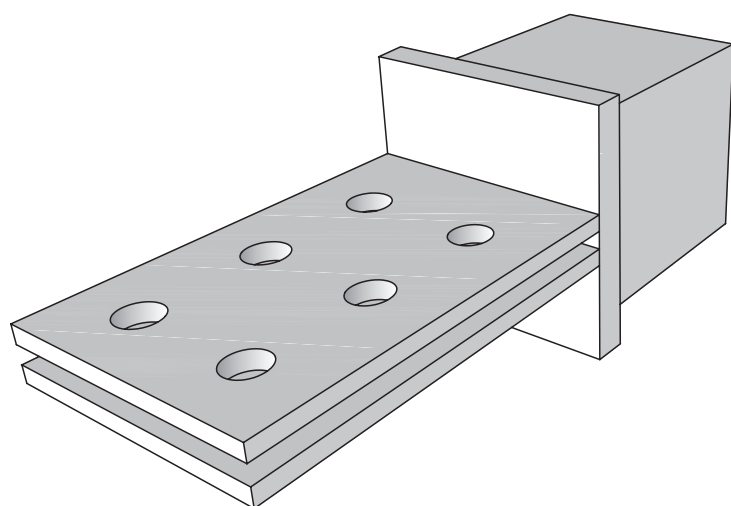
where P_c and M_p are the available strengths in axial compression and flexure. Using $M_r = P_r e/2$ (see Figure 7-4),

the equations can be solved for the maximum compressive strength.

There is also an option not commonly utilized to employ a built-up tee section with two tee stems as shown in Figures 7-5(a) and 7-5(b), where the gusset plate will be sandwiched between the two tee stems. This is referred to as a forked tee end or a double-stem T-connection and is most commonly utilized where compression loads are significant because it allows for concentric loading, negating the effects of the eccentricity of the single-tee stem configuration. In this case, the bearing length on the cap plate (or tee-stub flange) can be taken as the total distance between the out-sides of the two stems and used in lieu of t_s in Equations 7-1 and 7-2. In Equation 7-5, t_s would be replaced by $2t_s$.

7.2 SLOTTED HSS-TO-GUSSET PLATE CONNECTIONS

A very popular and simple HSS end connection is the slotted HSS-to-gusset plate connection shown in Figure 7-6. Typically used for the ends of diagonal bracing members, this type of connection is comprised of a gusset plate that is slotted into the end of an HSS member and fillet welded to both sides of the member. The projecting portion of the plate provides a more direct load path to the beam and column than the stem of an end-tee connection, negating any eccentricity. Figure 7-6(b) depicts a typical field welded HSS-to-gusset plate connection. The end of the member may be either sealed with end plates or left open.



(a) Forked tee end connection



(b) Installed forked tee end connection

Fig. 7-5. Cap plate tee-stub connections with double stems.

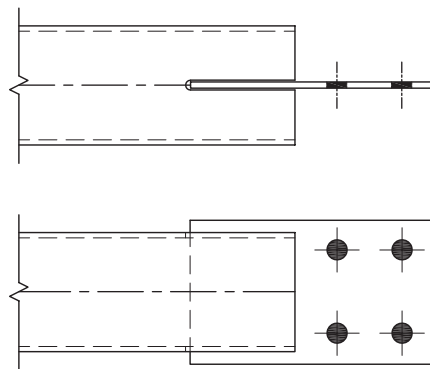
The slot in the HSS for the gusset plate is made wider than the plate thickness to facilitate fabrication and erection. AWS D1.1/D1.1M permits a $\frac{1}{16}$ in. gap with no increase in weld size. The weld size may need to be increased to account for the gap if larger than $\frac{1}{16}$ in. To avoid undercutting of the plate, it is recommended that the weld not be returned around the end of the plate in the slot. For field-welded braces, the slot length typically extends beyond the end of the plate to facilitate erection. Consequently, there is a reduced net area at the end of the gusset that, when combined with shear lag, could lead to HSS rupture as a critical limit state. Laboratory tests on such statically loaded connections (Martinez-Saucedo and Packer, 2006) have shown that the end of the slot in the HSS need not be machined or drilled to a smooth radius.

A similar connection where the gusset is slotted, or cut out around the HSS, is possible and has performed well under tension in experiments (Willibald et al., 2006). However, its

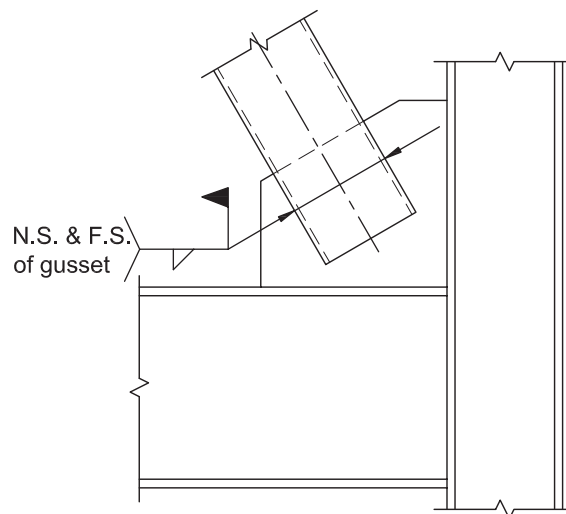
use under compression loading should be avoided because loading resulted in large deformations in the gusset plate, leading to premature failure.

Experimental and finite element analysis has shown the slotted HSS-to-gusset plate connection behavior is highly dependent on the length of the plate inserted into the HSS or, similarly, the length of weld. A long weld length results in high capacities. Conversely, for shorter weld lengths, the capacity is reduced due to shear lag effects, and the capacity is further influenced by the limit state of tensile rupture of the net section, as given in AISC *Specification* Equation D2-2.

Due to shear lag, the connections shown in Figure 7-6 often cannot develop the yield strength of the member according to AISC *Specification* Table D3.1. Shear lag is a concentration of force from the unconnected portions of the cross section into the gusset, as illustrated in Figure 7-7. Under tension or compression loading, the HSS can experience local failure initiating at the point of load concentration. Under tension



(a) Slotted HSS-to-gusset plate bolted connection



(b) Field-welded connection to gusset

Fig. 7-6. Slotted HSS-to-gusset plate connections.

loading of slotted HSS connections, shear lag results in a circumferential tensile fracture of the HSS at the end of the weld (section A-A in Figure 7-7). In calculating the member tensile capacity, consideration must be given to the effect of shear lag, by applying a reduction factor, U . Values for U are given in AISC *Specification* Table D3.1 for both round and rectangular HSS with a single concentric gusset plate. Equations for U have been updated to increase connection efficiencies for ratios of $\bar{x}/l_w < 0.23$ (reference Figure 7-8). For ratios of $\bar{x}/l_w > 0.23$, block shear failure will control. Although research has supported this finding (Martinez-Saucedo and Packer, 2009), the AISC *Specification* does not

distinguish between applicable ranges for circumferential tensile fracture and block shear failure, therefore both are checked.

Besides these two HSS member limit states, the other limit states that must be considered are:

1. Base metal shear in the HSS
2. Base metal shear in the gusset plate
3. Shear in the weld connecting the HSS to the gusset plate
4. Strength based on bolting to the gusset plate
5. Block shear rupture of gusset plate

7.3 END-PLATE CONNECTIONS

The bolted end-plate connection is comprised of a plate that is usually fillet welded around the perimeter of the end of an HSS member as shown in Figure 7-9. Pretensioned bolts are spaced around the perimeter to avoid shear lag. Under compressive loading, the force is distributed uniformly through the end plate. The full yield strength of the HSS can be developed with end-plate connections.

To obtain the minimum end-plate thickness, the distance, b , from the HSS wall to the bolt line should be as small as practical (around $1.5d$ to $2.0d$), keeping in mind clearances for an impact wrench, minimum spacing requirements (per AISC *Specification* Section J3.4), tolerances, and the possibility that weld legs may be slightly larger than what is specified.

Most connections where two plates are bolted together and subject to a tensile force tend to separate. This phenomenon is called prying action and results in an increase in the required bolt strength. A tee-stub model can be used to evaluate prying forces (see Figure 7-10). For relatively thick end plates, the plate does not bend significantly, and the sum of the bolt tensile forces is

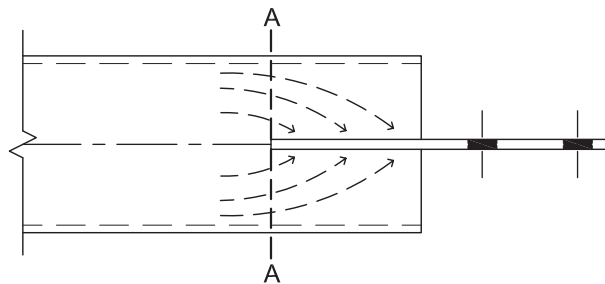


Fig. 7-7. Shear lag.

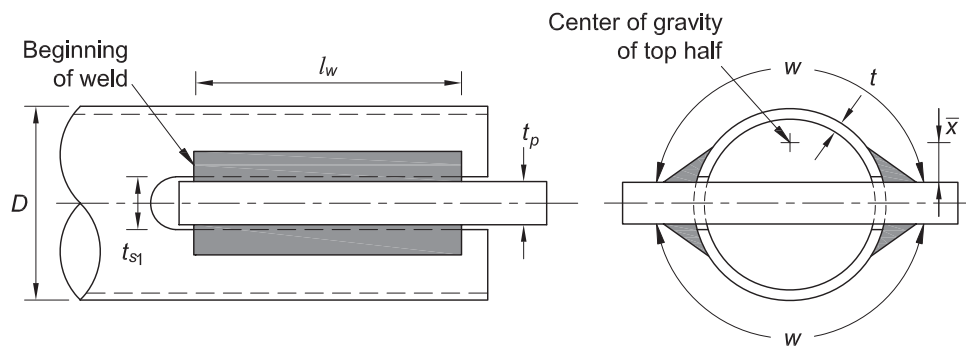


Fig. 7-8. Dimensions for slotted connections.

approximately equal to the applied tensile force, as shown in Figure 7-10(a). A thinner plate [Figure 7-10(b)] will develop hinge lines, developing prying forces at the edge of the plate. These prying forces are additive to the applied tension, thus increasing the actual tension in the bolts. Designers can usually choose between specifying a thicker plate with lower bolt loads or a thinner plate with higher bolt loads, the latter usually being more economical.

Due to the complexity of the analysis accounting for prying action and the position of yield lines in the plate, it is difficult to directly determine nominal capacities according to the AISC *Specification* provisions. Historically, and in many codes and publications based outside the United States, prying action has been based on F_y . Starting with the 13th Edition AISC *Manual* (2005b)

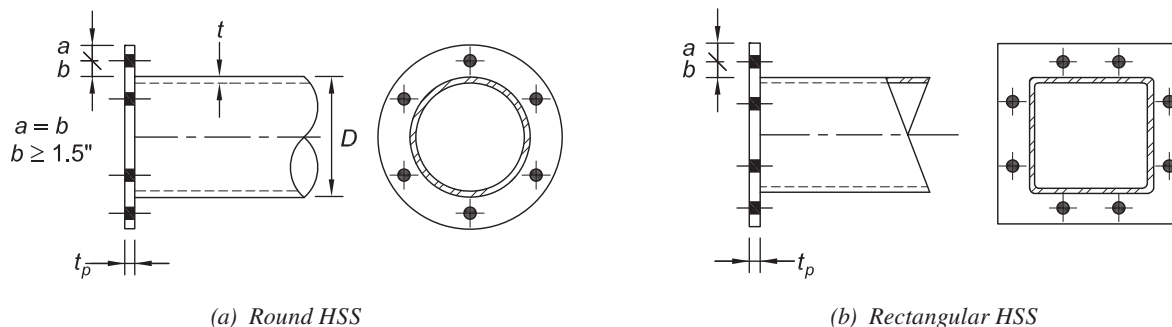


Fig. 7-9. End-plate connections.

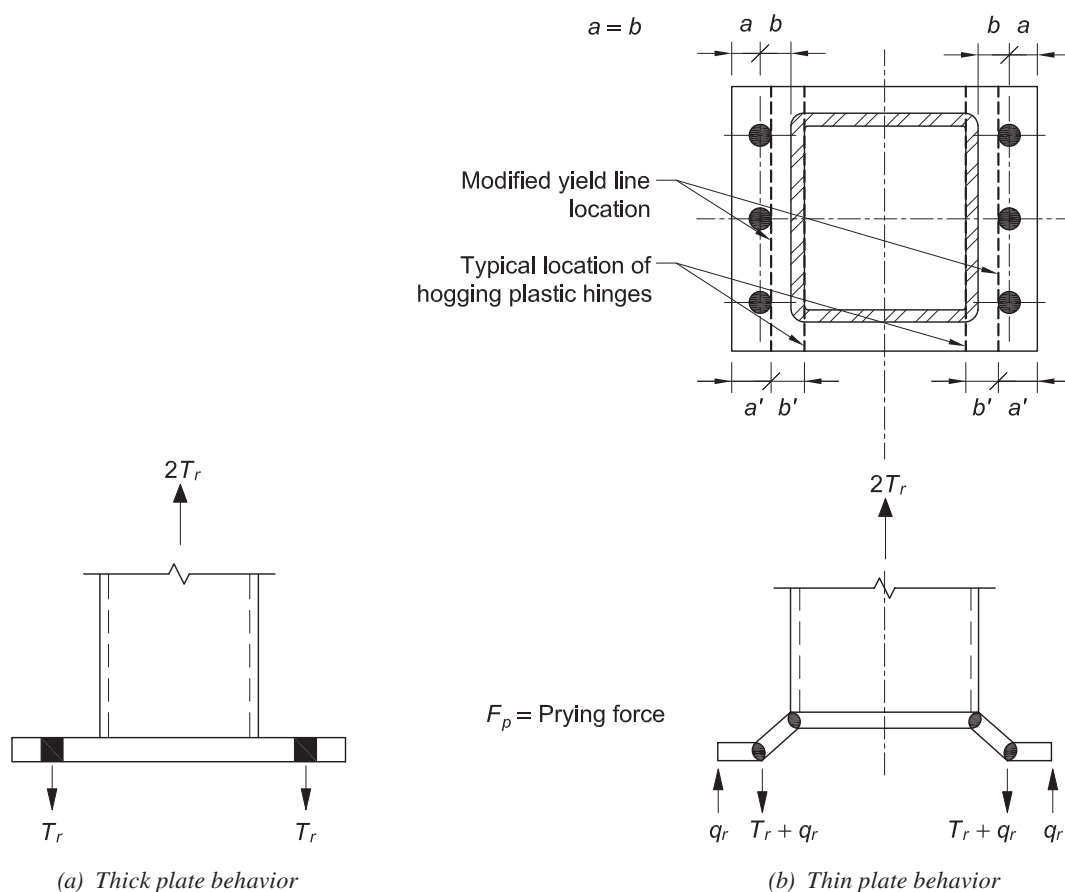


Fig. 7-10. Tee-stub model for assessment of prying forces.

and continuing through the present, F_u is used in lieu of F_y for prying calculations for WT and angle connections. This change resulted in better correlation with test data. The previous edition of this Design Guide introduced a prying action analysis for HSS end plates using F_y . While completely viable, this analysis conflicts with the current AISC *Manual* and there exists the desire to make prying checks, regardless of the section, cohesive in their approach.

A study of rectangular HSS end plates with bolts on four sides (Thornton, 2017) illustrated good correlation to test results while employing F_u in calculations. Additionally, Fidalgo and Packer (2022) studied the use of employing F_u for circular end plates and found established formulas to meet the required reliability index if using $\phi = 0.75$. Therefore, the prying analysis contained herein for these connections adopts the use of F_u . However, for the rectangular HSS to end-plate connection with bolts on two sides, adequate studies have not been performed to evaluate the correlation between available test data and F_u . Therefore, for that connection, the design methodology contained within this document still employs F_y .

The majority of research on bolted end-plate connections has focused on pure axial tension. Some research has focused on the pure moment loading case, but very little research has been performed on the combination of axial tension and bending. Where applied moment is present, for the methods described herein, it is assumed to be sufficiently small such that the combination of axial tension and moment does not result in any net compression in the joint. For more information pertaining to the combined effects on end-plate connections, refer to CIDECT Design Guide 3 (Packer et al., 2009), Section 11.4. Information regarding bolted moment end-plate connections for rectangular HSS can be found in the work by Wheeler et al., (1997, 1998, 2003), and for round HSS in the work by Fidalgo and Packer (2023).

7.3.1 End Plate on Round HSS

The method contained herein assumes the bolts in the end-plate connection shown in Figure 7-9(a) are equally spaced around the circumference and the connection is statically loaded. The distance, a , from the bolt line to the edge of the plate is assumed equal to b . The limit states are:

1. Yielding of the end plate
2. Tensile strength of the bolts, including prying action
3. Strength of the weld connecting the end plate to the HSS

Due to the complexity of the analysis accounting for prying action and flexure of the plate, it is difficult to directly determine nominal resistances according to the AISC *Specification* provisions. However, Igarashi et al. (1985) proposed a straightforward approach to design that assumes prying forces are always present, act at the edge of the plate, and amount to about a third of the total ultimate bolt force. Recently, this method has been studied by Fidalgo and Packer (2022) to explore the use of F_u in Igarashi's method. By employing $\phi = 0.75$, the required target reliability index is achieved. The available strengths are given in Equation 7-9 (end-plate plastification limit state) and Equation 7-10 (bolt fracture limit state) (Fidalgo and Packer, 2022).

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi P_n = 0.5\phi F_{up} t_p^2 \pi f_3$ (7-9a)	$\frac{P_n}{\Omega} = \frac{0.5 F_{up} t_p^2 \pi f_3}{\Omega}$ (7-9b)
$\phi P_n = \frac{n\phi r_n}{1 - \frac{1}{f_3} + \frac{1}{f_3 \ln(r_1/r_2)}}$ (7-10a)	$\frac{P_n}{\Omega} = \frac{n(r_n/\Omega)}{1 - \frac{1}{f_3} + \frac{1}{f_3 \ln(r_1/r_2)}}$ (7-10b)
(ln = natural logarithm)	(ln = natural logarithm)

where

$$f_3 = \frac{1}{2k_1} \left(k_3 + \sqrt{k_3^2 - 4k_1} \right) \quad (7-11)$$

$$k_1 = \ln(r_2/r_3) \quad (7-12)$$

$$k_3 = k_1 + 2 \quad (7-13)$$

$$r_1 = \frac{D}{2} + 2b \quad (7-14)$$

$$r_2 = \frac{D}{2} + b \quad (7-15)$$

$$r_3 = \frac{D-t}{2} \quad (7-16)$$

Because this design method produces a flexible end plate that imposes local bending on the HSS-to-plate weld, the HSS-to-plate weld needs to be designed to develop the HSS wall yield strength (Packer and Henderson, 1977).

7.3.2 End Plate on Rectangular HSS with Bolts on Four Sides

End-plate connections with bolts on all four sides are more common than with bolts on just two sides. Designs result in thinner plates but at the slight expense of an increased number of bolts. The limit states for the end-plate connection bolted on four sides as shown in Figure 7-11 are:

1. Yielding of the end plate
2. Tensile strength of the bolts, including prying action
3. Strength of the weld connecting the end plate to the HSS

Due to the complexity of the analysis accounting for prying action and the position of yield lines in the plate, it is difficult to directly determine nominal capacities according to the AISC *Specification* provisions. Details of a simple design procedure were presented by Willibald et al. (2003). Thornton (2017) later presented a study of end-plate connections with bolts on four sides using yield lines to match bolt patterns. It demonstrated that the prying action model given in Part 9 of the AISC *Manual* provides adequate results when the end plate minimum tensile strength is used in lieu of the minimum yield strength. The paper illustrates excellent correlation between experimental capacities in past research and the advocated prying action methodology.

The method by Thornton (2017) includes one additional step to what is presented in AISC *Manual* Part 9—an additional requirement on the maximum value of α' . When $\alpha' < 0$, bolt capacity controls the design. When $\alpha' > 1$, the plate capacity controls the design. If the plate is too thin, large values of α' result in large values of the prying ratio, β , and a thicker plate should be used. For the yield line pattern shown in Figure 7-12, the tributary yield line length per bolt given in Thornton (2017) is:

$$p = \frac{2(B+H+\pi b)}{n} \quad (7-17)$$

In the following, equations are presented to determine the thickness of the plate, t_p , the number of bolts, n , and the weld size, w . For the following design criteria to be valid, the centers of the bolt holes should not be positioned beyond the corners of the HSS.

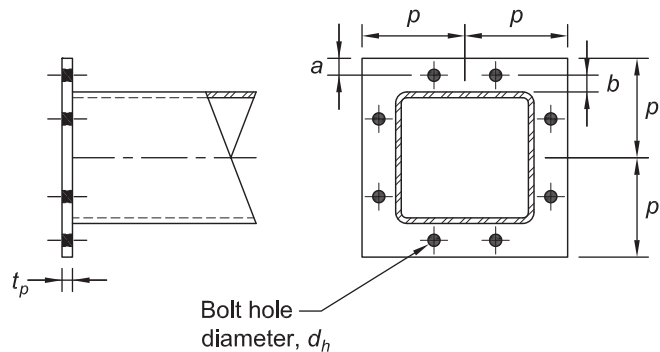


Fig. 7-11. Rectangular end plate with bolts on four sides with p and p' defined by Willibald et al. (2003).

The minimum plate thickness required for no prying action to occur, t_{np} , is:

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$t_{np} = \sqrt{\frac{4T_u b'}{\phi_b p F_u}}$ (from <i>Manual</i> Eq. 9-34a)	$t_{np} = \sqrt{\frac{\Omega_b 4T_u b'}{p F_u}}$ (from <i>Manual</i> Eq. 9-34b)

where

$$a' = a + \frac{d_b}{2} \leq 1.25b + \frac{d_b}{2} \quad (\text{from } Manual \text{ Eq. 9-23})$$

$$b' = b - \frac{d_b}{2} \quad (\text{from } Manual \text{ Eq. 9-24})$$

For the four-sided connection, the expression for b' has removed t compared to the definition of b' for the two-sided connection. This is based on experimental observations that the inner yield line occurs at the outside face of the HSS in the four-sided connection.

The method used herein, by Thornton (2017), uses a tributary end-plate length per bolt, p , according to Equation 7-17. On the other hand, the earlier method defined by Willibald et al. (2003) defined p as the lesser of p and p' as shown in Figure 7-11 (it also used F_{yp} for the plate strength). Both methods, however, reduce a 3D prying problem to a 2D problem, which enables the use of a T-stub prying model.

If the tributary end-plate length per bolt, p , is greater than $4\sqrt{b'(a+b)}$, independent yield line patterns can develop at each bolt (Dowswell, 2011), negating the assumed yield line patterns used in the development of this design method. Therefore,

$$p_{max} = 4\sqrt{b'(a+b)} \quad (7-18)$$

As discussed in AISC *Manual* Part 9, a smaller required thickness can be determined by designing the connecting element and bolted joint for the effects of prying action. The thickness required to ensure an acceptable combination of fitting strength, stiffness, and bolt strength is determined as follows:

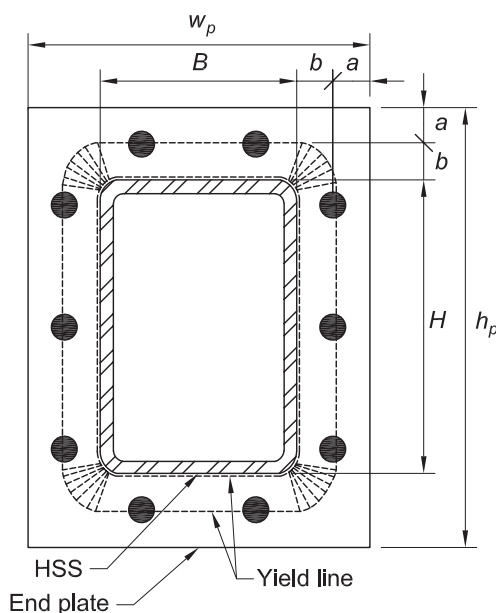


Fig. 7-12. Yield line pattern for square or rectangular HSS end plate by Thornton (2017).

$$t = t_c \sqrt{\frac{T_r}{T_c} \left(\frac{1}{1 + \delta \alpha'} \right)} \quad (7-19)$$

where

LRFD	ASD
$t_c = \sqrt{\frac{4T_c b'}{\phi_b p F_u}} \quad (\text{Manual Eq. 9-30a})$	$t_c = \sqrt{\frac{4\Omega_b T_c b'}{p F_u}} \quad (\text{Manual Eq. 9-30b})$

T_c = the available tension per bolt based on the limit state of tension only or the combined limit states of tension and shear rupture, ϕr_n or r_n/Ω , kips

$$\beta = \frac{1}{\rho} \left(\frac{T_c}{T_r} - 1 \right) \quad (\text{Manual Eq. 9-35})$$

$$\delta = 1 - \frac{d_h}{p} \quad (\text{from Manual Eq. 9-28})$$

d_h = nominal width of the hole along the length of the connecting element from AISC *Specification* Table J3.3, in.

$$\rho = \frac{b'}{a'} \quad (\text{Manual Eq. 9-29})$$

If $\beta \geq 1$

$$\alpha' = 1 \quad (\text{from Manual Eq. 9-36a})$$

If $0 \leq \beta < 1$

$$\alpha' = \min \left[\frac{1}{\delta} \left(\frac{\beta}{1 - \beta} \right), 1 \right] \quad (\text{from Manual Eq. 9-36b})$$

It is recommended when using this design method for end plates with bolts on four sides that α' be evaluated before finalizing a design. When $\alpha' < 0$, the bolts theoretically control the design. When $0 < \alpha' < 1$, the bolts and the plate are active in the design capacity. When $\alpha' > 1$, the plate controls the design capacity. As α' approaches 1.5, the prying ratio, β , approaches 50%. It is recommended that this value not be exceeded. Ratios greater than 50% showed excessive deformations in testing and therefore do not behave as assumed in the theory that this method is based on. Therefore, if $\alpha' > 1.5$, a thicker plate should be used (Thornton, 2017).

$$\alpha' = \frac{1}{\delta(1 + \rho)} \left[\left(\frac{t_c}{t} \right)^2 - 1 \right] \quad (\text{Manual Eq. 9-38})$$

The available tensile strength per bolt, T_c , can be determined from AISC *Manual* Table 7-2, and the following relationship can determine the available tensile strength of the bolts including the effects of prying, $T_{c,adj}$, as:

$$T_{c,adj} = Q T_c \quad (\text{Manual Eq. 9-40})$$

If $\alpha' \leq 0$

$$Q = 1 \quad (\text{Manual Eq. 9-39a})$$

If $0 < \alpha' \leq 1$

$$Q = \left(\frac{t}{t_c} \right)^2 (1 + \delta \alpha') \quad (\text{Manual Eq. 9-39b})$$

If $1 < \alpha' < 1.5$

$$Q = \left(\frac{t}{t_c} \right)^2 (1 + \delta)$$

(Manual Eq. 9-39c)

From AISC *Specification* Section J2.4, the fillet weld size, w , is determined based on the limit state of shear rupture as follows:

LRFD	ASD
$w \geq \frac{P_u \sqrt{2}}{F_{wc} l_w} \quad (7-20a)$	$w \geq \frac{P_a \sqrt{2}}{F_{wc} l_w} \quad (7-20b)$
where $F_{wc} = \phi_w F_w$ $F_w = 0.60 F_{EXX}$ $\phi_w = 0.75$	where $F_{wc} = \frac{F_w}{\Omega_w}$ $F_w = 0.60 F_{EXX}$ $\Omega_w = 2.00$

7.3.3 End Plate on Rectangular HSS with Bolts on Two Sides

While end-plate connections with bolts on two sides may be preferred for aesthetic reasons, they will inevitably result in thicker plates. For the design criteria to be valid, the centerline of the bolts in the end-plate connection should not be positioned beyond the corner of the rectangular HSS. The limit states for the end-plate connection bolted on two sides, as shown in Figure 7-13, are:

1. Yielding of the end plate
2. Tensile strength of the bolts, including prying action
3. Strength of the weld connecting the end plate to the HSS

By bolting along only two sides of an HSS member, the traditional tee-stub models were found to be inaccurate due in part to the location of the hogging plastic hinge lines forming within the member. Instead, a modified tee-stub design procedure can be used to evaluate the connection limit states (Packer and Henderson, 1997). To represent the observed location of the inner hogging plastic line, the distance b was adjusted to b' (see Figure 7-14)

$$b' = b - \frac{d_b}{2} + t \quad (7-21)$$

Because the connection design method for bolting on all four sides (see Section 7.3.2) reduces the 3D prying problem to a 2D one (by taking the worst case in two orthogonal directions), the tributary length approach by Willibald et. al in Section 7.3.2 can

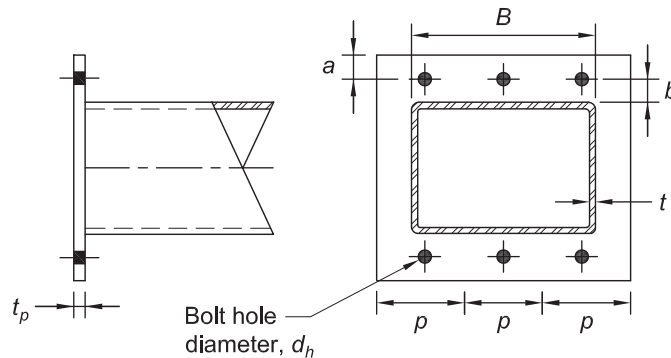


Fig. 7-13. Rectangular end plate with bolts on two sides.

be used for bolting on two sides, providing: (1) the b' dimension is adjusted per Equation 7-21; and (2) the plate yield stress, F_{yp} , is used instead of F_{up} . The tributary end-plate length per bolt, p , is shown in Figures 7-13 and 7-14. This design method has only been experimentally validated for end-plate thicknesses between ½ in. and 1 in.

7.4 SANDWICH GUSSET PLATES

For rectangular and square HSS, a stronger connection may typically be obtained by using two side gusset plates to sandwich the HSS, as shown in Figure 7-15. In this connection, the effect of shear lag is reduced relative to a single slotted HSS gusset plate, and the amount of gusset material is increased. However, there are more erection difficulties with this type of connection as opposed to a single, centered gusset plate. The connection also requires flare-bevel groove welds to attach the gussets to the HSS.

7.5 TENSION AND COMPRESSION CONNECTION DESIGN EXAMPLES

EXAMPLE 7.1—End-Tee Connection

Given:

Determine the required force, P_r , that the connection can withstand in tension and compression for the end connection shown in Figure 7-16. Use four 1-in.-diameter, Group 150 bolts with threads not excluded from the shear plane (thread condition N), in standard holes and 70 ksi filler metal. The built-up tee stem is attached to an identical plate that is welded to the support structure. The HSS is ASTM A500/A500M Grade C, and all plates are ASTM A572/A572M Grade 50.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

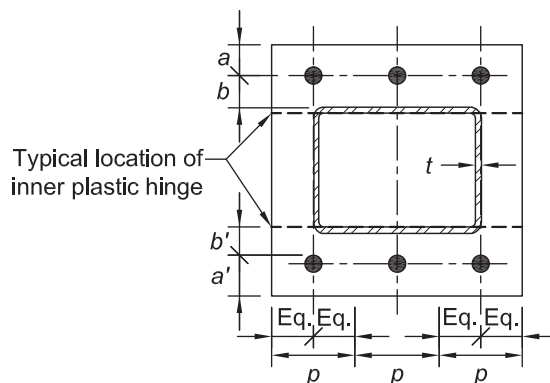


Fig. 7-14. Plastic hinge mechanism for two-sided connection.

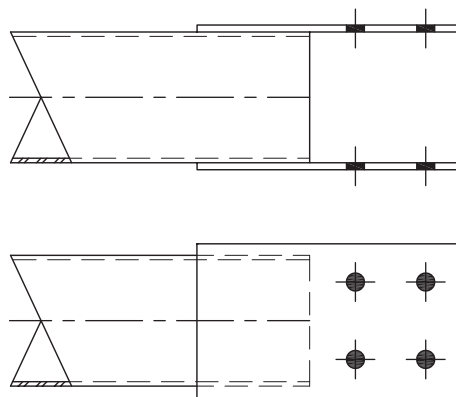


Fig. 7-15. Side gusset plates.

The force from the stem spreads on both sides through the cap plate at a 2.5:1 slope. From AISC *Specification* Section K2.3 Commentary and Figure C-K2.2, this produces a dispersed load width of $5t_p + b$. If the weld leg size is known, it is acceptable to assume load dispersion from the toes of the weld. In that case, b will be substituted for $(t_s + 2w)$. When this load width equals or exceeds the HSS wall width, B , the four walls of the HSS are engaged. This is determined as follows:

$$\begin{aligned} W &= 5t_p + t_s + 2w \\ &= 5(1 \text{ in.}) + \frac{3}{4} \text{ in.} + 2(\frac{5}{16} \text{ in.}) \\ &= 6.38 \text{ in.} > B = 6 \text{ in.} \end{aligned}$$

Therefore, all four HSS walls are fully effective, so use an effective weld length:

$$\begin{aligned} l &= 2(H + B) \\ &= 2(8.00 \text{ in.} + 6.00 \text{ in.}) \\ &= 28.0 \text{ in.} \end{aligned}$$

$$\begin{aligned} A_{we} &= l \frac{w}{\sqrt{2}} \\ &= (28.0 \text{ in.}) \left(\frac{\frac{5}{16} \text{ in.}}{\sqrt{2}} \right) \\ &= 6.19 \text{ in.}^2 \end{aligned}$$

From AISC *Specification* Section J2.4, for fillet welds to the end of rectangular HSS loaded in tension, the directional strength increase factor is $k_{ds} = 1.0$. Therefore:

$$\begin{aligned} R_n &= F_{nw} A_{we} k_{ds} \\ &= (42.0 \text{ ksi})(6.19 \text{ in.}^2)(1.0) \\ &= 260 \text{ kips} \end{aligned}$$

From AISC *Specification* Section J2.4, the available strength of the weld is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(260 \text{ kips})$ $= 195 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{260 \text{ kips}}{2.00}$ $= 130 \text{ kips}$

Available strength of the welds connecting the stem to the tee flange

From AISC *Specification* Section J2.4 and Table J2.5, the nominal strength of the weld metal is determined as follows:

$$R_n = F_{nw} A_{we} k_{ds} \quad (\text{Spec. Eq. J2.4})$$

From AISC *Specification* Table J2.5, the nominal strength of the weld metal per unit area is:

$$\begin{aligned} F_{nw} &= 0.60 F_{EXX} \\ &= 0.60(70 \text{ ksi}) \\ &= 42.0 \text{ ksi} \end{aligned}$$

Using the same load dispersion width of 2.5:1 as shown in AISC *Specification* Commentary Figure C-K2.2, the effective stem length is:

$$\begin{aligned} L &= 2(t + 2.5t_p + s) \\ &= 2[0.291 \text{ in.} + 2.5(1 \text{ in.}) + 0.500 \text{ in.}] \\ &= 6.58 \text{ in.} < 9 \text{ in.} \end{aligned}$$

Therefore, only part of the tee stem is effective and the effective weld length, l , is equal to the effective width on both sides of the stem. Thus,

$$\begin{aligned} l &= 2(6.58 \text{ in.}) \\ &= 13.2 \text{ in.} \\ A_{we} &= l \frac{w}{\sqrt{2}} \\ &= (13.2 \text{ in.}) \left(\frac{5/16 \text{ in.}}{\sqrt{2}} \right) \\ &= 2.92 \text{ in.}^2 \end{aligned}$$

From AISC *Specification* Section J2.4, the directional strength increase factor for the double-sided fillet weld is:

$$\begin{aligned} k_{ds} &= (1.0 + 0.50 \sin^{1.5} \theta) && (\text{Spec. Eq. J2-5}) \\ &= (1.0 + 0.50 \sin^{1.5} 90^\circ) \\ &= 1.5 \\ R_n &= F_{nw} A_{we} k_{ds} \\ &= (42.0 \text{ ksi})(2.92 \text{ in.}^2)(1.5) \\ &= 184 \text{ kips} \end{aligned}$$

From AISC *Specification* Section J2.4, the available strength of the weld is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(184 \text{ kips})$ $= 138 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{184 \text{ kips}}{2.00}$ $= 92.0 \text{ kips}$

Local yielding of the HSS wall

From previous calculations, all four walls are engaged; therefore, the available strength is the sum of the contribution of all four walls. The nominal strength per AISC *Specification* Section J4.1 is:

$$\begin{aligned} R_n &= F_y A_g && (\text{Spec. Eq. J4-1}) \\ &= (50 \text{ ksi})(7.59 \text{ in.}^2) \\ &= 380 \text{ kips} \end{aligned}$$

The available strength of the HSS walls for the limit state of local yielding is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(380 \text{ kips})$ $= 342 \text{ kips}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{380 \text{ kips}}{1.67}$ $= 228 \text{ kips}$

Shear yielding through the cap plate thickness

From AISC *Specification* Section J4.2, the nominal shear yielding strength is determined as follows:

$$R_n = 0.60F_y A_{gv} \quad (\text{Spec. Eq. J4-3})$$

Note that the total shear area is twice the effective length of the cap plate multiplied by the thickness of the cap plate. Half of the load is resisted at each shear plane adjacent to the welds. The total shear area is:

$$\begin{aligned}
 A_{gv} &= 2Lt_p \\
 &= 2(6.58 \text{ in.})(1 \text{ in.}) \\
 &= 13.2 \text{ in.}^2
 \end{aligned}$$

and therefore:

$$\begin{aligned}
 R_n &= 0.60(50 \text{ ksi})(13.2 \text{ in.}^2) \\
 &= 396 \text{ kips}
 \end{aligned}$$

The available strength of the cap plate for the limit state of shear yielding is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(396 \text{ kips})$ $= 396 \text{ kips}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{396 \text{ kips}}{1.50}$ $= 264 \text{ kips}$

Shear rupture through the cap plate thickness

From AISC *Specification* Section J4.2, the nominal shear rupture strength is determined as follows:

$$R_n = 0.60F_u A_{nv} \quad (\text{Spec. Eq. J4-4})$$

where

$$\begin{aligned}
 A_{nv} &= (2 \text{ planes})b_p t_p \\
 &= (2 \text{ planes})(9 \text{ in.})(1 \text{ in.}) \\
 &= 18.0 \text{ in.}^2
 \end{aligned}$$

and therefore

$$\begin{aligned}
 R_n &= 0.60(65 \text{ ksi})(18.0 \text{ in.}^2) \\
 &= 702 \text{ kips}
 \end{aligned}$$

The available strength of the cap plate for the limit state of shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(702 \text{ kips})$ $= 527 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{702 \text{ kips}}{2.00}$ $= 351 \text{ kips}$

Local yielding of tee stem

From the previous weld calculation, only part of the tee stem is effective. The nominal strength is:

$$\begin{aligned}
 R_n &= 2F_{ys}t_s(t + 2.5t_p + s) && \text{(from Eq. 7-5)} \\
 &= 2(50 \text{ ksi})(\frac{3}{4} \text{ in.})[0.291 \text{ in.} + 2.5(1 \text{ in.}) + 0.500 \text{ in.}] \\
 &= 247 \text{ kips}
 \end{aligned}$$

The available strength of the tee stem for the limit state of local yielding is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(247 \text{ kips})$ $= 222 \text{ kips}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{247 \text{ kips}}{1.67}$ $= 148 \text{ kips}$

Strength of bolted connection

The available shear transfer strength at bolt holes is the sum of the effective strength of individual bolts per the User Note in AISC *Specification* Section J3.7, which is the least of the available bolt shear strength determined in accordance with AISC *Specification* Section J3.7 and the available bearing or tearout strength of the tee stem at the bolt hole.

From AISC *Manual* Table 7-1, the available shear strength per bolt for 1-in.-diameter, Group 150 bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 40.0 \text{ kips}$	$\frac{r_n}{\Omega} = 26.7 \text{ kips}$

The available bearing and tearout strength of the tee stem at an edge bolt is determined using AISC *Manual* Table 7-5, with $l_e = 2 \text{ in.}$, as follows:

LRFD	ASD
$\phi r_n = (84.1 \text{ kips/in.})(\frac{3}{4} \text{ in.})$ $= 63.1 \text{ kips}$	$\frac{r_n}{\Omega} = (56.1 \text{ kips/in.})(\frac{3}{4} \text{ in.})$ $= 42.1 \text{ kips}$

The available bearing and tearout strength of the tee stem at the bolts not adjacent to the edge is determined using AISC *Manual* Table 7-4 conservatively assuming $s = 3 \text{ in.}$

LRFD	ASD
$\phi r_n = (110 \text{ kips/in.})(\frac{3}{4} \text{ in.})$ $= 82.5 \text{ kips}$	$\frac{r_n}{\Omega} = (73.1 \text{ kips/in.})(\frac{3}{4} \text{ in.})$ $= 54.8 \text{ kips}$

The available shear strength of the bolt controls over bearing or tearout of the tee stem. Therefore, the available strength of the bolted connection is determined as follows:

LRFD	ASD
$\phi R_n = \phi r_n n$ $= (40.0 \text{ kips})(4 \text{ bolts})$ $= 160 \text{ kips}$	$\frac{R_n}{\Omega} = \frac{r_n}{\Omega} n$ $= (26.7 \text{ kips})(4 \text{ bolts})$ $= 107 \text{ kips}$

Tensile yielding of the support plate

From AISC *Specification* Section J4.1(a), the nominal tensile yield strength of the plate is:

$$R_n = F_y A_g \quad (\text{Spec. Eq. J4-1})$$

where

$$A_g = l_w t_s$$

$$l_w = 2(4 \text{ in.}) \tan 30^\circ + 4 \text{ in.}$$

$$= 8.62 \text{ in.}$$

$$A_g = (8.62 \text{ in.})(\frac{3}{4} \text{ in.})$$

$$= 6.47 \text{ in.}^2$$

and therefore

$$R_n = (50 \text{ ksi})(6.47 \text{ in.}^2)$$

$$= 324 \text{ kips}$$

The available tensile yielding strength of the plate is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(324 \text{ kips})$ $= 292 \text{ kips}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{324 \text{ kips}}{1.67}$ $= 194 \text{ kips}$

Tensile rupture of the support plate

From AISC *Specification* Section J4.1(b), the nominal tensile rupture strength of the plate is:

$$R_n = F_u A_e \quad (\text{Spec. Eq. J4-2})$$

where

$$A_e = A_n U$$

$$U = 1.0 \text{ from AISC } \textit{Specification} \text{ Table D3.1 (no unconnected elements)}$$

The net tensile area is:

$$\begin{aligned} A_n &= A_g - 2(d_h + 1/16 \text{ in.})t_s \\ &= 6.47 \text{ in.}^2 - 2(1 1/8 \text{ in.} + 1/16 \text{ in.})(3/4 \text{ in.}) \\ &= 4.69 \text{ in.}^2 \end{aligned}$$

The effective tensile area is:

$$\begin{aligned} A_e &= A_n U \\ &= (4.69 \text{ in.}^2)(1.0) \\ &= 4.69 \text{ in.}^2 \end{aligned}$$

The nominal tensile rupture strength of the plate is:

$$\begin{aligned} R_n &= (65 \text{ ksi})(4.69 \text{ in.}^2) \\ &= 305 \text{ kips} \end{aligned}$$

The available tensile rupture strength of the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(305 \text{ kips})$ $= 229 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{305 \text{ kips}}{2.00}$ $= 153 \text{ kips}$

Block shear rupture of the support plate

The tee-stem plate and the support plate have the same geometry; therefore, they will both have the same block shear rupture strength. There are three cases for which block shear rupture of the plate must be checked as shown in Figure 7-17.

Case 1

Case 1 and Case 2 have the same shear area, but Case 2 has a smaller tension area. Therefore, Case 2 will control over Case 1.

Case 2

From AISC *Specification* Section J4.3, the nominal strength of the plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

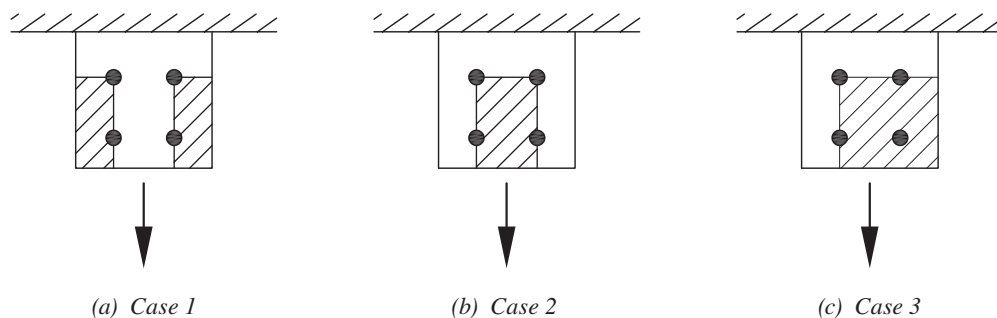


Fig. 7-17. Block shear cases.

where

$$\begin{aligned} A_{gv} &= 2[(n-1)s + l_{ev}]t \\ &= 2[(2-1)(4 \text{ in.}) + 2 \text{ in.}](\frac{3}{4} \text{ in.}) \\ &= 9.00 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - 2(n-0.5)(d_h + \frac{1}{16} \text{ in.})t \\ &= 9.00 \text{ in.}^2 - 2(2-0.5)(1\frac{1}{8} \text{ in.} + \frac{1}{16} \text{ in.})(\frac{3}{4} \text{ in.}) \\ &= 6.33 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= [g - (d_h + \frac{1}{16} \text{ in.})]t \\ &= [4 \text{ in.} - (1\frac{1}{8} \text{ in.} + \frac{1}{16} \text{ in.})](\frac{3}{4} \text{ in.}) \\ &= 2.11 \text{ in.}^2 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned} R_n &= 0.60(65 \text{ ksi})(6.33 \text{ in.}^2) + 1.0(65 \text{ ksi})(2.11 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(9.00 \text{ in.}^2) + 1.0(65 \text{ ksi})(2.11 \text{ in.}^2) \\ &= 384 \text{ kips} < 407 \text{ kips} \\ &= 384 \text{ kips} \end{aligned}$$

Therefore:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(384 \text{ kips})$ $= 288 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{384 \text{ kips}}{2.00}$ $= 192 \text{ kips}$

Case 3

From AISC *Specification* Section J4.3, the nominal strength of the plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \quad (\text{Spec. Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= [(n-1)s + l_{ev}]t \\ &= [(2-1)(4 \text{ in.}) + 2 \text{ in.}](\frac{3}{4} \text{ in.}) \\ &= 4.50 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (n-0.5)(d_h + \frac{1}{16} \text{ in.})t \\ &= 4.50 \text{ in.}^2 - (2-0.5)(1\frac{1}{8} \text{ in.} + \frac{1}{16} \text{ in.})(\frac{3}{4} \text{ in.}) \\ &= 3.16 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= [g + l_{eh} - 1.5(d_h + \frac{1}{16} \text{ in.})]t \\ &= [4 \text{ in.} + 2\frac{1}{2} \text{ in.} - 1.5(1\frac{1}{8} \text{ in.} + \frac{1}{16} \text{ in.})](\frac{3}{4} \text{ in.}) \\ &= 3.54 \text{ in.}^2 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned} R_n &= 0.60(65 \text{ ksi})(3.16 \text{ in.}^2) + 1.0(65 \text{ ksi})(3.54 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(4.50 \text{ in.}^2) + 1.0(65 \text{ ksi})(3.54 \text{ in.}^2) \\ &= 353 \text{ kips} < 365 \text{ kips} \\ &= 353 \text{ kips} \end{aligned}$$

Therefore:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(353 \text{ kips})$ $= 265 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{353 \text{ kips}}{2.00}$ $= 177 \text{ kips}$

The available strength in tension is therefore controlled by the available strength of the weld connecting the stem to the tee flange.

LRFD	ASD
$\phi R_n = 138 \text{ kips}$	$\frac{R_n}{\Omega} = 92.0 \text{ kips}$

Additional limit states for available compressive strength

HSS wall local crippling

The nominal compressive strength due to wall local crippling is:

$$\begin{aligned}
 R_n &= 1.60t^2 \left[1 + 3 \left(\frac{t_s}{B/2} \right) \left(\frac{t}{t_p} \right)^{1.5} \right] \sqrt{\frac{EF_y t_p}{t}} \\
 &= 1.60(0.291 \text{ in.})^2 \left\{ 1 + 3 \left[\frac{3/4 \text{ in.}}{(6.00 \text{ in.})/2} \right] \left(\frac{0.291 \text{ in.}}{1 \text{ in.}} \right)^{1.5} \right\} \sqrt{\frac{(29,000 \text{ ksi})(50 \text{ ksi})(1 \text{ in.})}{0.291 \text{ in.}}} \\
 &= 338 \text{ kips}
 \end{aligned} \tag{7-2}$$

The available compressive strength of the HSS for the limit state of wall local crippling is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(338 \text{ kips})$ $= 254 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{338 \text{ kips}}{2.00}$ $= 169 \text{ kips}$

Buckling of the stem and interaction for combined flexure and compression

Because there is no shear force in the tee stem, the effect of combined flexure and compression can be determined using Equation 7-8.

$$\left(\frac{P_r}{P_c} \right)^2 + \frac{M_r}{M_p} \leq 1.0 \tag{7-8}$$

The first-order moment caused by an eccentricity, e , measured between the centerline of the tee stem and support plate will be amplified to account for the second-order effects (Dowswell, 2018b).

The total required flexural strength is:

$$M_r = B_2 M$$

where the second-order multiplier, B_2 , is:

$$B_2 = \frac{1}{1 - \frac{\alpha P_r}{P_e}}$$

$$M = \frac{P_r e}{2}$$

$$\alpha = 1.0 \text{ (LRFD)}; \alpha = 1.6 \text{ (ASD)}$$

$$e = \frac{\frac{3}{4} \text{ in.} + \frac{3}{4} \text{ in.}}{2} \\ = 0.750 \text{ in.}$$

To determine the available compressive strength, P_c , the slenderness ratio, L_c/r , is first calculated as follows:

$$r = \frac{t_s}{\sqrt{12}} \\ = \frac{\frac{3}{4} \text{ in.}}{\sqrt{12}} \\ = 0.217 \text{ in.}$$

Because both ends are assumed to be rotationally fixed with the ability to move laterally, from AISC *Specification* Table C-A-7.1, the effective length factor, $K = 1.2$, is used. Therefore:

$$L_c = KL \\ = 1.2(3.00 \text{ in.} + 4.00 \text{ in.} + 3.00 \text{ in.}) \\ = 12.0 \text{ in.}$$

$$\frac{L_c}{r} = \frac{12.0 \text{ in.}}{0.217 \text{ in.}} \\ = 55.3$$

$$I = \frac{b_p t_s^3}{12} \\ = \frac{(9 \text{ in.})(\frac{3}{4} \text{ in.})^3}{12} \\ = 0.316 \text{ in.}^3$$

$$P_e = \frac{\pi^2 EI}{(L_c)^2} \\ = \frac{\pi^2 (29,000 \text{ ksi})(0.316 \text{ in.}^3)}{(12.0 \text{ in.})^2} \\ = 200 \text{ kips}$$

From AISC *Manual* Table 4-14, the available nominal compressive stress is found by interpolation, which can then be used to determine the available compressive strength, P_c , as follows:

LRFD	ASD
$\phi_c F_n = 36.0 \text{ ksi}$ $P_c = \phi P_n$ $= \phi_c F_n A_g$	$\frac{F_n}{\Omega_c} = 23.9 \text{ ksi}$ $P_c = \frac{P_n}{\Omega}$ $= \left(\frac{F_n}{\Omega_c} \right) A_g$
where	where
$A_g = b_p t_s$ $= (9 \text{ in.})(\frac{3}{4} \text{ in.})$ $= 6.75 \text{ in.}^2$ $P_c = (36.0 \text{ ksi})(6.75 \text{ in.}^2)$ $= 243 \text{ kips}$	$A_g = b_p t_s$ $= (9.00 \text{ in.})(\frac{3}{4} \text{ in.})$ $= 6.75 \text{ in.}^2$ $P_c = (23.9 \text{ ksi})(6.75 \text{ in.}^2)$ $= 161 \text{ kips}$

The plastic section modulus for a rectangular bar is:

$$\begin{aligned}
 Z &= \frac{b_p t_s^2}{4} \\
 &= \frac{(9 \text{ in.})(\frac{3}{4} \text{ in.})^2}{4} \\
 &= 1.27 \text{ in.}^3
 \end{aligned}$$

Using AISC *Specification* Section F11, the available flexural strength, M_n , for a rectangular bar is:

LRFD	ASD
$\phi_b = 0.90$ $M_c = \phi_b F_{yp} Z$ $= 0.90(50 \text{ ksi})(1.27 \text{ in.}^3)$ $= 57.2 \text{ kip-in.}$	$\Omega_b = 1.67$ $M_c = \frac{F_{yp} Z}{\Omega_b}$ $= \frac{(50 \text{ ksi})(1.27 \text{ in.}^3)}{1.67}$ $= 38.0 \text{ kip-in.}$

Using the interaction equation, the largest required compressive strength, P_r , can be solved for:

LRFD	ASD
$\left(\frac{P_r}{P_c} \right)^2 + \frac{B_2 \left(\frac{P_r e}{2} \right)}{M_c} = 1.0$ $\left(\frac{P_r}{243 \text{ kips}} \right)^2 + \frac{\left[\frac{1}{1 - \left(\frac{1.0 P_r}{200 \text{ kips}} \right)} \right] \left[\frac{P_r (\frac{3}{4} \text{ in.})}{2} \right]}{57.2 \text{ kip-in.}} = 1.0$ $P_r = 80.9 \text{ kips}$	$\left(\frac{P_r}{P_c} \right)^2 + \frac{B_2 \left(\frac{P_r e}{2} \right)}{M_c} = 1.0$ $\left(\frac{P_r}{161 \text{ kips}} \right)^2 + \frac{\left[\frac{1}{1 - \frac{1.6 P_r}{200 \text{ kips}}} \right] \left[\frac{P_r (\frac{3}{4} \text{ in.})}{2} \right]}{38.0 \text{ kip-in.}} = 1.0$ $P_r = 52.5 \text{ kips}$

The available compressive strength is therefore controlled by buckling of the stem.

LRFD	ASD
$\phi R_n = 80.9 \text{ kips}$	$\frac{R_n}{\Omega} = 52.5 \text{ kips}$

EXAMPLE 7.2—Slotted Round HSS-to-Gusset Plate Connection

Given:

Determine the available tensile strength for the ASTM A500/A500M Grade C HSS6.000×0.250 in the connection illustrated in Figure 7-18. The slot in the HSS for the ASTM A572/A572M Grade 50 gusset plate is concentric and is $\frac{1}{16}$ in. wider than the plate thickness to allow for clearance. The four $\frac{7}{8}$ -in.-diameter Group 120 bolts with threads not excluded from the shear plane are in standard holes. Use 70 ksi weld filler metal.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

HSS

ASTM A500/A500M Grade C

$F_y = 50 \text{ ksi}$

$F_u = 62 \text{ ksi}$

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50 \text{ ksi}$

$F_{up} = 65 \text{ ksi}$

From AISC *Manual* Table 1-13, the HSS geometric properties are as follows:

HSS6.000×0.250

$D = 6.00 \text{ in.}$

$t = 0.233 \text{ in.}$

$A = 4.22 \text{ in.}^2$

Solution:

HSS tensile yielding

From AISC *Specification* Section D2, the nominal tensile strength for tensile yielding in the gross section is:

$$\begin{aligned}
 P_n &= F_y A_g && (\text{Spec. Eq. D2-1}) \\
 &= (50 \text{ ksi})(4.22 \text{ in.}^2) \\
 &= 211 \text{ kips}
 \end{aligned}$$

The available tensile strength for tensile yielding in the gross section is:

LRFD	ASD
$\phi_t = 0.90$	$\Omega_t = 1.67$
$\phi_t P_n = 0.90(211 \text{ kips})$	$\frac{P_n}{\Omega_t} = \frac{211 \text{ kips}}{1.67}$
$= 190 \text{ kips}$	$= 126 \text{ kips}$

HSS tensile rupture

From AISC *Specification* Section D2, the nominal tensile strength for tensile rupture in the net section is:

$$P_n = F_u A_e \quad (\text{Spec. Eq. D2-2})$$

where the effective area, A_e , of tension members is determined from AISC *Specification* Section D3 as follows:

$$A_e = A_n U \quad (\text{Spec. Eq. D3-1})$$

Accounting for a slot clearance of $1/16$ in.,

$$\begin{aligned} A_n &= A_g - 2(t_p + 1/16 \text{ in.})t \\ &= 4.22 \text{ in.}^2 - 2(3/8 \text{ in.} + 1/16 \text{ in.})(0.233 \text{ in.}) \\ &= 4.02 \text{ in.}^2 \end{aligned}$$

From AISC *Specification* Table D3.1, Case 5:

$$\bar{x} = \frac{R \sin \theta}{\theta} - \frac{1}{2} t_p$$

Using geometry,

$$2\theta = 3.02 \text{ rad}$$

$$\theta = 1.51 \text{ rad}$$

Therefore,

$$\begin{aligned} R &= D/2 \\ &= (6.00 \text{ in.})/2 \\ &= 3.00 \text{ in.} \end{aligned}$$

$$\begin{aligned} \bar{x} &= \frac{(3.00 \text{ in.}) \sin(1.51 \text{ rad})}{1.51 \text{ rad}} - \frac{1}{2} (3/8 \text{ in.}) \\ &= 1.80 \text{ in.} \end{aligned}$$

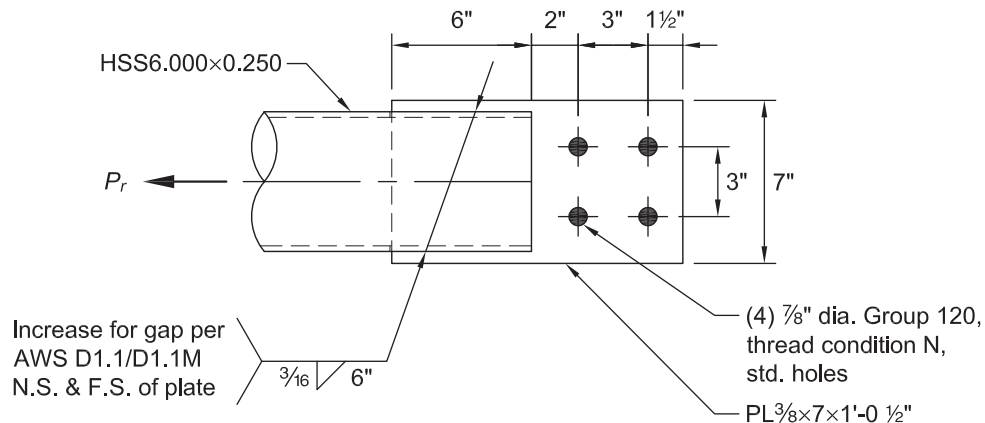


Fig. 7-18. Slotted round HSS-to-concentric gusset-plate connection for Example 7.2.

$$U = \left[1 + \left(\frac{\bar{x}}{l} \right)^{3.2} \right]^{-10}$$

$$= \left[1 + \left(\frac{1.80 \text{ in.}}{6.00 \text{ in.}} \right)^{3.2} \right]^{-10}$$

$$= 0.811$$

$$A_e = (4.02 \text{ in.}^2)(0.811)$$

$$= 3.26 \text{ in.}^2$$

Therefore, the available tensile strength for the limit state of tensile rupture in the net section is determined as follows:

$$P_n = (62 \text{ ksi})(3.26 \text{ in.}^2)$$

$$= 202 \text{ kips}$$

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(202 \text{ kips})$ $= 152 \text{ kips}$	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{202 \text{ kips}}{2.00}$ $= 101 \text{ kips}$

Gusset plate tensile yielding

From AISC *Specification* Section J4, the nominal tensile strength for the limit state of tensile yielding of the gusset plate is:

$$R_n = F_y A_g \quad (\text{Spec. Eq. J4-1})$$

where

$$A_g = W t_p$$

$$= (7.00 \text{ in.})(\frac{3}{8} \text{ in.})$$

$$= 2.63 \text{ in.}^2$$

and therefore:

$$R_n = (50 \text{ ksi})(2.63 \text{ in.}^2)$$

$$= 132 \text{ kips}$$

The available tensile yielding strength of the gusset plate is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(132 \text{ kips})$ $= 119 \text{ kips}$	$\Omega = 1.67$ $\frac{R_n}{\Omega} = \frac{132 \text{ kips}}{1.67}$ $= 79.0 \text{ kips}$

Gusset plate tensile rupture

From AISC *Specification* Section J4, the nominal tensile rupture strength of the gusset plate is:

$$R_n = F_u A_e \quad (\text{Spec. Eq. J4-2})$$

The effective area, A_e , is determined from AISC *Specification* Section D3 as follows:

$$A_e = A_n U$$

$$U = 1.0 \text{ from AISC } \textit{Specification} \text{ Table D3.1 (no unconnected elements)}$$

The net tensile area is:

$$\begin{aligned} A_n &= A_g - 2(d_h + 1/16 \text{ in.})t_p \\ &= 2.63 \text{ in.}^2 - 2(15/16 \text{ in.} + 1/16 \text{ in.})(3/8 \text{ in.}) \\ &= 1.88 \text{ in.}^2 \end{aligned}$$

The effective tensile area is:

$$\begin{aligned} A_e &= A_n \\ &= 1.88 \text{ in.}^2 \end{aligned}$$

The nominal tensile rupture strength of the gusset plate is:

$$\begin{aligned} R_n &= (65 \text{ ksi})(1.88 \text{ in.}^2) \\ &= 122 \text{ kips} \end{aligned}$$

The available tensile rupture strength of the gusset plate is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(122 \text{ kips})$	$\frac{R_n}{\Omega} = \frac{122 \text{ kips}}{2.00}$
$= 91.5 \text{ kips}$	$= 61.0 \text{ kips}$

Block shear rupture of the gusset plate

There are three cases for which block shear rupture of the gusset plate must be checked; see Figure 7-19. From AISC *Specification* Section J4.3, the nominal strength of the gusset plate based on the limit state of block shear rupture is:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \quad (\text{Spec. Eq. J4-5})$$

Case 1

Case 1 and Case 2 have the same shear area, but Case 2 has a smaller tension area. Therefore, Case 2 will control over Case 1.

Case 2

The available block shear rupture strength of the gusset plate is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b, and 9-3c, and AISC *Specification* Equation J4-5, with $n = 2$, $l_{eh} = 2 \text{ in.}$, $l_{ev} = 1\frac{1}{2} \text{ in.}$, and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC <i>Manual</i> Table 9-3a: $\frac{\phi F_u A_{nt}}{t} = 73.1 \text{ kips/in.}$	Tension rupture component from AISC <i>Manual</i> Table 9-3a: $\frac{F_u A_{nt}}{\Omega t} = 48.8 \text{ kips/in.}$
Shear yielding component from AISC <i>Manual</i> Table 9-3b: $\frac{\phi 0.60 F_y A_{gv}}{t} = 101 \text{ kips/in.}$	Shear yielding component from AISC <i>Manual</i> Table 9-3b: $\frac{0.60 F_y A_{gv}}{\Omega t} = 67.5 \text{ kips/in.}$
Shear rupture component from AISC <i>Manual</i> Table 9-3c: $\frac{\phi 0.60 F_u A_{nv}}{t} = 87.8 \text{ kips/in.}$	Shear rupture component from AISC <i>Manual</i> Table 9-3c: $\frac{0.60 F_u A_{nv}}{\Omega t} = 58.5 \text{ kips/in.}$
The available block shear rupture strength is: $\begin{aligned} \phi R_n &= \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt} \leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt} \\ &= \left(\frac{3}{8} \text{ in.}\right) \left[(2 \text{ planes})(87.8 \text{ kips/in.}) + 1.0(73.1 \text{ kips/in.}) \right] \\ &\leq \left(\frac{3}{8} \text{ in.}\right) \left[(2 \text{ planes})(101 \text{ kips/in.}) + 1.0(73.1 \text{ kips/in.}) \right] \\ &= 93.3 \text{ kips} < 103 \text{ kips} \end{aligned}$	The available block shear rupture strength is: $\begin{aligned} \frac{R_n}{\Omega} &= \frac{0.60 F_u A_{nv}}{\Omega} + \frac{U_{bs} F_u A_{nt}}{\Omega} \leq \frac{0.60 F_y A_{gv}}{\Omega} + \frac{U_{bs} F_u A_{nt}}{\Omega} \\ &= \left(\frac{3}{8} \text{ in.}\right) \left[(2 \text{ planes})(58.5 \text{ kips/in.}) + 1.0(48.8 \text{ kips/in.}) \right] \\ &\leq \left(\frac{3}{8} \text{ in.}\right) \left[(2 \text{ planes})(67.5 \text{ kips/in.}) + 1.0(48.8 \text{ kips/in.}) \right] \\ &= 62.2 \text{ kips} < 68.9 \text{ kips} \end{aligned}$
Therefore: $\phi R_n = 93.3 \text{ kips}$	Therefore: $\frac{R_n}{\Omega} = 62.2 \text{ kips}$

Case 3

$$\begin{aligned} A_{gv} &= [(n-1)s + l_{ev}] t_p \\ &= [(2-1)(3 \text{ in.}) + 1\frac{1}{2} \text{ in.}] \left(\frac{3}{8} \text{ in.}\right) \\ &= 1.69 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (n-0.5)(d_h + \frac{1}{16} \text{ in.}) t_p \\ &= 1.69 \text{ in.}^2 - (2-0.5)(\frac{15}{16} \text{ in.} + \frac{1}{16} \text{ in.}) \left(\frac{3}{8} \text{ in.}\right) \\ &= 1.13 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= [g + l_{eh} - 1.5(d_h + \frac{1}{16} \text{ in.})] t_p \\ &= [3 \text{ in.} + 2 \text{ in.} - 1.5(\frac{15}{16} \text{ in.} + \frac{1}{16} \text{ in.})] \left(\frac{3}{8} \text{ in.}\right) \\ &= 1.31 \text{ in.}^2 \end{aligned}$$

$U_{bs} = 1.0$ because tension is uniform

$$\begin{aligned} R_n &= 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \\ &= 0.60(65 \text{ ksi})(1.13 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.31 \text{ in.}^2) \leq 0.60(50 \text{ ksi})(1.69 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.31 \text{ in.}^2) \\ &= 129 \text{ kips} < 136 \text{ kips} \\ &= 129 \text{ kips} \end{aligned}$$

Therefore, the available strength of the gusset plate for the limit state of block shear rupture is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(129 \text{ kips})$ $= 96.8 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{129 \text{ kips}}{2.00}$ $= 64.5 \text{ kips}$

Shear of the weld metal and shear of the HSS and gusset plate base metals

Because the HSS slot is $\frac{1}{16}$ in. wider than the gusset thickness, the gap for either weld will not exceed $\frac{1}{16}$ in., and no adjustment of the weld size is required per AWS D1.1/D1.1M. Therefore, the effective weld size is the full weld: $w = \frac{3}{16}$ in. From Table 3-3, $t_{min} = 0.150$ in. for ASTM A500/A500M Grade C HSS with a $\frac{3}{16}$ in. weld size. The design thickness of the HSS is $t = 0.233$ in., which is greater than $t_{min} = 0.150$ in. Therefore, the shear strength of the weld metal controls over the shear strength of the HSS base metal.

From AISC *Specification* Section J2.4, the nominal weld strength, per unit length, is:

$$\begin{aligned}
 R_n &= 0.60 F_{EXX} \frac{w}{\sqrt{2}} l (4 \text{ lines}) \\
 &= 0.60 (70 \text{ ksi}) \left(\frac{\frac{3}{16} \text{ in.}}{\sqrt{2}} \right) (6.00 \text{ in.}) (4 \text{ lines}) \\
 &= 134 \text{ kips}
 \end{aligned}$$

The available strength of the weld is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(134 \text{ kips})$ $= 101 \text{ kips}$	$\Omega = 2.00$ $\frac{R_n}{\Omega} = \frac{134 \text{ kips}}{2.00}$ $= 67.0 \text{ kips}$

Strength of bolted connection

Assume the supporting material is at least as thick as the plate and will not control the strength. From AISC *Manual* Table 7-1, the available shear strength per bolt for $\frac{7}{8}$ -in.-diameter Group 120 bolts with threads not excluded from the shear plane (thread condition N) loaded in single shear is:

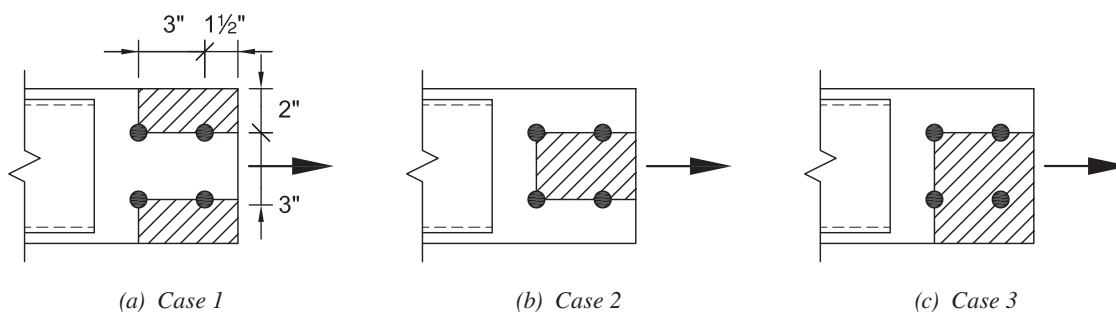


Fig. 7-19. Block shear cases.

LRFD	ASD
$\phi r_n = 24.3 \text{ kips/bolt}$	$\frac{r_n}{\Omega} = 16.2 \text{ kips/bolt}$

From AISC *Specification* Section J3.11, the nominal bearing strength of a bolt in a standard hole when deformation at the bolt hole at service load is a design consideration is:

$$\begin{aligned}
 r_n &= 2.4dtF_u && \text{(from Spec. Eq. J3-6a)} \\
 &= 2.4\left(\frac{7}{8} \text{ in.}\right)\left(\frac{3}{8} \text{ in.}\right)(65 \text{ ksi}) \\
 &= 51.2 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(51.2 \text{ kips/bolt})$ $= 38.4 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{51.2 \text{ kips/bolt}}{2.00}$ $= 25.6 \text{ kips/bolt}$

The available tearout strength of the plate at the interior bolts is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration.

$$\begin{aligned}
 l_c &= s - d_h \\
 &= 3 \text{ in.} - \frac{15}{16} \text{ in.} \\
 &= 2.06 \text{ in.} \\
 r_n &= 1.2l_ctF_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(2.06 \text{ in.})\left(\frac{3}{8} \text{ in.}\right)(65 \text{ ksi}) \\
 &= 60.3 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(60.3 \text{ kips/bolt})$ $= 45.2 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{60.3 \text{ kips/bolt}}{2.00}$ $= 30.2 \text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at the interior bolts.

The available tearout strength of the plate at the edge bolts is determined from AISC *Specification* Section J3.11, assuming deformation at service load is a design consideration.

$$\begin{aligned}
 l_c &= l_{ev} - 0.5d_h \\
 &= 1\frac{1}{2} \text{ in.} - 0.5\left(\frac{15}{16} \text{ in.}\right) \\
 &= 1.03 \text{ in.} \\
 r_n &= 1.2l_ctF_u && \text{(from Spec. Eq. J3-6c)} \\
 &= 1.2(1.03 \text{ in.})\left(\frac{3}{8} \text{ in.}\right)(65 \text{ ksi}) \\
 &= 30.1 \text{ kips/bolt}
 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(30.1 \text{ kips/bolt})$ $= 22.6 \text{ kips/bolt}$	$\Omega = 2.00$ $\frac{r_n}{\Omega} = \frac{30.1 \text{ kips/bolt}}{2.00}$ $= 15.1 \text{ kips/bolt}$

Therefore, tearout controls over bolt bearing or bolt shear at the edge bolts.

The strength of the bolt group is determined as follows:

LRFD	ASD
$\phi R_n = (2 \text{ bolts})(24.3 \text{ kips/bolt})$ $+ (2 \text{ bolts})(22.6 \text{ kips/bolt})$ $= 93.8 \text{ kips}$	$\frac{R_n}{\Omega} = (2 \text{ bolts})(16.2 \text{ kips/bolt})$ $+ (2 \text{ bolts})(15.1 \text{ kips/bolt})$ $= 62.6 \text{ kips}$

The available strength in tension is controlled by the limit state of gusset plate tensile rupture.

LRFD	ASD
$\phi R_n = 91.5 \text{ kips}$	$\frac{R_n}{\Omega} = 61.0 \text{ kips}$

EXAMPLE 7.3—End Plate on Round HSS Connection

Given:

Determine the end-plate thickness, weld size, and the number of bolts required to resist a tensile dead load $P_D = 25 \text{ kips}$ and a live load $P_L = 75 \text{ kips}$ for the connection shown in Figure 7-20. Use an ASTM A500/A500M Grade C HSS6.625×0.375 section, an ASTM A572/A572M Grade 50 end plate, 7/8-in.-diameter Group 120 bolts, and 70 ksi weld filler metal.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

HSS

ASTM A500/A500M Grade C

$F_y = 50 \text{ ksi}$

$F_u = 62 \text{ ksi}$

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50 \text{ ksi}$

$F_{up} = 65 \text{ ksi}$

From AISC *Manual* Table 1-13, the HSS geometric properties are as follows:

HSS6.625×0.375

$D = 6.625 \text{ in.}$

$t = 0.349 \text{ in.}$

$A = 6.88 \text{ in.}^2$

Solution:

From ASCE/SEI 7, Chapter 2, the required tensile strength is:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(25 \text{ kips}) + 1.6(75 \text{ kips})$ $= 150 \text{ kips}$	$P_a = P_D + P_L$ $= 25 \text{ kips} + 75 \text{ kips}$ $= 100 \text{ kips}$

Required end-plate thickness, t_p

LRFD	ASD
$\phi P_n = 0.5\phi F_{up} t_p^2 \pi f_3 \quad (7-9a)$ $t_p = \sqrt{\frac{2P_u}{\phi F_{up} \pi f_3}}$	$\frac{P_n}{\Omega} = \frac{0.5 F_{up} t_p^2 \pi f_3}{\Omega} \quad (7-9b)$ $t_p = \sqrt{\frac{2P_a \Omega}{F_{up} \pi f_3}}$

where

$$r_1 = \frac{D}{2} + 2b \quad (7-14)$$

$$= \frac{6.625 \text{ in.}}{2} + 2(1\frac{1}{2} \text{ in.})$$

$$= 6.31 \text{ in.}$$

$$r_2 = \frac{D}{2} + b \quad (7-15)$$

$$= \frac{6.625 \text{ in.}}{2} + 1\frac{1}{2} \text{ in.}$$

$$= 4.81 \text{ in.}$$

$$r_3 = \frac{D-t}{2} \quad (7-16)$$

$$= \frac{6.625 \text{ in.} - 0.349 \text{ in.}}{2}$$

$$= 3.14 \text{ in.}$$

$$k_1 = \ln(r_2/r_3) \quad (7-12)$$

$$= \ln\left(\frac{4.81 \text{ in.}}{3.14 \text{ in.}}\right)$$

$$= 0.426$$

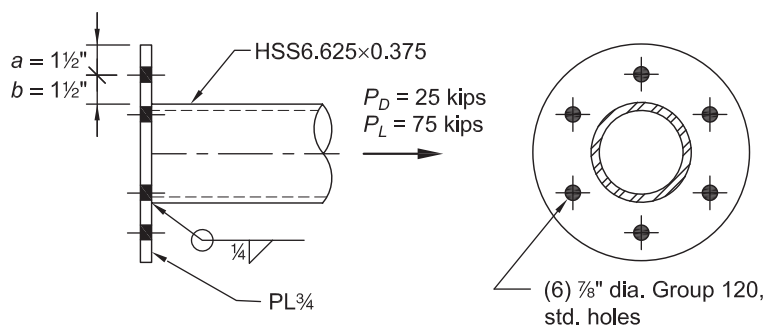


Fig. 7-20. End plate-to-round HSS end connection for Example 7.3.

$$\begin{aligned}
 k_3 &= k_1 + 2 \\
 &= 0.428 + 2 \\
 &= 2.43
 \end{aligned}
 \tag{7-13}$$

$$\begin{aligned}
 f_3 &= \frac{1}{2k_1} \left(k_3 + \sqrt{k_3^2 - 4k_1} \right) \\
 &= \frac{1}{2(0.426)} \left[2.43 + \sqrt{2.43^2 - 4(0.426)} \right] \\
 &= 5.26
 \end{aligned}
 \tag{7-11}$$

The thickness of the plate is:

LRFD	ASD
$\phi = 0.75$ $t_p \geq \sqrt{\frac{2P_u}{\phi F_{up} \pi f_3}}$ $\geq \sqrt{\frac{2(150 \text{ kips})}{0.75(65 \text{ ksi})\pi(5.26)}}$ $\geq 0.610 \text{ in.}$	$\Omega = 2.00$ $t_p \geq \sqrt{\frac{2P_a \Omega}{F_{up} \pi f_3}}$ $\geq \sqrt{\frac{2(100 \text{ kips})(2.00)}{(65 \text{ ksi})\pi(5.26)}}$ $\geq 0.610 \text{ in.}$

Use a $\frac{3}{4}$ in. end plate.

Tensile strength of the bolts

From AISC *Manual* Table 7-2, the available tensile strength for a $\frac{7}{8}$ -in.-diameter Group 120 bolt is:

LRFD	ASD
$\phi r_n = 40.6 \text{ kips}$	$r_n/\Omega = 27.1 \text{ kips}$

The minimum number of bolts required is:

LRFD	ASD
$\phi P_n = \frac{n\phi r_n}{1 - \frac{1}{f_3} + \frac{1}{f_3 \ln(r_1/r_2)}} \tag{7-10a}$ Solving for n: $ \begin{aligned} n &\geq \frac{P_u}{\phi r_n} \left[1 - \frac{1}{f_3} + \frac{1}{f_3 \ln(r_1/r_2)} \right] \\ &\geq \frac{150 \text{ kips}}{40.6 \text{ kips}} \left[1 - \frac{1}{5.26} + \frac{1}{5.26 \ln(6.31 \text{ in.}/4.81 \text{ in.})} \right] \\ &\geq 5.58 \end{aligned} $	$\frac{P_n}{\Omega} = \frac{n(r_n/\Omega)}{1 - \frac{1}{f_3} + \frac{1}{f_3 \ln(r_1/r_2)}} \tag{7-10b}$ Solving for n: $ \begin{aligned} n &\geq \frac{P_a}{r_n/\Omega} \left[1 - \frac{1}{f_3} + \frac{1}{f_3 \ln(r_1/r_2)} \right] \\ &\geq \frac{100 \text{ kips}}{27.1 \text{ kips}} \left[1 - \frac{1}{5.26} + \frac{1}{5.26 \ln(6.31 \text{ in.}/4.81 \text{ in.})} \right] \\ &\geq 5.57 \end{aligned} $

Use six bolts.

Check minimum bolt spacing

$$s = \frac{2\pi r_2}{n}$$

$$= \frac{2\pi(4.81 \text{ in.})}{6}$$

$$= 5.04 \text{ in.}$$

From the User Note in AISC *Specification* Section J3.4, the preferred minimum spacing is:

$$3d_b = 3(7/8 \text{ in.})$$

$$= 2.63 \text{ in.} < s = 5.04 \text{ in.} \quad \mathbf{o.k.}$$

From AISC *Specification* Table J3.4, the minimum edge distance for a 7/8-in.-diameter bolt in a standard hole is 1 1/8 in.

$$a = 1\frac{1}{2} \text{ in.} > 1\frac{1}{8} \text{ in.} \quad \mathbf{o.k.}$$

Weld between HSS and end plate

For this connection design procedure, the HSS-to-plate weld needs to develop the HSS wall yield strength. For a fillet weld, the strength is permitted to be increased by the directional strength increase factor, k_{ds} , per AISC *Specification* Section J2.4(a). Therefore, the required weld size can be calculated as:

$$w \geq \frac{P_r \sqrt{2}}{F_{wc} k_{ds} \pi D}$$

where

$$k_{ds} = 1.0 + 0.5 \sin^{1.5} \theta$$

(Spec. Eq. J2-5)

The HSS wall yield strength is:

LRFD	ASD
$\phi_t = 0.90$ $P_u = \phi_t F_y A$ $= 0.90(50 \text{ ksi})(6.88 \text{ in.}^2)$ $= 310 \text{ kips}$	$\Omega_t = 1.67$ $P_a = \frac{F_y A}{\Omega_t}$ $= \frac{(50 \text{ ksi})(6.88 \text{ in.}^2)}{1.67}$ $= 206 \text{ kips}$

The minimum weld size required is:

LRFD	ASD
$F_{wc} = \phi_w F_w$ $F_w = 0.60 F_{EXX}$ $\phi_w = 0.75$ $w \geq \frac{P_u \sqrt{2}}{\phi_w F_w k_{ds} \pi D}$ $\geq \frac{(310 \text{ kips}) \sqrt{2}}{0.75(0.60)(70 \text{ ksi})(1.0 + 0.5 \sin^{1.5} 90^\circ) \pi (6.625 \text{ in.})}$ $\geq 0.446 \text{ in.}$	$F_{wc} = F_w / \Omega_w$ $F_w = 0.60 F_{EXX}$ $\Omega_w = 2.00$ $w \geq \frac{P_a \sqrt{2}}{(F_w / \Omega_w) k_{ds} \pi D}$ $\geq \frac{(206 \text{ kips}) \sqrt{2}}{[0.60(70 \text{ ksi}) / 2.00](1.0 + 0.5 \sin^{1.5} 90^\circ) \pi (6.625 \text{ in.})}$ $\geq 0.444 \text{ in.}$

Use a 1/2 in. fillet weld.

Rupture strength of end plate at weld

$$t_{min} = \frac{3.09D}{F_u} \quad (\text{Manual Eq. 9-6})$$

$$= \frac{3.09(8 \text{ sixteenths})}{65 \text{ ksi}}$$

$$= 0.380 \text{ in.} < t_p = 3/4 \text{ in.} \quad \mathbf{o.k.}$$

Strength of HSS in tension

Tensile yielding:

$$P_n = F_y A_g \quad (\text{Spec Eq. D2-1})$$

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(50 \text{ ksi})(6.88 \text{ in.}^2)$ $= 310 \text{ kips} > 150 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_t = 1.67$ $\frac{P_n}{\Omega_t} = \frac{(50 \text{ ksi})(6.88 \text{ in.}^2)}{1.67}$ $= 206 \text{ kips} > 100 \text{ kips} \quad \mathbf{o.k.}$

Tensile rupture:

$$A_e = A_g$$

$$= 6.88 \text{ in.}^2$$

$$P_n = F_u A_e \quad (\text{Spec. Eq. D2-2})$$

$$= (62 \text{ ksi})(6.88 \text{ in.}^2)$$

$$= 427 \text{ kips}$$

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(427 \text{ kips})$ $= 320 \text{ kips} > 150 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{427 \text{ kips}}{2.00}$ $= 214 \text{ kips} > 100 \text{ kips} \quad \mathbf{o.k.}$

EXAMPLE 7.4—End Plate on Rectangular HSS Connection (Bolts on Four Sides)

Given:

Determine the end-plate thickness, weld size, and the number of bolts required to resist a tensile dead load $P_D = 30$ kips and a live load, $P_L = 80$ kips for the end plate shown in Figure 7-21. Use an ASTM A500/A500M Grade C HSS6×6× $\frac{3}{8}$ section, an ASTM A572/A572M Grade 50 end plate, $\frac{3}{4}$ -in.-diameter Group 120 bolts, and 70 ksi weld filler metal.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

HSS

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50$ ksi

$F_{up} = 65$ ksi

From AISC *Manual* Table 1-12, the HSS geometric properties are as follows:

HSS6×6× $\frac{3}{8}$

$H = 6.00$ in.

$B = 6.00$ in.

$t = 0.349$ in.

$A = 7.58$ in.²

Solution:

From ASCE/SEI 7, Chapter 2, the required tensile strength is:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(30 \text{ kips}) + 1.6(80 \text{ kips})$ $= 164 \text{ kips}$	$P_a = P_D + P_L$ $= 30 \text{ kips} + 80 \text{ kips}$ $= 110 \text{ kips}$

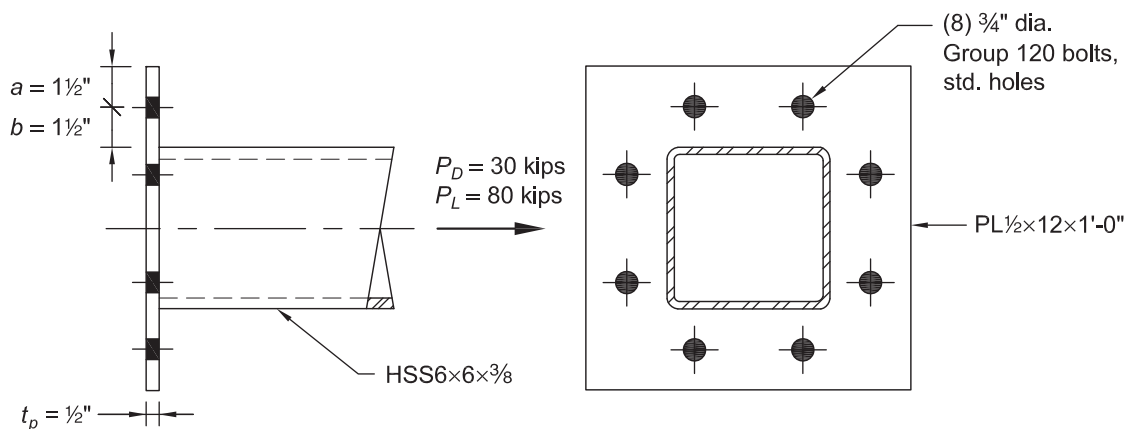


Fig. 7-21. End plate-to-square HSS end connection for Example 7.4.

Find the required end-plate thickness, t_p

From Figure 7-21,

$$a = 1\frac{1}{2} \text{ in.}$$

$$b = 1\frac{1}{2} \text{ in.}$$

Check $a \leq 1.25b$; if not, set $a = 1.25b$

$$\begin{aligned} 1.25b &= 1.25(1\frac{1}{2} \text{ in.}) \\ &= 1.88 \text{ in.} \geq a = 1\frac{1}{2} \text{ in.} \quad \mathbf{o.k.} \end{aligned}$$

Calculate α' , b' , ρ , and d' :

$$\begin{aligned} a' &= a + d_b/2 && \text{(from Manual Eq. 9-23)} \\ &= 1\frac{1}{2} \text{ in.} + \frac{\frac{3}{4} \text{ in.}}{2} \\ &= 1.88 \text{ in.} \end{aligned}$$

$$\begin{aligned} b' &= b - d_b/2 && \text{(from Manual Eq. 9-24)} \\ &= 1\frac{1}{2} \text{ in.} - \frac{\frac{3}{4} \text{ in.}}{2} \\ &= 1.13 \text{ in.} \end{aligned}$$

$$\begin{aligned} \rho &= \frac{b'}{a'} && \text{(Manual Eq. 9-29)} \\ &= \frac{1.13 \text{ in.}}{1.88 \text{ in.}} \\ &= 0.601 \end{aligned}$$

$$\begin{aligned} d_h &= d_b + \frac{1}{16} \text{ in.} \\ &= \frac{3}{4} \text{ in.} + \frac{1}{16} \text{ in.} \\ &= 0.813 \text{ in.} && (7-17) \end{aligned}$$

$$\begin{aligned} p &= \frac{2(B + H + \pi b)}{n} \\ &= \frac{2[6.00 \text{ in.} + 6.00 \text{ in.} + \pi(1\frac{1}{2} \text{ in.})]}{8} \\ &= 4.18 \text{ in.} \end{aligned}$$

Check p_{max} :

$$\begin{aligned} p_{max} &= 4\sqrt{b'(a+b)} \\ &= 4\sqrt{(1.13 \text{ in.})(1\frac{1}{2} \text{ in.} + 1\frac{1}{2} \text{ in.})} \\ &= 7.36 \text{ in.} > p = 4.18 \text{ in.} \quad \mathbf{o.k.} \end{aligned}$$

$$\begin{aligned} \delta &= 1 - \frac{d_h}{p} && \text{(from Manual Eq. 9-28)} \\ &= 1 - \frac{0.813 \text{ in.}}{4.18 \text{ in.}} \\ &= 0.806 \end{aligned}$$

The required tensile force per bolt with no prying, T_r , is:

LRFD	ASD
$T_u = \frac{P_u}{n}$ $= \frac{164 \text{ kips}}{8 \text{ bolts}}$ $= 20.5 \text{ kips/bolt}$	$T_a = \frac{P_a}{n}$ $= \frac{110 \text{ kips}}{8 \text{ bolts}}$ $= 13.8 \text{ kips/bolt}$

From AISC *Manual* Table 7-2, the available tensile strength per bolt is:

LRFD	ASD
$T_c = \phi r_n$ $= 29.8 \text{ kips/bolt} > T_u = 20.5 \text{ kips/bolt} \quad \text{o.k.}$	$T_c = r_n / \Omega$ $= 19.9 \text{ kips/bolt} > T_a = 13.8 \text{ kips/bolt} \quad \text{o.k.}$

Calculate the prying ratio, β :

$$\beta = \frac{1}{\rho} \left(\frac{T_c}{T_r} - 1 \right) \quad (\text{Manual Eq. 9-35})$$

LRFD	ASD
$\beta = \frac{1}{0.601} \left(\frac{29.8 \text{ kips}}{20.5 \text{ kips}} - 1 \right)$ $= 0.755$	$\beta = \frac{1}{0.601} \left(\frac{19.9 \text{ kips}}{13.8 \text{ kips}} - 1 \right)$ $= 0.735$

Because $\beta < 1$,

$$\begin{aligned} \alpha^* &= \frac{1}{\delta} \left(\frac{\beta}{1-\beta} \right) \leq 1 && (\text{from Manual Eq. 9-36b}) \\ &= \frac{1}{0.806} \left(\frac{0.755}{1-0.755} \right) \leq 1 \\ &= 3.82 > 1 \\ &= 1.00 \end{aligned}$$

Calculate t :

$$t = t_c \sqrt{\frac{T_r}{T_c} \left(\frac{1}{1 + \delta \alpha^*} \right)} \quad (7-19)$$

LRFD	ASD
$t_c = \sqrt{\frac{4T_c b'}{\phi_b p F_u}} \quad (\text{Manual Eq. 9-30a})$ $\phi_b = 0.90$ $t_c = \sqrt{\frac{4(29.8 \text{ kips})(1.13 \text{ in.})}{0.90(4.18 \text{ in.})(65 \text{ ksi})}}$ $= 0.742 \text{ in.}$ $t = (0.742 \text{ in.}) \sqrt{\frac{20.5 \text{ kips}}{29.8 \text{ kips}} \left[\frac{1}{1 + 0.806(1.00)} \right]}$ $= 0.458 \text{ in.}$	$t_c = \sqrt{\frac{4\Omega_b T_c b'}{p F_u}} \quad (\text{Manual Eq. 9-30b})$ $\Omega_b = 1.67$ $t_c = \sqrt{\frac{4(1.67)(19.9 \text{ kips})(1.13 \text{ in.})}{(4.18 \text{ in.})(65 \text{ ksi})}}$ $= 0.744 \text{ in.}$ $t = (0.744 \text{ in.}) \sqrt{\frac{13.8 \text{ kips}}{19.9 \text{ kips}} \left[\frac{1}{1 + 0.806(1.00)} \right]}$ $= 0.461 \text{ in.}$

Use a 1/2-in.-thick end plate.

Calculate α' :

LRFD	ASD
$\alpha' = \frac{1}{\delta(1+\rho)} \left[\left(\frac{t_c}{t} \right)^2 - 1 \right]$ $= \frac{1}{0.806(1+0.601)} \left[\left(\frac{0.742 \text{ in.}}{1/2 \text{ in.}} \right)^2 - 1 \right]$ $= 0.932 < 1.5 \quad \text{o.k.}$	$\alpha' = \frac{1}{\delta(1+\rho)} \left[\left(\frac{t_c}{t} \right)^2 - 1 \right]$ $= \frac{1}{0.806(1+0.601)} \left[\left(\frac{0.744 \text{ in.}}{1/2 \text{ in.}} \right)^2 - 1 \right]$ $= 0.941 < 1.5 \quad \text{o.k.}$

Weld between HSS and end plate

The length of weld is:

$$\begin{aligned}
 l_w &= 2(B + H) \\
 &= 2(6.00 \text{ in.} + 6.00 \text{ in.}) \\
 &= 24.0 \text{ in.}
 \end{aligned}$$

Note that $k_{ds} = 1.0$ because the fillet weld is connected to a rectangular HSS that is loaded in tension.

The minimum weld size required is:

LRFD	ASD
$F_{wc} = \phi_w F_w$ $F_w = 0.60 F_{EXX}$ $\phi_w = 0.75$ From Equation 7-20a: $w \geq \frac{P_u \sqrt{2}}{\phi_w F_w l_w}$ $= \frac{(164 \text{ kips}) \sqrt{2}}{0.75(0.60)(70 \text{ ksi})(24.0 \text{ in.})}$ $= 0.307 \text{ in.}$	$F_{wc} = F_w / \Omega_w$ $F_w = 0.60 F_{EXX}$ $\Omega_w = 2.00$ From Equation 7-20b: $w \geq \frac{P_a \sqrt{2}}{(F_w / \Omega_w) l_w}$ $= \frac{(110 \text{ kips}) \sqrt{2}}{[0.60(70 \text{ ksi}) / 2.00](24.0 \text{ in.})}$ $= 0.309 \text{ in.}$

Use a $\frac{5}{16}$ in. fillet weld.

Rupture strength of end plate at weld

$$t_{min} = \frac{3.09D}{F_u} \quad (\text{Manual Eq. 9-6})$$

$$= \frac{3.09(5 \text{ sixteenths})}{65 \text{ ksi}}$$

$$= 0.238 \text{ in.} < t_p = \frac{1}{2} \text{ in.} \quad \mathbf{o.k.}$$

Strength of HSS in tension

Tensile yielding:

$$P_n = F_y A_g \quad (\text{Spec Eq. D2-1})$$

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(50 \text{ ksi})(7.58 \text{ in.}^2)$ $= 341 \text{ kips} > 164 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_t = 1.67$ $\frac{P_n}{\Omega_t} = \frac{(50 \text{ ksi})(7.58 \text{ in.}^2)}{1.67}$ $= 227 \text{ kips} > 110 \text{ kips} \quad \mathbf{o.k.}$

Tensile rupture:

$$A_e = A_g$$

$$= 7.58 \text{ in.}^2$$

$$P_n = F_u A_e \quad (\text{Spec. Eq. D2-2})$$

$$= (62 \text{ ksi})(7.58 \text{ in.}^2)$$

$$= 470 \text{ kips}$$

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(470 \text{ kips})$ $= 353 \text{ kips} > 164 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_t = 2.00$ $\frac{P_n}{\Omega_t} = \frac{470 \text{ kips}}{2.00}$ $= 235 \text{ kips} > 110 \text{ kips} \quad \mathbf{o.k.}$

Chapter 8

Line Loads and Concentrated Forces on HSS

8.1 SCOPE AND BASIS

This chapter is intended to cover local “line loads” applied to the face of an HSS or to the end of an HSS member via a cap plate. The line loads are typically applied by a welded branch plate, oriented either longitudinal or transverse to the main HSS member axis, with the plate generally loaded in axial tension or compression. Strong-axis bending moments applied to branch plates are also covered where information is available. For axial loading and moment loading combinations to branch plates, a linear interaction between these two load effects is advocated. Through plates to both rectangular and round HSS main members, with the plate subjected to axial loading, are also considered. The case of shear loading on branch plates or through plates is covered in Chapter 5.

The design methods in this chapter comply with AISC *Specification* Chapters J and K, the AISC *Specification* Commentary, and the AISC *Manual*, whenever guidance is given by these documents. In general, AISC has adopted rational yield-line models for branch plate-to-HSS connections (for flexure of the connecting face of a rectangular HSS), extrapolated the web behavior of a wide-flange or I-shaped section (to apply to the two webs of a rectangular HSS), and included a set of empirical strength-based failure modes (along with their limits of applicability) that were proposed by the International Institute of Welding (IIW, 1989). Thus, the AISC design recommendations, as encompassed by AISC *Specification* Chapters J and K and elaborated upon in this chapter, largely resemble the IIW Second Edition (1989) recommendations that were in turn included in CIDECT Design Guides 1 and 3, First Editions (Wardenier et al., 1991; Packer et al., 1992). Since that time, a broad reevaluation of branch plate-to-HSS connection design rules has been made, resulting in the IIW Third Edition (2012) recommendations that have in turn been incorporated into CIDECT Design Guides 1 and 3, Second Editions (Wardenier et al., 2008a; Packer et al., 2009), ISO (2013), and Wardenier et al. (2010). These IIW Third Edition (2012) design recommendations, and their derivatives, were influenced by considerable recent numerical research and also include a connection deformation limit as an ultimate limit state (Wardenier et al., 2008b).

As pointed out in the AISC *Specification* Chapter K Commentary, and also discussed in Section 2.2 of this Design Guide, AISC has not adopted this connection deformation limit for HSS because it would be inconsistent with the remainder of the AISC *Specification*, which does not include deformation limit states for connections. Most recently, plate-to-round HSS welded connections have been

re-assessed (Wardenier et al., 2018), to generate a pragmatic update of the current edition of Eurocode 3 (CEN, 2005). The HSS connections part of this code (EN 1993-1-8) are now in an advanced draft form (CEN, 2021).

Wide-flange beam-to-HSS column directly welded moment connections are also indirectly covered by the scope of this chapter. The flexural capacity of such connections can be determined by ignoring the beam web and considering the beam flanges as a pair of transverse plates welded to the HSS. The moment capacity of the connection—which would be semi-rigid or PR—can then be computed by multiplying the capacity of one plate-to-HSS connection by the lever arm between the centroids of the two flanges (the beam depth minus one flange thickness). It is believed that this model is conservative, even for shallow beams, because the contribution of the beam web to the moment capacity is ignored. In the analysis of a structure using PR connections, the force-deformation response characteristics of the PR connections need to be included [AISC *Specification* Section B3.4b(b)]. Besides the strength limit state (moment capacity of a connection), the connection stiffness and deformation capacity need to be considered. Although some experimental testing of wide-flange beam-to-HSS column moment connections has been performed, the force-deformation response characteristics of such PR connections—which is a topic beyond the scope of this Design Guide—are best obtained by analytical means. Finite element analysis is appropriate, or the “Component Method” (which is now adopted for connections in Eurocode 3) can be used. Guidance on the latter, for application to hollow section connections, is available in Jaspart et al. (2005) and Weynand et al. (2015). Moment connections are addressed in Chapter 6 of this Design Guide.

When branch plates are oriented longitudinal to the HSS member axis, it is assumed that the plate is aligned with the HSS member axis. However, if a longitudinal plate is slightly offset laterally from the centerline—perhaps so that a beam or diagonal bracing member centerline can coincide with the HSS column centerline—the difference in connection capacity is small and can be ignored in practice. For the limit state of wall plastification for an offset longitudinal plate-to-rectangular HSS connection, yield-line solutions are provided in Part 9 of the AISC *Manual* for the branch plate under axial load or in-plane bending moment. Note that although the generic out-of-plane branch bending case is also covered in Figure 9-5 of the AISC *Manual*, the flexural capacity of a plate connection loaded by a moment that bends the plate out of its plane is so low that it can be considered to be zero (as is given in Tables 8.1 and 8.2).

The local loads applied by branch plates to HSS (which excludes the cap-plate case) are assumed to be remote from the chord end, or at least a distance of l_{end} from an open chord end, where:

For axially loaded plates to rectangular HSS chords with $\beta \leq 0.85$:

$$l_{end} \geq B\sqrt{1-\beta} \quad (\text{Spec. Eq. K1-7})$$

For axially loaded plates to round HSS chords:

$$l_{end} \geq D(1.25 - 0.5\beta) \quad (\text{Spec. Eq. K1-8})$$

The l_{end} inequality for round HSS chords provides a sensible end distance requirement for all β values; however, the inequality for rectangular HSS chords (Equation 2-5 and AISC *Specification* Equation K1-7) is a closed-form yield-line solution, validated by experiments (Fan and Packer, 2017a, 2017b) over a β range for which flexural failure of the chord face governs (i.e., $\beta < \text{approximately } 0.85$). Further explanation is given in Chapter 2.

In cases where the plate branch is a distance less than l_{end} from an open chord end, and the concentrated load cannot be dispersed to either side of the connection, a cap plate welded to the chord end can be used to restore the full connection strength (see Section 8.4.1). Alternatively, a reduced connection strength (typically around 50%) should be assumed, unless a specific capacity is available (e.g., Equation 2-4).

8.2 LIMIT STATES

A considerable amount of experimental and numerical research has been performed on branch plate-to-HSS connections (Kurobane, 1981; Wardenier et al., 1981; Davies and Packer, 1982; Makino et al., 1991; Cao et al., 1998a; Kostas and Packer, 2003a; Voth and Packer, 2012a, 2012b, 2012c, 2012d; Wardenier et al., 2018), stiffened branch

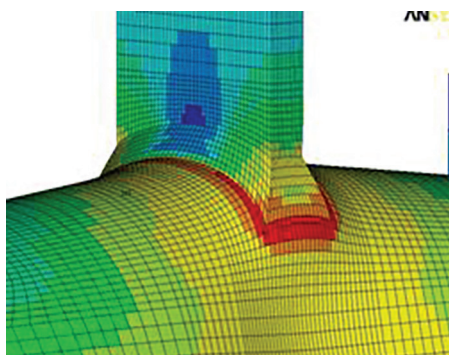
plate-to-HSS connections (Kostas and Packer, 2003b), and through plate-to-HSS connections (Kostas and Packer, 2003a; Voth and Packer, 2016). These studies have highlighted the pertinent limit states, described in Sections 8.2.1 to 8.2.4, that have been adopted in the AISC *Specification*.

8.2.1 Branch Plates-to-Round HSS

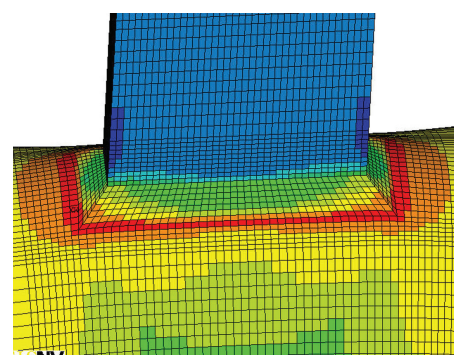
Most contemporary HSS connection design guides (Wardenier et al., 2008a; IIW, 2012; ISO, 2013) identify two limit state checks (wall plastification, Section 2.3.4, and chord punching shear, Section 2.4.2) for branch plate-to-round HSS connections. Available strengths of such connections are given in AISC *Specification* Table K2.1, and because they are empirical for the HSS plastification limit state, they are subject to the limits of applicability given in Table K2.1A. Figures 8-1(a) and 8-1(b), as well as Figure 2-8(b), illustrate the high degree of chord deformation associated with the HSS plastification failure mode. Table 8-1 in this Design Guide presents available strengths for transverse and longitudinal plate connections subject to branch axial loading and branch moment loading (in the plane of the connection, or out of the plane of the connection), subject to the geometric and material limits of Table 8-1A (both tables are in Section 8-3). In Table 8-1, the plate is assumed to be perpendicular to the round HSS chord member, and the applied branch load is assumed to act in the plane of the plate, although for the axial load case, the force could be applied at an acute angle to the HSS. In the latter situation the force component normal to the HSS is considered to dominate the connection behavior, and the connection nominal strength is given in terms of $R_n \sin \theta$. Example 8.2 demonstrates the design procedure for a transverse plate connection. The AISC *Manual Companion Design Examples* (AISC, 2023a) also includes a design example for a longitudinal plate connection (Example K.8).



(a) Post-test bisection of a transverse plate connection experiment



(b) Finite element model of a transverse plate connection, showing the hot spot at the saddle point



(c) Finite element model of a longitudinal plate connection, showing the uniform stress intensity around the perimeter

Fig. 8-1. Branch plate-to-round HSS T-connections.

The design recommendations in AISC *Specification* Table K2.1, and Table 8-1 herein, make no distinction between branch plate loading in axial tension and compression. For plate-to-round HSS connections, the ultimate connection capacity is usually higher under plate tension loading than under plate compression loading, as shown by the two lower lines in Figure 8-2. For compression loading, the ultimate capacity is often associated with very large connection deformations. For tension loading, there may sometimes be early cracking or little deformation capacity beyond the connection available strength. The design capacity or available strength is therefore limited by a maximum deformation criterion, and furthermore—following the example of round-to-round HSS welded T-, Y-, and cross-connections in Chapter 9—the same available strength is conservatively used for both plate axial tension and compression load cases.

8.2.2 Branch Plates-to-Rectangular HSS

The following limit states are, in general, applicable for branch plate-to-rectangular HSS connections: chord face plastification (Section 2.3.2), chord punching shear (Section 2.4.1), local yielding of the branch plate due to uneven load distribution (Section 2.5), local yielding of the chord side walls (Section 2.6), local crippling of the chord side

walls (Section 2.7), and buckling of the chord side walls (Section 2.8). These are not tabulated in the AISC *Specification*, but Section K2.3 refers to the “applicable limit states from Chapter J.” No distinction is made between branch plates in axial tension and compression, and this decision is supported by connection behavior observed experimentally and numerically. Figure 8-3 illustrates some of these limit states for longitudinal and transverse plate connections.

Longitudinal Plate Connections

These connections are characterized by a very low branch-to-chord width ratio, β , and the strength is therefore governed by just the limit state of chord face plastification, as given by Equation 2-3 and in Table 8-2 (see Section 8-3). Because this equation is derived from a closed-form analytical (yield-line) solution, there are no limits of applicability. In Table 8-2, the longitudinal plate is assumed to be perpendicular to the rectangular HSS chord member, and the applied branch load is assumed to act in the plane of the plate, although the force could be applied at an acute angle to the HSS (such as in a connection to a diagonal brace member). In the latter situation, the force component normal to the HSS is considered to dominate the connection behavior, and the HSS plastification nominal strength is given in

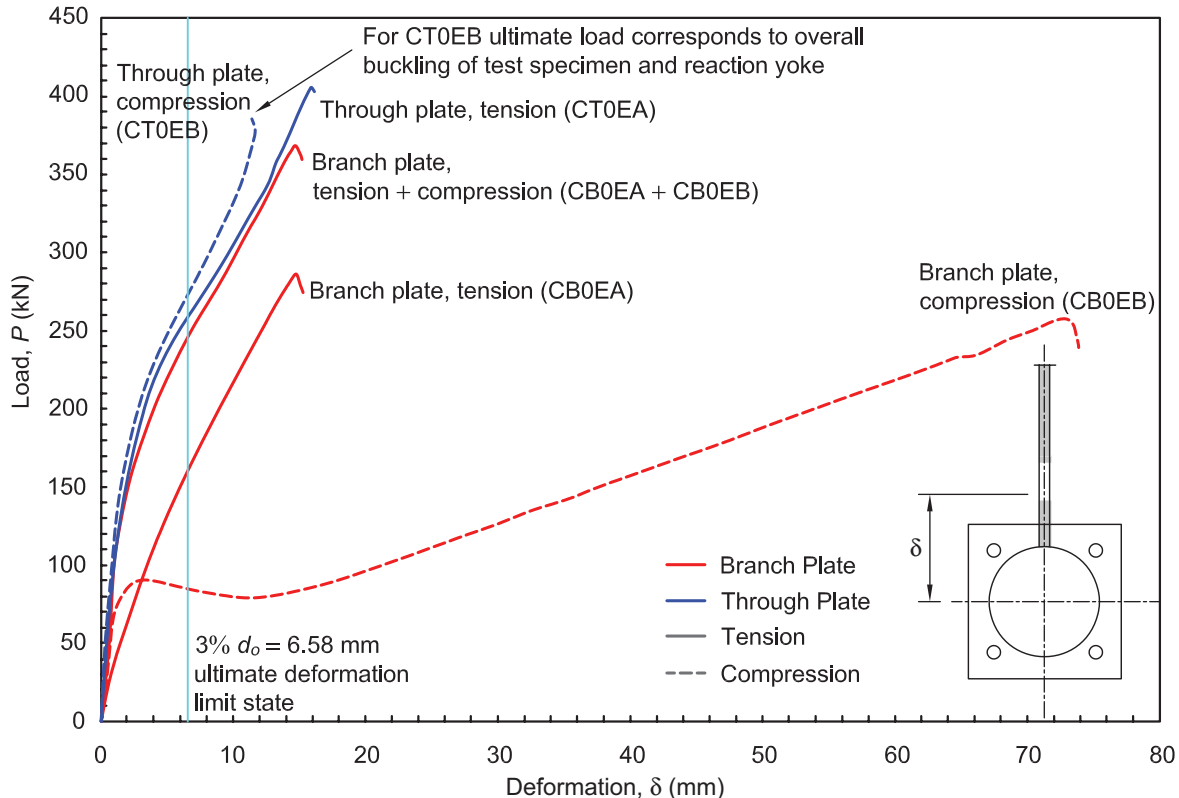


Fig. 8-2. Branch tension vs. compression experimental behavior for a longitudinal plate-to-round HSS T-connection.

terms of $R_n \sin \theta$. Note that longitudinal plate connections in which the plate is predominantly subject to shear (i.e., shear tab connections) are covered in Chapter 5. Example 8.1 demonstrates the design procedure for a longitudinal plate connection.

Transverse Plate Connections

All six potential limit states are listed in Table 8-2. To achieve proper welding of full-width ($\beta = 1.0$) plate connections, contouring of the plate extremities may be required to fit to the HSS corners, especially if the HSS wall thickness is large. As with transverse plates fitted to round HSS, the plate is generally not contoured to a point, and instead the outer edges of the plate are clipped. If the width of a transverse plate exceeds the width, B , of the HSS member (which is not recommended), the maximum plate width that can be used in calculations (Table 8-2) is $B_p = B$, and $\beta_{max} = 1.0$. Example 8.3 demonstrates the design procedure for a transverse plate connection.

Local yielding of the branch and chord face plastification

It is interesting to examine the limit state of local yielding of the branch plate due to uneven load distribution further. This is illustrated in Figure 2-12, and the plate effective width, B_e , is given by Equation 2-14 or AISC *Specification* Equation K1-1. This equation can be rewritten in terms of plate nominal strength, R_n :

$$\begin{aligned} R_n &= F_{yp} A_{eb} \\ &= F_{yp} B_e t_p \end{aligned} \quad (8-1)$$

where A_{eb} is the branch effective area (AISC *Specification* Commentary Table C-K1.1). Thus,

$$R_n = \left(\frac{10}{B/t} \right) F_{yp} t_p B_p \leq F_{yp} t_p B_p \quad (8-2a)$$

or

$$R_n = 10 F_{yp} t^2 \beta \leq F_{yp} t_p B_p \quad (8-2b)$$

where B_p , t_p , and F_{yp} refer to the plate width, thickness, and yield strength, respectively. Recognizing that $\eta = t_p/B$, Equation 2-1 for the limit state of chord face plastification can be plotted in a non-dimensional form ($R_n/F_{yp} t^2$) versus β and compared to Equation 8-2b. Figure 8-4 shows that local yielding of the branch plate, due to the effective width phenomenon, always governs over plastification of the HSS connecting face. For this reason, chord plastification was omitted as a limit state check in international design recommendations for transverse plate-to-HSS connections in the 20th century [e.g., Packer et al. (1992), Packer and Henderson (1997)]. This limit state was also omitted from Table K1.2 in the 2010 AISC *Specification* because the provisions in this edition were based on these CIDECT/IIW recommendations.

Relatively recently, it has been recognized that the normal stress level in the chord connecting face can influence the connection capacity, and this effect is captured by the Q_f term in Equation 2-2. Thus, it became apparent that it was theoretically possible for the chord plastification limit state to govern if $Q_f < 1.0$ and β and B/t_p have particular values. The separation of the available strengths for the two failure modes shown in Figure 8-4 is actually greater because of the different resistance factors assigned ($\phi = 1.00$ for HSS plastification by yield-line analysis and $\phi = 0.95$ for local yielding of the plate). Plastification of the HSS connecting face has



(a) Chord face plastification for a longitudinal plate connection



(b) Chord face plastification, combined with chord punching shear and local yielding of the chord side walls, for a transverse plate connection

Fig. 8-3. Failure modes associated with branch plate-to-rectangular HSS T- and cross-connections.

therefore been included as a viable limit state for transverse plate-to-rectangular HSS connections in very recent international design recommendations [e.g., Packer et al. (2009) and ISO (2013)], albeit with a modernized Q_f -function. Thus, plastification of the HSS connecting face has been included as a limit state since the 2016 AISC *Specification*. Q_f is now given in AISC *Specification* Section K1.3 for rectangular chords, and this limit state is included in Table 8-2 herein.

Chord punching shear

In addition to the relationship between the two limit states or failure modes in Figure 8-4, the relationship with chord punching shear (described in Section 2.4.1) is also interesting. For the case of a transverse plate at 90° to the chord, Equation 2-10 simplifies to:

$$R_n = 0.6F_y t (2t_p + 2B_{ep}) \quad (8-3a)$$

as also presented in the AISC *Specification* Commentary Table C-K1.1, or it can be expressed as:

$$R_n = F_y t^2 \left(12\beta + 1.2 \frac{t_p}{t} \right) \quad (8-3b)$$

where $B_{ep} \leq B_p$. A conservative simplification of Equation 8-3b would be to take $t_p = 0$. If this is then plotted on Figure 8-4, punching shear would then lie above the local yielding of the branch plate line for all β . Chord punching shear, based on shear yielding as used here, has a resistance factor of $\phi = 1.00$ [AISC *Specification* Section J4.2(a)]. Thus,

the chord punching shear available strength can be expected to never control design for normal parameter combinations.

The limit state of chord punching shear also has to be physically possible, and the branch plate must be able to punch out a patch of the HSS connecting face; thus, the plate width must be such that $B_p \leq B - 2t$. This limit state has also been observed, in laboratory experiments, to often occur in conjunction with flexure of the HSS connecting face, as illustrated in Figure 8-3(b). Thus, a combined failure mode for the chord face—consisting of both chord face plastification and chord punching shear—has been proposed. Davies and Packer (1982) derived a solution for such a combined failure mechanism, but it was too complex for routine design, and design recommendations have opted instead for discrete individual limit state checks. Importantly though, Davies and Packer showed that the formulation to capture local yielding of the branch plate, Equation 8-2a or Equation 8-2b, represented a lower bound to both the combined plastification-and-punching mechanism as well as experimental results, further substantiating the discussion regarding these three failure modes.

Local yielding of the chord side walls

This limit state, also illustrated in Figure 8-3(b), can occur with the branch plate loaded in either tension or compression when β is close to unity. It is manifested by yielding of the two HSS side walls, or webs, along a short, dispersed load length directly beneath the branch. A “four-hinge model” for this failure mode was proposed (Packer, 1978), as shown in Figure 8-5, then simplified (Wardenier, 1982) by omitting the

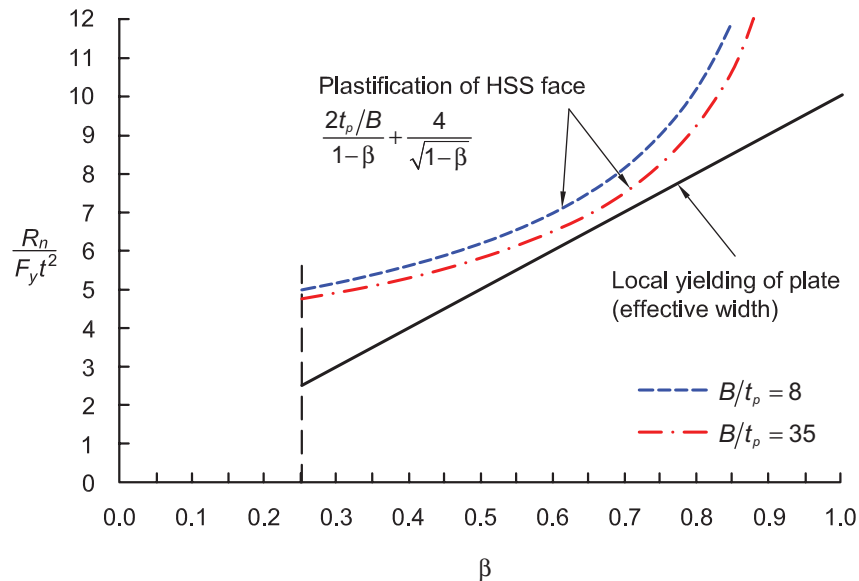


Fig. 8-4. Interaction between the limit states of plastification of the HSS face and local yielding of the branch plate for transverse plate-to-rectangular HSS connections.

work done in the four plastic hinges transverse to the chord to conform to the model for bearing on wide-flange webs. This results in Equation 2-17, where the bearing length, l_b , is equal to the plate thickness, t_p . This model produces very good correlation with test results on plate-to-rectangular HSS connections, and the use of $\phi = 1.00$ in AISC *Specification* Section J10.2 is supported (Fan, 2017).

Chord web crippling and buckling

These two limit states pertain to full-width branch plates loaded in compression, but neither has been observed as a failure mode in experiments on plate-to-rectangular HSS welded connections. Also, for the same tests, Equations 2-19 and 2-23 were found not to govern capacity prediction over Equation 2-17 because of the low bearing length (Fan, 2017), as discussed in Sections 2.6 through 2.8. Nevertheless, these two limit states are included in Table 8-2 because the range of test parameters investigated is not exhaustive.

Limits of applicability

Table K1.2A in the 2010 AISC *Specification* gave a range of validity for a number of geometric and mechanical properties for which all of Table K1.2 would be valid. However, this was a conservative stance because all limits would not apply to all limit states in Table K1.2 of that edition. Table 8-2A (see Section 8.3) gives geometric and material limitations where there are known empirical bounds on parameters for certain limit state expressions.

8.2.3 Through Plates-to-Round HSS

A plate-to-tube connection in which the plate is slotted through the HSS and welded to two opposite faces of the tube [Figure 8-6(a)] is a widely used stiffening and strengthening method. This has the advantage of engaging much more of the cross section in load resistance, and hence has a higher capacity than the equivalent branch plate connection. However, the through-plate connection is a more difficult and expensive fabrication procedure and is therefore not favored by fabricators (Sherman, 1996). Also, a part of the through plate protrudes beyond the far side of the HSS [Figure 8-6(a)], which may affect connections to that side of the HSS. Fillet welding of the plate is generally possible, and the welds can be sized by apportioning half of the branch load to each side of the HSS. Note that the plate is axially loaded in this case, whereas the longitudinal through plates considered in Chapter 5, Figure 5.8 and Example 5.3 are loaded in shear.

An experimental study (Voth and Packer, 2012a) of through plate-to-round HSS connections indicated that their behavior is actually comprised of two independent mechanisms—one in compression and one in tension, which occur on opposite sides of the hollow section chord during load application, as can be seen in Figure 8-6(b). Examination of this behavior also revealed that the capacity of a through-plate connection is approximately the summation of the capacities of a branch plate-to-round HSS connection tested in tension and a branch plate-to-round HSS connection tested in compression, provided that connection geometry for the branch

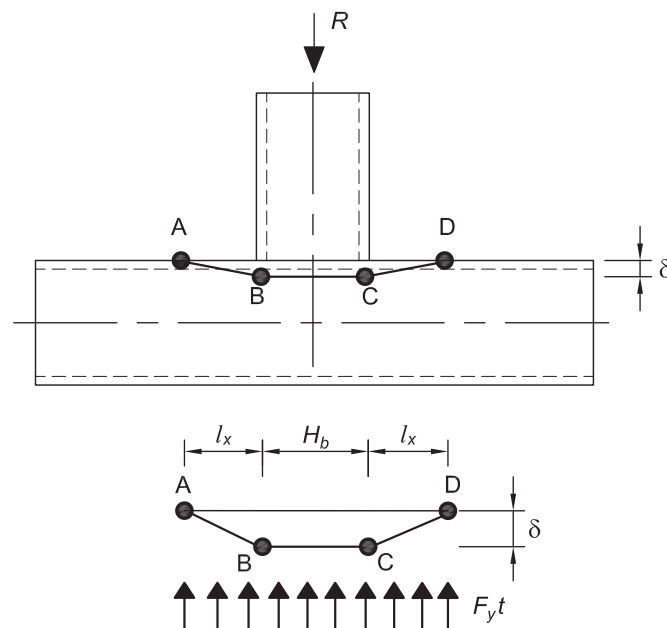


Fig. 8-5. Model for bearing failure of HSS webs (Packer, 1978).

connection and the through connection are similar. This is shown in Figure 8-2. As expected, connection load versus deformation behavior is very similar for through plates loaded in axial tension and compression. Further numerical “tests” were then generated in a finite element modeling study, resulting in expressions for the capacity of longitudinal and transverse through-plate connections, each of which was based on the sum of unique equations for the tension and compression capacity of the corresponding branch-plate connections (Voth and Packer, 2016). In the most recent evolution of these recommendations (Wardenier et al., 2018), some simplifications have been made for design purposes

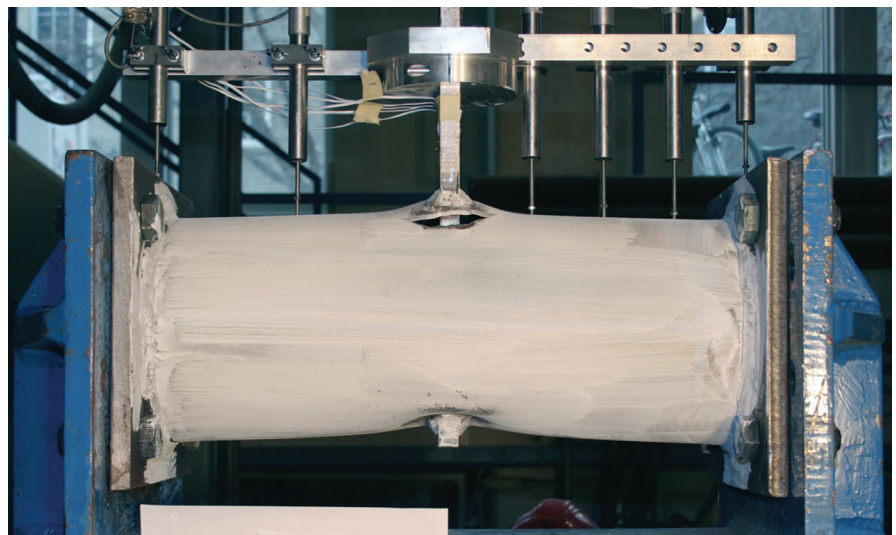
(including using through-plate capacity as twice the newly revised compression-loaded branch-plate capacity) and it is these through-plate nominal strength equations that are given in Table 8-1.

8.2.4 Through Plates and Stiffened Plates to Rectangular HSS

Given the option of orienting a branch plate in any direction, the transverse branch plate [Figure 8-7(d)] will generally give a higher available strength than the longitudinal alternate [Figure 8-7(a)]. However, there are cases where an increased connection strength may be desired while still



(a) Longitudinal through plate-to-round HSS connection in a structure



(b) Transverse through plate-to-round HSS connection in a laboratory experiment (at ultimate load)

Fig. 8-6. Examples of through plate-to-round HSS connections.

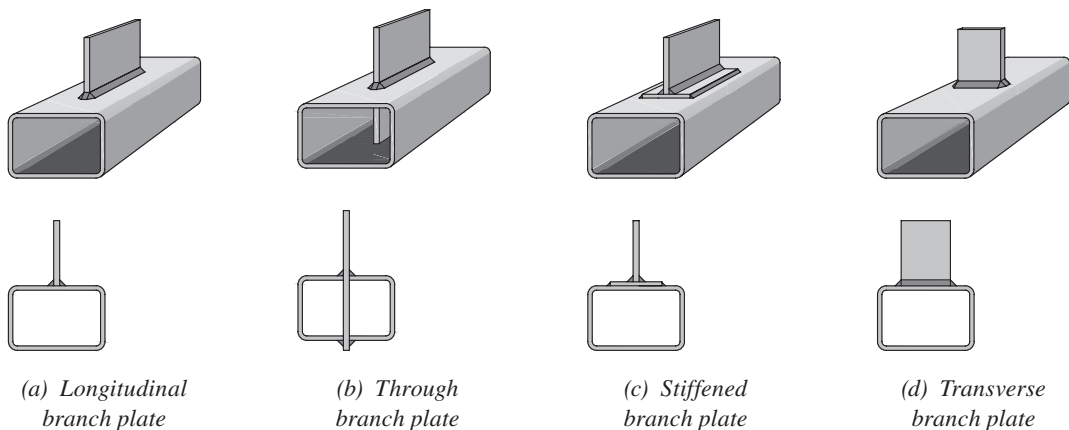


Fig. 8-7. Options for plate-to-rectangular HSS connections.

keeping the branch plate in the longitudinal orientation. An axially loaded through plate [Figure 8-7(b) and Figure 8-8] loads the two connected faces of a rectangular HSS in a near-equal manner, resulting in the same failure mechanism on both the top and bottom faces of the HSS. As a consequence, the through plate connection nominal strength has been shown to be twice that of the corresponding branch plate connection nominal strength (Kosteski and Packer, 2003a). This is reflected in Table 8-2 for the applicable limit states. Fillet welding of the plate is generally possible, and the welds can be sized by apportioning half of the branch axial load to each side of the HSS.

Figure 8-7(c) shows a connection solution in which the chord face is stiffened with a doubler plate. If this stiffening plate is made sufficiently thick, such that it behaves as a near-rigid base for the branch plate, the footprint of this stiffening plate can be considered to be equivalent to a rectangular HSS branch of the same outside dimensions, and the connection can then be analysed as a rectangular HSS-to-HSS T-connection (which is covered in Chapter 9). The stiffened connection will have a much higher available strength than its unstiffened counterpart because of the increased width ratio, β . The stiffening plate width is assumed to be less than the HSS width and to be welded all-around to the flat of the HSS. If the stiffening plate is fillet-welded to the chord, under branch-plate tension, the loading would tend to cause opening of the weld root, so the directional strength increase factor would be inapplicable. To obtain adequate stiffening plate rigidity, Kosteski and Packer (2003b) showed that a minimum stiffening plate thickness, t_{sp} , was required:

$$t_{sp} \geq 0.5t(e^{3\beta^*}) \quad (8-4a)$$

where

$$\beta^* = \frac{B_{sp} - t_p - 2w}{B - t} \quad (8-4b)$$

In Equation 8-4b, t_p refers to the thickness of a longitudinal branch (gusset) plate, B_{sp} is the width of the stiffening plate, w is the leg size of the fillet weld along the edge of the longitudinal branch plate, and the length of the stiffening plate is assumed to be the same as the longitudinal branch plate. Equation 8-4a was derived for a parameter range of $12 \leq B/t \leq 48$ and $0.1 \leq \beta^* \leq 0.66$. At the upper limit of β^* , it can be seen that the stiffening plate is very thick ($t_{sp} = 3.62t$). As a general rule, reinforcing connections is usually not the most economical option. Also, if connection design is delegated, the AISC *Code of Standard Practice* requires the contract documents to include bidding quantities for the reinforcing plates, and any deviation from the bidding quantity may require an adjustment in the contract price and schedule.

8.3 BRANCH-PLATE AND THROUGH-PLATE CONNECTION AVAILABLE STRENGTH TABLES

The available strength equations for plate connections to round and rectangular HSS as well as their limits of applicability are presented in Tables 8-1, 8-1A, 8-2, and 8-2A.

8.4 CAP-PLATE CONNECTIONS

8.4.1 Closure of an Open End

A plate, with dimensions slightly greater than the outside of an HSS, is frequently welded to the end of an HSS to cap the



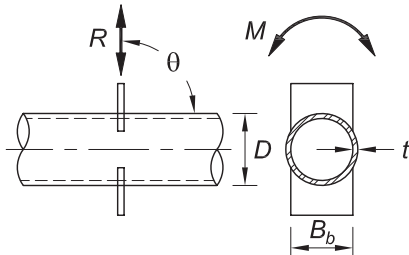
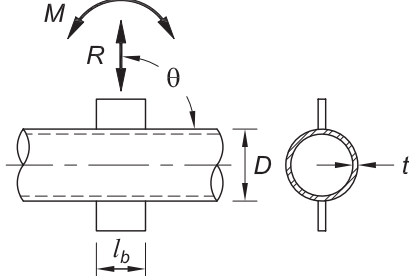
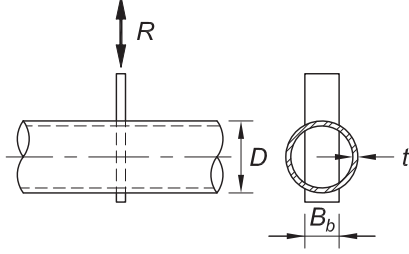
(a) Through plate with axial loading from hanger



(b) Through plate with axial loading from X-brace in tension or compression

Fig. 8-8. Through plate-to-rectangular HSS connections.

Table 8-1. Available Strengths of Plate-to-Round HSS Connections

Connection Type	Connection Available Strength ¹		Plate Bending ²	
	Plate Axial Load	In-Plane	Out-of-Plane	
Transverse plate T- and cross-connections 	Limit State 1: HSS plastification			
	$R_n \sin \theta = F_y t^2 \left(\frac{5.5}{1 - 0.81 \frac{B_b}{D}} \right) Q_f$ (K2-1a)	0	$M_n = 0.5 B_b R_n$ (K2-1b)	
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)			
	Limit State 2: HSS shear yielding (punching shear)			
	$R_n = 0.6 F_y t (2t_p + 2B_b)$	$M_{n-op} = 0.6 F_y t \left(\frac{B_b^2}{3} \right)$		
	$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)			
Longitudinal plate T-, Y-, and cross-connections 	Plate Axial Load		In-Plane	Out-of-Plane
	Limit State 3: HSS plastification			
	$R_n \sin \theta = 5.5 F_y t^2 \left(1 + 0.25 \frac{l_b}{D} \right) Q_f$ (K2-2a)	$M_n = 0.8 l_b R_n$ (K2-2b)	0	
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)			
	Limit State 4: HSS shear yielding (punching shear)			
	$R_n = 0.6 F_y t (2t_p + 2l_b)$	$M_{n-ip} = 0.6 F_y t \left(\frac{l_b^2}{3} \right)$		
$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)				
Transverse through-plate connections 	Plate Axial Load		In-Plane	Out-of-Plane
	Limit State 5: HSS plastification			
	$R_n = 4.6 F_y t^2 \left[1 + 3 \left(\frac{B_b}{D} \right)^2 \right] \left(\frac{D}{2t} \right)^{0.35} Q_f$	0	—	
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)			
	Limit State 6: HSS shear yielding (punching shear)			
	$R_n = 2R_n \text{ from Limit State 2}$	0	—	
$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)				

(Table 8-1 continues on the next page)

Table 8-1. Available Strengths of Plate-to-Round HSS Connections (continued)

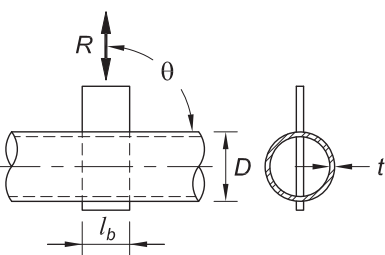
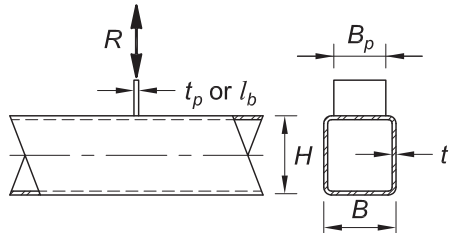
Connection Type	Connection Available Strength ¹	Plate Bending ²	
Longitudinal through plate T- and Y-connections 	Plate Axial Load	In-Plane	Out-of-Plane
	Limit State 7: HSS plastification		
	$R_n \sin \theta = 14.2 F_y t^2 \left[1 + 0.4 \left(\frac{l_b}{D} \right) \right] Q_f$	—	0
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)		
	Limit State 8: HSS shear yielding (punching shear)		
	$R_n = 2R_n$ from Limit State 4	—	0
	$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)		
Functions			
$Q_f = 1.0$ for HSS connecting surface(s) in tension			
$= 1.0 - 0.3U$ ($1 + U \leq 1.0$ for HSS connecting surface(s) in compression)			
$U = \left \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right \leq 1.0$			
where P_{ro} and M_{ro} are determined on the side of the connection that has the lower compression stress.			
P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD and P_a for ASD; $M_{ro} = M_u$ for LRFD and M_a for ASD.			
¹ Equation references are to the AISC Specification, where applicable			
² Where no formula is given, “0” represents zero available strength and “—” indicates that information is not available.			

Table 8-1A. Limits of Applicability for Limit States in Table 8-1

For Limit States 1, 3, 5, and 7:		
HSS wall slenderness:	$D/t \leq 50$ for T-connections under branch plate axial load or bending and for through-plate connections	(Spec. Table K2.1A)
	$D/t \leq 40$ for cross-connections under branch plate axial load or bending	(Spec. Table K2.1A)
Width ratio:	$0.2 < B_b/D \leq 1.0$ for transverse branch-plate connections	(Spec. Table K2.1A)
	$0.2 < B_b/D \leq 1 - 2t/D$ for transverse through-plate connections	
Material strength:	$F_y \leq 52$ ksi	(Spec. Table K2.1A)
Ductility:	$F_y/F_u \leq 0.8$ Note: ASTM A500/A500M Grade C is acceptable.	(Spec. Table K2.1A)
End distance:	$l_{end} \geq D \left(1.25 - \frac{B_b/D}{2} \right)$ for transverse and longitudinal branch-plate connections, and for through-plate connections	(Spec. Eq. K1-8)
Length ratio:	$0.6 \leq l_b/D \leq 4$ for longitudinal branch and through-plate connections	
For Limit States 2, 4, 6, and 8:		
Width ratio:	$B_b \leq D - 2t$, for transverse branch-plate and transverse through-plate connections	
End distance:	$l_{end} \geq D \left(1.25 - \frac{B_b/D}{2} \right)$ for transverse and longitudinal branch-plate connections, and for through-plate connections	(Spec. Eq. K1-8)

Table 8-2. Available Strengths of Plate-to-Rectangular HSS Connections

Connection Type	Connection Available Strength ¹	Plate Bending ²	
	Plate Axial Load	In-Plane	Out-of-Plane
Transverse plate T- and cross-connections  where $B_b = B_p$ and $\beta = B_b/B$ or B_p/B	Limit State 1: HSS plastification		
	$R_n = F_y t^2 \left[\frac{2t_p/B}{1-\beta} + \frac{4}{\sqrt{1-\beta}} \right] Q_f$ (2-2)	0	$M_n = F_y t^2 [X + Y] Q_f$ $X = \frac{0.5t_p(1+\beta)}{1-\beta}$ $Y = \sqrt{\frac{2BB_b(1+\beta)}{1-\beta}}$ (2-8)
	$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)		
	Limit State 2: HSS shear yielding (punching shear)		
	$R_n = 0.6F_y t (2t_p + 2B_{ep})$ $B_{ep} = \left(\frac{10t}{B} \right) B_b \leq B_b$ (J4-3), (K1-2), (2-11), and (8-3a)	0	—
	$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)		
	Limit State 3: Local yielding of the branch plate		
	$R_n = \left(\frac{10t}{B} \right) F_y t B_b \leq F_{yp} t_p B_b$ (K1-1), (2-14), (8-2a)	0	—
	$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)		
	Limit State 4: Local yielding of the HSS sidewalls		
	$R_n = 2F_y t (7.5t + t_p)$ (J10-2), (2-17)	0	$M_n = 0.5(B - t) R_n$
	$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)		
	Limit State 5: Local crippling of the HSS sidewalls		
	$R_n = 1.6t^2 \left(1 + \frac{3t_p}{H} \right) \sqrt{EF_y} Q_f$ (J10-4), (2-19)	0	—
	$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)		
	Limit State 6: Buckling of the HSS sidewalls		
	$R_n = \left(\frac{48t^3}{H - 3t} \right) \sqrt{EF_y} Q_f$ (J10-8), (2-23)	0	—
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)		

(Table 8-2 continues on the next page)

Table 8-2. Available Strengths of Plate-to-Rectangular HSS Connections (continued)

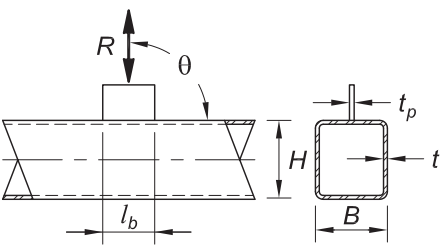
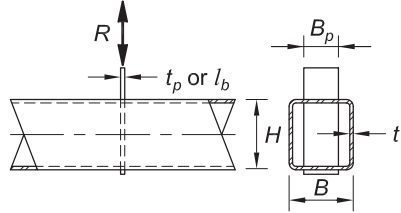
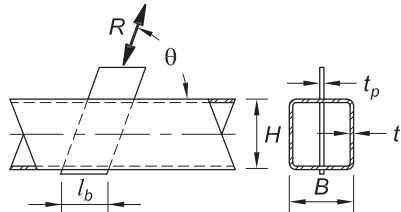
Connection Type	Connection Available Strength ¹	Plate Bending ²	
		In-Plane	Out-of-Plane
Longitudinal plate T-, Y-, and cross-connection  where $\beta = t_p/B$	Plate Axial Load Limit State 7: HSS plastification $R_n \sin \theta = F_y t^2 \left[\frac{2l_b/B}{1-\beta} + \frac{4}{\sqrt{1-\beta}} \right] Q_f \quad (2-3)$	$M_n = F_y t^2 l_b Z Q_f$ $Z = \frac{B}{2l_b} + \frac{2}{\sqrt{1-\beta}} + \frac{l_b/B}{1-\beta} \quad (2-6)$	0
	$\phi = 1.00$ (LRFD)	$\Omega = 1.50$ (ASD)	
	Plate Axial Load	In-Plane	Out-of-Plane
	Limit State 8: HSS plastification		
Transverse through-plate connections 	$R_n = 2R_n$ from Limit State 1	0	—
	$\phi = 1.00$ (LRFD)	$\Omega = 1.50$ (ASD)	
	Limit State 9: HSS shear yielding (punching shear)		
	$R_n = 2R_n$ from Limit State 2	0	—
Longitudinal through-plate T- and Y-connections 	$\phi = 1.00$ (LRFD)	$\Omega = 1.50$ (ASD)	
	Plate Axial Load	In-Plane	Out-of-Plane
	Limit State 10: HSS plastification		
	$R_n = 2R_n$ from Limit State 7	—	0
Functions			
$Q_f = 1.0$ for HSS connecting surface(s) in tension $= 0.4 \leq 1.3 - 0.4 \left(\frac{U}{\beta} \right) \leq 1.0$ for HSS connecting surface(s) in compression, except for connections involving longitudinal plates (K1-4) $= 1 - 0.3U(1+U) \leq 1.0$ for HSS connecting surface(s) in compression, for longitudinal plate connections (K1-3) $U = \left \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right \leq 1.0$ (K1-6) where P_{ro} and M_{ro} are determined on the side of the connection that has the higher compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD and P_a for ASD; $M_{ro} = M_u$ for LRFD and M_a for ASD.			
¹ Equation references are to the AISC <i>Specification</i> or in this Design Guide ² Where no formula is given, "0" represents zero available strength and "—" indicates that information is not available.			

Table 8-2A. Limits of Applicability for Limit States in Table 8-2

Limit State 1: $\beta = B_b/B < 1.0$; plus restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical Q_f), and end distance apply.	
Limit State 2: $B_b \leq (B - 2t)$, plus restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical B_{ep}), and end distance apply.	
Limit State 3: Restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical B_e), and end distance apply.	
Limit State 4: $\beta \approx 1.0$; plus restriction on end distance.	
Limit State 5: Applicable for T-connections, plate is loaded in compression, and $\beta \approx 1.0$. Also, restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical Q_f), and end distance apply.	
Limit State 6: Applicable for cross-connections, plates are loaded in compression, and $\beta \approx 1.0$. Also, restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical Q_f), and end distance apply.	
Limit State 7: Restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical Q_f), and end distance apply.	
Limit State 8: $B_b \leq (B - 2t)$; plus restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical Q_f), and end distance apply.	
Limit State 9: $B_b \leq (B - 2t)$; plus restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical B_{ep}), and end distance apply.	
Limit State 10: Restrictions on HSS wall slenderness, width ratio, material strength and ductility (because of empirical Q_f), and end distance apply.	
HSS wall slenderness:	B/t or $H/t \leq 35$ for loaded wall, for transverse branch and through-plate connections B/t or $H/t \leq 40$ for loaded wall, for longitudinal branch and through plate connections
Width ratio:	$0.25 < B_b/B \leq 1.0$ for transverse branch plate connections $0.25 < B_b/B \leq 1 - 2t/B$ for transverse through plate connections
Material strength:	$F_y \leq 52$ ksi
Ductility:	$F_y/F_u \leq 0.8$ Note: ASTM A500/A500M Grade C is acceptable
End distance:	$l_{end} \geq B\sqrt{1-\beta}$ for connections under axial load with $\beta \leq 0.85$ (Spec. Eq. K1-7)

open end of the member. An all-around fillet weld is usually used to provide a seal for the HSS. The plate thickness and weld size can be of minimum practical size if the cap plate only serves as a seal from the elements.

A cap plate is also used in situations where a plate or branch of an HSS connection is a distance less than l_{end} (Tables 8-1A and 8-2A) from an open chord end (as also discussed in Section 8.1). In such cases, a minimum cap plate thickness of $1.5t$ has been suggested in an amendment to EN 1993-1-8 (CEN, 2005) so that the cap plate has sufficient thickness relative to the chord side walls, to restrain local deformation of the HSS chord and to develop the required connection strength. A single-pass fillet weld for such a cap plate is usually sufficient.

8.4.2 Axial Compression or Tension on a Cap Plate from a Tee-Stub

The design method for cap plate connections to the ends of HSS members, where the axial force in a plate is transferred to the HSS via a cap plate (or via the flange of a tee-stub), is presented in Section 7.1.

8.4.3 Compression on a Cap Plate from a Beam, Girder, or Joist

In single-story frames, a simple connection solution to an HSS column is to place the beam (or the top chord member of a joist) over the top of the HSS column, with a cap plate (Fisher, 2019). The connection detail shown in Figure 8-9 will subject the cap plate to bending and will transfer moment into the HSS column through a force couple that consists of both compressive and tensile reactions from the cap plate. The nominal strength of the cap plate to a rectangular HSS, expressed as a reaction, is (AISC, 1997b, 2011):

$$R_n = \frac{Bt_p^2}{4\left(\frac{l_b}{2} + a - \frac{H}{2}\right)} F_{yp} \quad (8-5)$$

where $\phi = 0.90$ (LRFD) and $\Omega = 1.67$ (ASD), t_p and F_{yp} are the thickness and yield strength of the cap plate, a is the distance from the HSS centroid to the end of the attached member, H is the HSS depth oriented as shown in Figure 8-9, B is the HSS width normal to the plane of Figure 8-9, and l_b is the bearing length for the attached member.

Equation 8-5 only applies if the cap plate is subjected to cantilever bending, as shown in Figure 8-9. This occurs when the beam or joist reaction point is outside of the HSS face. If a bearing stiffener is used on the beam or girder web, and is positioned over the HSS wall, then Equation 8-5 does not apply because the cap plate is not subjected to bending. Also, if the denominator of Equation 8-5 results in a negative number, bending of the cap plate can be disregarded (AISC, 1997b, 2011); that is, bending will not occur unless $l_b > (H - 2a)$. When selecting the bearing length, note that it must also be adequate to satisfy the bearing limit state checks applied to the beam or girder.

With a bearing reaction onto a cap plate as shown in Figure 8-9, the two other limit states in Section 7.1 still need to be checked for the compressed HSS wall. For local yielding of one rectangular HSS wall in compression:

$$R_n = F_y t (5t_p + l_b) \leq F_y t B \quad (8-6)$$

where $\phi = 1.00$ (LRFD), $\Omega = 1.50$ (ASD), and l_b in this case is the bearing length of the load on top of the cap plate, across the width of the HSS (normal to the plane of Figure 8-9 or parallel to B). This can be taken as twice the k distance for wide-flange shapes without stiffeners, or the width of the joist or joist girder seat (AISC, 1997b).

For local crippling of one rectangular HSS wall in compression, the available strength is given by:

$$R_n = 0.80 t^2 \left[1 + 3 \left(\frac{l_b}{B/2} \right) \left(\frac{t}{t_p} \right)^{1.5} \right] \sqrt{\frac{E F_y t_p}{t}} \quad (8-7)$$

where $\phi = 0.75$ (LRFD), $\Omega = 2.00$ (ASD), and the bearing length, l_b , is the same as used for local yielding of the HSS wall. Thus, the available strength for the three limit states associated with Equations 8-5, 8-6, and 8-7 should exceed the required strength from the beam shear reaction.

8.5 LINE LOAD CONNECTION DESIGN EXAMPLES

EXAMPLE 8.1—Longitudinal Branch-Plate/Through-Plate Connection with Rectangular HSS

Given:

The welded connection shown in Figure 8-10 is subject to the plate axial tension force indicated, produced by a diagonal bracing member. The column is subjected to axial compression dead and live loads of $P_D = 20.0$ kips and $P_L = 60.0$ kips. Determine if the connection is adequate as a branch-plate connection or if a through-plate connection is required (as shown in Figure 8-10). Note that the connection is checked under the action of the plate force component normal to the column because this has the dominant effect.

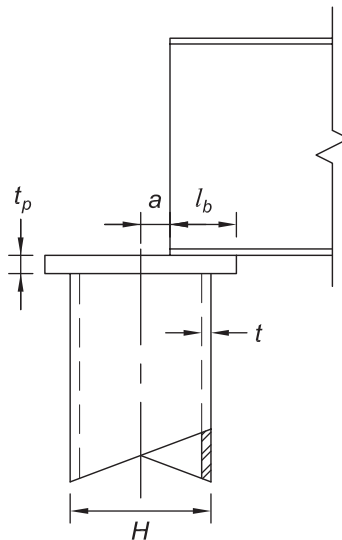


Fig. 8-9. Cap plate subjected to cantilever bending.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column

ASTM A1085/A1085M

$F_y = 50$ ksi

$F_u = 65$ ksi

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50$ ksi

$F_{up} = 65$ ksi

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the HSS geometric properties are as follows:

HSS8×8×¼

$B = 8.00$ in.

$H = 8.00$ in.

$t = 0.250$ in.

$A = 7.59$ in.²

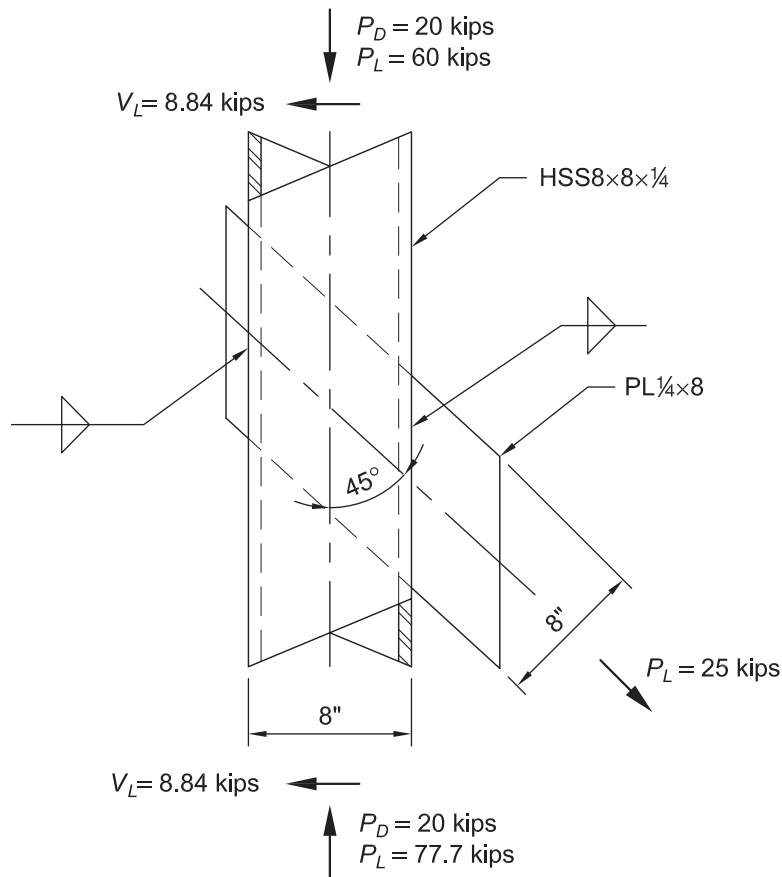


Fig. 8-10. Through-plate connection with rectangular HSS for Example 8.1.

Solution:*Limits of applicability*

From Table 8-2, it is seen that Limit States 7 and 10 apply to branch-plate and through-plate connections, respectively. The limits of applicability for these Limit States are defined in Table 8-2A.

HSS wall slenderness:

$$\begin{aligned} B/t &= H/t \\ &= 8.00 \text{ in.} / 0.250 \text{ in.} \\ &= 32.0 \leq 40 \quad \mathbf{o.k.} \end{aligned}$$

Material strength:

$$F_y = 50 \text{ ksi} \leq 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$\begin{aligned} F_y/F_u &= 50 \text{ ksi} / 65 \text{ ksi} \\ &= 0.769 \leq 0.8 \quad \mathbf{o.k.} \end{aligned}$$

End distance:

$$\begin{aligned} \beta &= t/B \\ &= 0.250 \text{ in.} / 8.00 \text{ in.} \\ &= 0.0313 \leq 0.85 \quad \mathbf{o.k.} \end{aligned}$$

$$\begin{aligned} l_{end} &\geq B\sqrt{1-\beta} && (\text{Spec. Eq. K1-7}) \\ &\geq (8.00 \text{ in.})\sqrt{1-0.0313} \\ &\geq 7.87 \text{ in.} \end{aligned}$$

Thus, each edge of the plate must be located a minimum of 7.87 in. from an uncapped HSS end.

Required connection strength (expressed as a force in the plate)

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.6P_L$ $= 1.6(25.0 \text{ kips})$ $= 40.0 \text{ kips}$	$R_a = P_L$ $= 25.0 \text{ kips}$

HSS plastification limit state, assuming a branch plate connection

From Table 8-2, Limit State 7 gives the nominal connection strength under plate axial load as:

$$R_n \sin \theta = F_y t^2 \left[\frac{2l_b/B}{1-\beta} + \frac{4}{\sqrt{1-\beta}} \right] Q_f \quad (2-3)$$

For a rectangular HSS column in compression with a longitudinal plate connection:

$$Q_f = 1 - 0.3U(1 + U) \leq 1.0 \quad (\text{Spec. Eq. K1-3})$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{Spec. Eq. K1-6})$$

From ASCE/SEI 7, Chapter 2, the required HSS column strength, P_{ro} , is determined as follows (noting that $M_{ro} = 0$):

LRFD	ASD
In column on the side of the connection with higher compression stress: $P_{ro} = P_u$ $= 1.2P_D + 1.6P_L$ $= 1.2(20 \text{ kips}) + 1.6(77.7 \text{ kips})$ $= 148 \text{ kips}$ $F_c = F_y$ $= 50 \text{ ksi}$ $U = \left \frac{148 \text{ kips}}{(50 \text{ ksi})(7.59 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.390$ $Q_f = 1 - 0.3(0.390)(1 + 0.390) \leq 1.0$ $= 0.837$	In column on the side of the connection with higher compression stress: $P_{ro} = P_a$ $= P_D + P_L$ $= 20 \text{ kips} + 77.7 \text{ kips}$ $= 97.7 \text{ kips}$ $F_c = 0.6F_y$ $= 0.6(50 \text{ ksi})$ $= 30.0 \text{ ksi}$ $U = \left \frac{97.7 \text{ kips}}{(30.0 \text{ ksi})(7.59 \text{ in.}^2)} \right \leq 1.0$ $= 0.429$ $Q_f = 1 - 0.3(0.429)(1 + 0.429) \leq 1.0$ $= 0.816$

The available strength for the HSS plastification limit state, as a branch plate connection, is:

LRFD	ASD
$R_n = (50 \text{ ksi})(0.250 \text{ in.})^2$ $\times \left\{ \left[\frac{2(8 \text{ in.}/\sin 45^\circ)}{8 \text{ in.}} \right] + \frac{4}{\sqrt{1 - 0.0313}} \right\}$ $\times (0.837)/\sin 45^\circ$ $= 25.8 \text{ kips}$ $\phi R_n = 1.00(25.8 \text{ kips})$ $= 25.8 \text{ kips} < 40.0 \text{ kips} \quad \mathbf{n.g.}$	$R_n = (50 \text{ ksi})(0.250 \text{ in.})^2$ $\times \left\{ \left[\frac{2(8 \text{ in.}/\sin 45^\circ)}{8 \text{ in.}} \right] + \frac{4}{\sqrt{1 - 0.0313}} \right\}$ $\times (0.816)/\sin 45^\circ$ $= 25.2 \text{ kips}$ $\frac{R_n}{\Omega} = \frac{25.2 \text{ kips}}{1.50}$ $= 16.8 \text{ kips} < 25.0 \text{ kips} \quad \mathbf{n.g.}$

By treating the connection as a branch-plate connection, the available strength for the limit state of HSS plastification is not adequate. Check this limit state by treating the connection as a through-plate connection.

HSS plastification limit state, assuming a through-plate connection

From Table 8-2, Limit State 10 gives the nominal connection strength under plate axial load as:

$$R_n \sin \theta = 2F_y t^2 \left[\frac{2l_b/B}{1-\beta} + \frac{4}{\sqrt{1-\beta}} \right] Q_f$$

This gives a nominal connection strength that is double that of a branch-plate connection (from Equation 2-3 or Limit State 7); hence:

LRFD	ASD
$\phi R_n = 2(25.8 \text{ kips})$ $= 51.6 \text{ kips} > 40.0 \text{ kips} \quad \text{o.k.}$	$\frac{R_n}{\Omega} = 2(16.8 \text{ kips})$ $= 33.6 \text{ kips} > 25.0 \text{ kips} \quad \text{o.k.}$

The through-plate connection is adequate.

Tensile yielding in the gross section of the plate

From AISC *Specification* Section D2, the nominal strength for the limit state of tensile yielding of the plate is:

$$\begin{aligned} A_g &= b_p t_p \\ &= (8 \text{ in.})(\frac{1}{4} \text{ in.}) \\ &= 2.00 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} P_n &= F_y A_g \\ &= (50 \text{ ksi})(2.00 \text{ in.}^2) \\ &= 100 \text{ kips} \end{aligned} \quad (\text{Spec. Eq. D2-1})$$

The available strength is:

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(100 \text{ kips})$ $= 90.0 \text{ kips} > 40.0 \text{ kips} \quad \text{o.k.}$	$\Omega_t = 1.67$ $\frac{P_n}{\Omega_t} = \frac{100 \text{ kips}}{1.67}$ $= 59.9 \text{ kips} > 25.0 \text{ kips} \quad \text{o.k.}$

The through plate could be fillet-welded to both sides of the HSS along all longitudinal (11.3 in.) sides of the plate. The full length of the welds would be effective, and because double-sided welds are used on the plate, the fillet weld directional strength increase factor could be used (with $\theta = 45^\circ$ for the longitudinal welds).

EXAMPLE 8.2—Wide-Flange Beam-to-Round HSS Column Moment Connection

Given:

An ASTM A992/A992M W18x35 cantilever beam is profiled at its end and directly welded to an ASTM A500/A500M Grade C HSS8.625x0.500 column to support a small canopy. Determine the adequacy of the PR connection under the loads indicated in Figure 8-11 with respect to its moment capacity. Note that the beam will be treated as a pair of transverse plates. The critical plate connection is the lower beam flange because the column connecting surface adjoining that plate develops compressive stresses due to both axial force and bending. These compressive stresses cumulatively affect the U factor. The moment capacity of the entire beam-to-column connection is then based on the capacity of the bottom (compression) flange connection multiplied by the distance between the beam flange centers.

From AISC *Manual* Table 2-4, the material properties are as follows:

Column

ASTM A500/A500M Grade C

$$F_v = 50 \text{ ksi}$$
$$F_u = 62 \text{ ksi}$$

Beam

ASTM A992/A992M

$$F_{yp} = 50 \text{ ksi}$$
$$F_{up} = 65 \text{ ksi}$$

From AISC *Manual* Tables 1-1 and 1-13, the geometric properties are as follows:

HSS8.625×0.500

$$D = 8.625 \text{ in.}$$
$$t = 0.465 \text{ in.}$$
$$A = 11.9 \text{ in.}^2$$
$$S = 23.1 \text{ in.}^3$$

W18×35

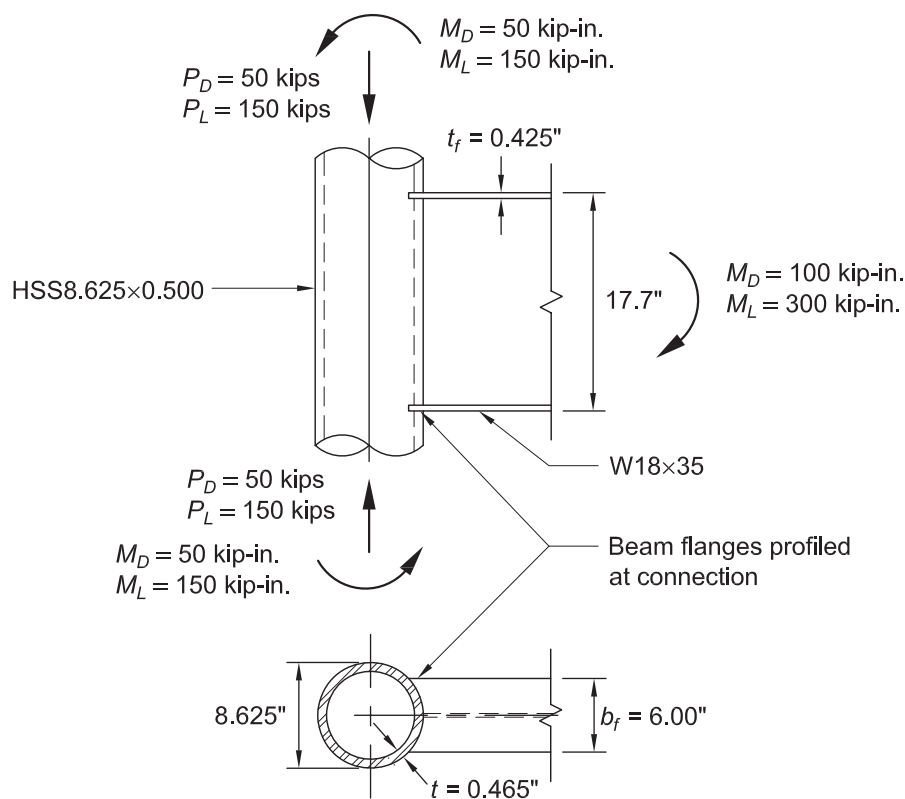
$$d = 17.7 \text{ in.}$$
$$b_f = 6.00 \text{ in.}$$
$$t_f = 0.425 \text{ in.}$$


Fig. 8-11. Beam-to-round HSS moment connection for Example 8.2.

Solution:*Limits of applicability*

From AISC *Specification* Table K2.1A and Table 8-1A, the limits of applicability for a transverse plate connection are:

HSS wall slenderness:

$$\begin{aligned} D/t &= 8.625 \text{ in.}/0.465 \text{ in.} \leq 50 \\ &= 18.5 < 50 \quad \mathbf{o.k.} \end{aligned}$$

Width ratio:

$$\begin{aligned} B_b/D &= b_f/D \\ &= 6.00 \text{ in.}/8.625 \text{ in.} \\ &= 0.696 \\ 0.2 &< 0.696 < 1.0 \quad \mathbf{o.k.} \end{aligned}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$\begin{aligned} F_y/F_u &= 50 \text{ ksi}/62 \text{ ksi} \\ &= 0.806 > 0.8 \end{aligned}$$

However, the note in Table 8-1A indicates ASTM A500/A500M Grade C is acceptable. **o.k.**

End distance:

$$\begin{aligned} l_{end} &\geq D \left(1.25 - \frac{B_b/D}{2} \right) \\ &\geq (8.625 \text{ in.}) \left[1.25 - \frac{6.00 \text{ in.}/8.625 \text{ in.}}{2} \right] \\ &\geq 7.78 \text{ in.} \end{aligned}$$

Thus, the surfaces of both beam flanges must be located a minimum of 7.78 in. from an uncapped HSS end.

Required flexural strength (expressed as a moment in the beam)

From ASCE/SEI 7, Chapter 2, the required flexural strength is:

LRFD	ASD
$\begin{aligned} M_u &= 1.2M_D + 1.6M_L \\ &= 1.2(100 \text{ kip-in.}) + 1.6(300 \text{ kip-in.}) \\ &= 600 \text{ kip-in.} \end{aligned}$	$\begin{aligned} M_a &= M_D + M_L \\ &= 100 \text{ kip-in.} + 300 \text{ kip-in.} \\ &= 400 \text{ kip-in.} \end{aligned}$

HSS plastification, as a transverse plate connection

From AISC Specification Section K2.2 and Table 8-1, the nominal strength for the limit state of HSS plastification is:

$$R_n = F_y t^2 \left[\frac{5.5}{1 - 0.81 \left(\frac{B_b}{D} \right)} \right] Q_f \quad (\text{from Spec. Eq. K2-1a})$$

For a round HSS column surface in compression:

$$Q_f = 1.0 - 0.3U(1 + U) \leq 1.0 \quad (\text{Spec. Eq. K1-3})$$

The chord utilization ratio is:

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{Spec. Eq. K1-6})$$

Both sides of the bottom flange connection have approximately the same compression stress in the HSS connecting surface, due to axial force plus bending moment.

From ASCE/SEI 7, Chapter 2, the required strengths, P_{ro} and M_{ro} , in the HSS are determined as follows:

LRFD	ASD
$P_{ro} = P_u$ $= 1.2P_D + 1.6P_L$ $= 1.2(50 \text{ kips}) + 1.6(150 \text{ kips})$ $= 300 \text{ kips}$ $M_{ro} = M_u$ $= 1.2M_D + 1.6M_L$ $= 1.2(50 \text{ kip-in.}) + 1.6(150 \text{ kip-in.})$ $= 300 \text{ kip-in.}$ $F_c = F_y$ $= 50 \text{ ksi}$ $U = \left \frac{300 \text{ kips}}{(50 \text{ ksi})(11.9 \text{ in.}^2)} + \frac{300 \text{ kip-in.}}{(50 \text{ ksi})(23.1 \text{ in.}^3)} \right $ $= 0.764 \leq 1.0$ $Q_f = 1.0 - 0.3(0.764)(1 + 0.764) \leq 1.0$ $= 0.596$	$P_{ro} = P_a$ $= P_D + P_L$ $= 50 \text{ kips} + 150 \text{ kips}$ $= 200 \text{ kips}$ $M_{ro} = M_a$ $= M_D + M_L$ $= 50 \text{ kip-in.} + 150 \text{ kip-in.}$ $= 200 \text{ kip-in.}$ $F_c = 0.6F_y$ $= 0.6(50 \text{ ksi})$ $= 30.0 \text{ ksi}$ $U = \left \frac{200 \text{ kips}}{(30.0 \text{ ksi})(11.9 \text{ in.}^2)} + \frac{200 \text{ kip-in.}}{(30.0 \text{ ksi})(23.1 \text{ in.}^3)} \right $ $= 0.849 \leq 1.0$ $Q_f = 1.0 - 0.3(0.849)(1 + 0.849) \leq 1.0$ $= 0.529$

The available axial and flexural strengths for the limit state of HSS local yielding (or plastification) are determined as follows:

LRFD	ASD
$\phi = 0.90$ $R_n = (50 \text{ ksi})(0.465 \text{ in.})^2$ $\times \left[\frac{5.5}{1 - 0.81 \left(\frac{6.00 \text{ in.}}{8.625 \text{ in.}} \right)} \right] (0.596)$ $= 81.2 \text{ kips}$ $\phi R_n = 0.90(81.2 \text{ kips})$ $= 73.1 \text{ kips}$ $\phi M_n = \phi R_n (d - t_f)$ $= (73.1 \text{ kips})(17.7 \text{ in.} - 0.425 \text{ in.})$ $= 1,260 \text{ kip-in.} > 600 \text{ kip-in.} \quad \mathbf{o.k.}$	$\Omega = 1.67$ $R_n = (50 \text{ ksi})(0.465 \text{ in.})^2$ $\times \left[\frac{5.5}{1 - 0.81 \left(\frac{6.00 \text{ in.}}{8.625 \text{ in.}} \right)} \right] (0.529)$ $= 72.1 \text{ kips}$ $\frac{R_n}{\Omega} = \frac{72.1 \text{ kips}}{1.67}$ $= 43.2 \text{ kips}$ $\frac{M_n}{\Omega} = \frac{R_n}{\Omega} (d - t_f)$ $= (43.2 \text{ kips})(17.7 \text{ in.} - 0.425 \text{ in.})$ $= 746 \text{ kip-in.} > 400 \text{ kip-in.} \quad \mathbf{o.k.}$

HSS shear yielding (or punching shear), as a transverse plate connection

From Table 8-1, Limit State 2 provides the nominal strength for HSS shear yielding, with $B_b \leq D - 2t$, as follows:

$$\begin{aligned}
 R_n &= 0.6F_y t (2t_p + 2B_b) \\
 &= 0.6(50 \text{ ksi})(0.465 \text{ in.}) [2(0.425 \text{ in.}) + 2(6.00 \text{ in.})] \\
 &= 179 \text{ kips}
 \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(179 \text{ kips})$ $= 179 \text{ kips}$ $\phi M_n = \phi R_n (d - t_f)$ $= (179 \text{ kips})(17.7 \text{ in.} - 0.425 \text{ in.})$ $= 3,090 \text{ kip-in.} > 600 \text{ kip-in.} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{179 \text{ kips}}{1.50}$ $= 119 \text{ kips}$ $\frac{M_n}{\Omega} = \frac{R_n}{\Omega} (d - t_f)$ $= (119 \text{ kips})(17.7 \text{ in.} - 0.425 \text{ in.})$ $= 2,060 \text{ kip-in.} > 400 \text{ kip-in.} \quad \mathbf{o.k.}$

EXAMPLE 8.3—Transverse Branch-Plate Connection with Rectangular HSS

Given:

Determine if the hanger connection with a transverse plate, shown in Figure 8-12, is adequate for the required forces shown and if it can be fillet-welded. Use an ASTM A1085/A1085M HSS6×6×¼ member with ASTM A572/A572M Grade 50 plate. The “workable flat” of a rectangular HSS is based on an outside corner radius of 2.25 times the HSS nominal thickness (AISC *Manual* Part 1); hence for an HSS6×6×¼ chord member, the workable flat is 4⅞ in. This is also shown in Table 1-12 of the AISC *Manual*. Thus, a 4 in. plate adequately sits on the HSS flat and allows fillet welding. See Example 3.3 in Chapter 3 for a design example of a transverse fillet weld to a rectangular HSS.

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

HSS

ASTM A1085/A1085M

$F_y = 50$ ksi

$F_u = 65$ ksi

Plate

ASTM A572/A572M Grade 50

$F_{yp} = 50$ ksi

$F_{up} = 65$ ksi

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the HSS geometric properties are as follows:

HSS6×6×¼

$B = 6.00$ in.

$H = 6.00$ in.

$t = 0.250$ in.

$A = 5.59$ in.²

Solution:

Limits of applicability

From Table 8-2, there are six potential limit states for transverse plate T-connections. Referring to Table 8-2A:

$$\beta = B_b/B$$

$$= 4 \text{ in.}/6.00 \text{ in.}$$

$$= 0.667 < 1.0$$

$$B_b \leq (B - 2t)$$

$$B - 2t = 6.00 \text{ in.} - 2(0.250 \text{ in.})$$

$$= 5.50 \text{ in.}$$

$$B_b = 4 \text{ in.} \leq 5.50 \text{ in.}$$

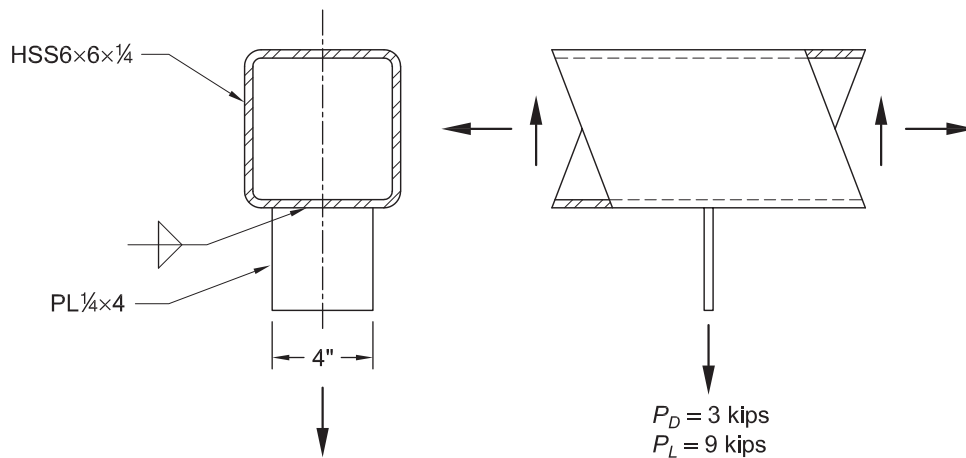


Fig. 8-12. Transverse plate connection to rectangular HSS for Example 8.3.

Therefore, Limit States 1, 2, and 3 apply.

HSS wall slenderness:

$$\begin{aligned} B/t &= H/t \\ &= 6.00 \text{ in.}/0.250 \text{ in.} \leq 35 \\ &= 24.0 < 35 \quad \text{o.k.} \end{aligned}$$

Width ratio:

$$\begin{aligned} 0.25 < \beta &\leq 1.0 \\ 0.25 < 0.667 &< 1.0 \quad \text{o.k.} \end{aligned}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \text{o.k.}$$

Ductility:

$$\begin{aligned} F_y/F_u &= 50 \text{ ksi}/65 \text{ ksi} \\ &= 0.769 < 0.8 \quad \text{o.k.} \end{aligned}$$

End distance:

$$\begin{aligned} \beta &= 0.667 < 0.85 \quad \text{o.k.} \\ l_{end} &\geq B\sqrt{1-\beta} \\ &\geq (6.00 \text{ in.})\sqrt{1-0.667} \\ &\geq 3.46 \text{ in.} \end{aligned}$$

Thus, each edge of the plate must be located a minimum of 3.46 in. from an uncapped HSS end.

Required connection strength (expressed as a force in the plate)

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$\begin{aligned} R_u &= 1.2P_D + 1.6P_L \\ &= 1.2(3 \text{ kips}) + 1.6(9 \text{ kips}) \\ &= 18.0 \text{ kips} \end{aligned}$	$\begin{aligned} R_a &= P_D + P_L \\ &= 3 \text{ kips} + 9 \text{ kips} \\ &= 12.0 \text{ kips} \end{aligned}$

HSS plastification

From Table 8-2, Limit State 1 gives the nominal connection strength under plate axial load. From AISC *Specification* Section K1.3(a), for an HSS connected surface in tension, $Q_f = 1.0$.

$$\begin{aligned} R_n &= F_y t^2 \left[\frac{2t_p/B}{1-\beta} + \frac{4}{\sqrt{1-\beta}} \right] Q_f \quad (2-2) \\ &= (50 \text{ ksi})(0.250 \text{ in.})^2 \left[\frac{2(1/4 \text{ in.})/6.00 \text{ in.}}{1-0.667} + \frac{4}{\sqrt{1-0.667}} \right] (1.0) \\ &= 22.4 \text{ kips} \end{aligned}$$

The available strength for the HSS plastification limit state is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(22.4 \text{ kips})$ $= 22.4 \text{ kips} > 18.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{22.4 \text{ kips}}{1.50}$ $= 14.9 \text{ kips} > 12.0 \text{ kips} \quad \mathbf{o.k.}$

HSS shear yielding (punching shear)

From Table 8-2, Limit State 2 gives the nominal connection strength under plate axial load.

$$R_n = 0.6F_y t(2t_p + 2B_{ep}) \quad (8-3a)$$

where

$$B_{ep} = \left(\frac{10t}{B} \right) B_b \leq B_b \quad (\text{Spec. Eq. K1-2})$$

$$= \left[\frac{10(0.250 \text{ in.})}{6.00 \text{ in.}} \right] (4 \text{ in.}) \leq 4 \text{ in.}$$

$$= 1.67 \text{ in.} < 4 \text{ in.}$$

$$= 1.67 \text{ in.}$$

$$R_n = 0.6(50 \text{ ksi})(0.250 \text{ in.})[2(\frac{1}{4} \text{ in.}) + 2(1.67 \text{ in.})]$$

$$= 28.8 \text{ kips}$$

The available strength for the HSS shear yielding limit state is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(28.8 \text{ kips})$ $= 28.8 \text{ kips} > 18.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{28.8 \text{ kips}}{1.50}$ $= 19.2 \text{ kips} > 12.0 \text{ kips} \quad \mathbf{o.k.}$

Local yielding of the branch plate

From Table 8-2, Limit State 3 gives the nominal connection strength under plate axial load.

$$R_n = \left(\frac{10t}{B} \right) F_y t B_b \leq F_{yp} t_p B_b \quad (8-2a)$$

$$= \left[\frac{10(0.250 \text{ in.})}{6.00 \text{ in.}} \right] (50 \text{ ksi})(0.250 \text{ in.})(4 \text{ in.}) \leq (50 \text{ ksi})(\frac{1}{4} \text{ in.})(4.00 \text{ in.})$$

$$= 20.8 \text{ kips} \leq 50.0 \text{ kips}$$

$$= 20.8 \text{ kips}$$

The available strength for the limit state of local yielding of the branch plate is:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(20.8 \text{ kips})$ $= 20.8 \text{ kips} > 18.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{20.8 \text{ kips}}{1.50}$ $= 13.9 \text{ kips} > 12.0 \text{ kips} \quad \mathbf{o.k.}$

Thus, local yielding of the branch plate governs, and all relevant limit states are satisfied.

Chapter 9

HSS-to-HSS Truss Connections

9.1 SCOPE AND BASIS

This chapter covers—and goes beyond—the requirements of AISC *Specification* Section K3 and, by implication, the requirements of Chapter J, for truss-type connections between HSS (or box-section) members directly welded to one another, in the form of T-, Y-, cross-, K- (or N-), and KT-connections. Round HSS web members to either round or rectangular HSS chord members, as well as rectangular HSS web members to rectangular chord members, are considered. Pertinent design criteria for planar round HSS-to-HSS truss connections are succinctly tabulated in Table 9-1, Section 9.7, followed by two design examples on such sections (Section 9.9). Similarly, pertinent design criteria for planar rectangular (which includes square) HSS-to-HSS truss connections are tabulated in Table 9-2, Section 9.7, followed by four design examples, one of which is for a KT-connection (with three branch members).

A considerable number of multi-planar welded connections are covered in Section 9.8. For connection configurations beyond the scope of this chapter, such as connections with partially or fully flattened branch member ends, double chord connections, or even HSS branch members joined to a wide-flange chord member, one can refer to other authoritative design guidance. Such publications include the CISC Design Guide (Packer and Henderson, 1997), CIDECT Design Guide No. 1 (Wardenier et al., 2008a), CIDECT Design Guide No. 3 (Packer et al., 2009), the CIDECT book (Wardenier et al., 2010), IIW (2012), and ISO (2013). For rectangular HSS connections in which a branch member is laterally offset from the centerline of the chord, such that one side of the branch aligns with the chord side wall, design guidance is available in Bu et al. (2021).

The design criteria in this chapter are based on failure modes, or limit states, that have been reported in international research on HSS, as discussed in Chapter 2. Much of this has been sponsored and synthesized by CIDECT since the 1960s. This work has also received critical review by the IIW Subcommittee XV-E on “Tubular Structures.” The HSS connection design provisions herein are generally in accord with the second edition of the design recommendations by this Subcommittee (IIW, 1989). Some minor modifications to these IIW recommendations—for some limit states—have been made by the adoption of the formulas for the same limit states elsewhere in the AISC *Specification*. The CISC Design Guide (Packer and Henderson, 1997) and EN 1993-1-8 (CEN, 2005) are also based on the second edition of the IIW recommendations. It should be noted that

ISO (2013) and the latest CIDECT guidance (Wardenier et al., 2008a; Packer et al., 2009; Wardenier et al., 2010) are based on the third edition of the IIW recommendations (IIW, 2012). Among other refinements, these latest IIW recommendations include a connection deformation limit state, as discussed in AISC *Specification* Chapter K Commentary and also Section 2.2 of this Design Guide. A large amount of connection research data generated by CIDECT research programs up to the early 1980s is summarized in CIDECT Monograph No. 6 (Giddings and Wardenier, 1986) and by Wardenier (1982). Further information on CIDECT publications and reports can be obtained from its website www.cidect.org.

Some of the foregoing international guides, as well as the first edition of this Design Guide (Packer et al., 2010b) and the 2010 AISC *Specification*, required HSS-to-HSS welded connections to have a minimum branch-to-chord member angle of 30°. This was included to ensure that nearly all fabricators could form proper welds. However, as noted in Section 3.3, this is not an absolute restriction. If designing truss-type connections with branches at angles less than 30°, confirmation of weldability should first be sought with the eventual fabricator—especially for rectangular-to-rectangular HSS connections.

The design recommendations in this chapter presume that there is continuity of the chord member beyond the connection for a reasonable distance, and thus the connection capacity is not affected by the termination of the chord member. As discussed in Chapter 2, there is a minimum chord “end distance,” l_{end} , beyond a branch member to an open chord end to avoid any reduction in connection strength. For an axially loaded branch to a rectangular HSS chord, with $\beta \leq 0.85$, this is given by:

$$l_{end} \geq B\sqrt{1-\beta} \quad (\text{Spec. Eq. K1-7})$$

For axially loaded branches to round HSS chords:

$$l_{end} \geq D\left(1.25 - \frac{\beta}{2}\right) \quad (\text{Spec. Eq. K1-8})$$

As noted in Chapter 8, the l_{end} inequality for rectangular HSS chords is only valid over a β range for which flexural failure of the chord face governs (i.e., $\beta < \text{approximately } 0.85$). Further discussion is given in Chapter 2 for other limit states.

In cases where the HSS branch is a distance less than the limiting l_{end} from an open chord end, and where load

dispersion is not available equally to both sides of the connection, a cap plate welded to the chord end can be used to restore the full connection strength. Alternatively, a reduced connection strength (conservatively, 50%) may be used, unless a specific connection capacity based on end distance is available (e.g., Equation 2-4). Near an open end, the branch member must also have sufficient lateral and rotational restraint.

As indicated by its title, AISC *Specification* Chapter K is also applicable to welded connections made from box sections (i.e., rectangular sections fabricated from plates). To guarantee that a welded box-section connection behaves in a manner similar to a connection formed between closed HSS members, under any loading, box-section members require CJP groove welds in their four corners, in the connection region. The latter can be interpreted as a distance equal to the member width (B or B_b) from the intersecting member (AISC *Specification* Chapter K Commentary).

9.2 NOTATION, TERMINOLOGY, AND LIMIT STATES

Some common notation associated with welded HSS connections that is used in this chapter is shown in Figure 9-1 for a K-connection. An N-connection (as found in a Pratt truss) can be considered a special case, with one branch at 90° , of the general K-connection (as found in a Warren truss). The T-connection, with one branch at 90° , can similarly be considered as a special case of the general Y-connection.

9.2.1 Gap, Overlap, and Eccentricity in K-Connections

For a gapped K-connection, the gap, g , is the distance between the toes of the branch members, measured along the chord connecting surface, neglecting the welds. For an overlapped K-connection, the amount of overlap, O_v , is defined as $(l_{ov}/l_p) \times 100\%$, where l_{ov} is the overlap length measured along the chord connecting surface beneath the two branches, and l_p is the projected length (or imaginary footprint) of the overlapping branch onto the chord surface (see Figure 9-1).

For K-connections, both AISC *Specification* Table K3.1A (for round-to-round HSS) and Table K3.2A (for rectangular-to-rectangular HSS) have minimum values for the amount of gap, g (to facilitate welding around each branch), and the amount of overlap, O_v (to ensure proper load transfer between the overlapping branches). Although beyond the limits of applicability of AISC *Specification* Table K3.2, recent research has shown that very small gap or even zero-gap K-connections are possible between rectangular HSS, with a predictable capacity, but the welding requirements are very stringent (Bu and Packer, 2022).

Another important K-connection parameter related to the gap or overlap is the eccentricity, e , between the intersection of the branch centerlines and the chord centerline. As shown in Figure 9-1, this eccentricity is termed positive if it is on the side of the chord centerline away from the branches. AISC *Specification* Table K3.1A and Table K3.2A have a common eccentricity range (nondimensional with respect to the chord depth) within which the bending moment produced by the nodding (work point) eccentricity can be neglected (according to empirical research) as far as its primary influence on connection strength is concerned. Any nonzero value of eccentricity will produce a primary bending moment in the chord member (whether doing truss analysis manually or using a frame analysis program) that is necessary for equilibrium of the truss. This eccentricity moment has only a secondary influence on the connection strength in that it affects the value of the chord-stress influence parameter, Q_f . The eccentricity moment will also influence the design of the chord members, too, because they will need to be designed for combined axial force and flexure according to AISC *Specification* Chapter H.

The gap or overlap, g or l_{ov} , may be represented by x in the following equations, which are useful for calculating the eccentricity, e , the gap, g , or the overlap, l_{ov} .

$$x = \left(\frac{e + E}{C} \right) - (A + B) \quad (9-1)$$

$$e = C(A + B + x) - E \quad (9-2)$$

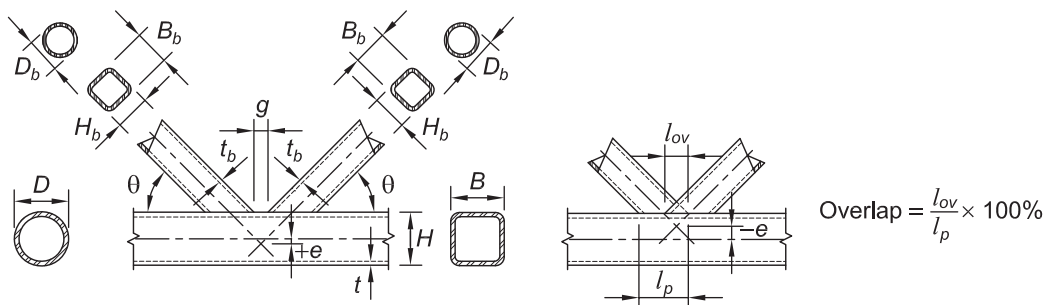


Fig. 9-1. Common notation for HSS truss connections.

where

$$A = \frac{H_{b1}}{2 \sin \theta_{b1}} \text{ or } \frac{D_{b1}}{2 \sin \theta_{b1}} \quad (9-3)$$

$$B = \frac{H_{b2}}{2 \sin \theta_{b2}} \text{ or } \frac{D_{b2}}{2 \sin \theta_{b2}} \quad (9-4)$$

$$C = \frac{\sin \theta_{b1} \sin \theta_{b2}}{\sin(\theta_{b1} + \theta_{b2})} \quad (9-5)$$

$$E = \frac{H}{2} \text{ or } \frac{D}{2} \quad (9-6)$$

When there is a gap, $x = g$; when there is an overlap, $x = -l_{ov}$.

The subscripts $b1$ and $b2$ designate branch 1 and branch 2. These equations also apply for panel points that have a stiffening plate welded to the surface of the chord. In this case, if t_{sp} is the thickness of the doubler plate stiffener, the term E becomes:

$$E = \frac{H}{2} + t_{sp} \text{ or } \frac{D}{2} + t_{sp} \quad (9-7)$$

The fabrication of overlapped K-connections is discussed in Section 3.3 of this Design Guide, including welding options for the “hidden toe” of the overlapped branch member.

9.2.2 Limit States

Typical failure modes (and hence limit states) for truss-type connections are discussed in Chapter 2. Figure 9-2 illustrates, as an example, the potential limit states for a rectangular HSS gapped K-connection. These limit states are summarized for this particular connection type. For K-connections, one branch will be in tension and the other in compression (Section 9.3).

- Chord wall plastification [Figure 9-2(a)] is described in Section 2.3.3 and is represented by the empirical AISC *Specification* Equation K3-7 in Table K3.2 and Table 9-2 in this Design Guide. This is the most prevalent failure mode and applies for all β values. If all the HSS members are square, and limits of applicability are satisfied, tests have shown that this failure mode is critical, and Equation K3-7 is the only one to be checked. The connection available strength can be expressed as a force in either the compression branch member or the tension branch member.
- Shear yielding (punching) of the chord [Figure 9-2(b)] is described in Section 2.4.1 and is represented by AISC *Specification* Equation K3-8 in Table K3.2 and Table 9-2 in this Design Guide. Note that Equation K3-8 is a rewritten version of Equation 2-12 and contains an empirical term, β_{eop} . Figure 9-2(b) is shown for punching out of the tension branch, but it equally applies to punching in of the compression branch.

- Local yielding of the branch due to uneven load distribution [Figures 9-2(c) and 9-2(d)] is described in Section 2.5 and is represented by AISC *Specification* Equation K3-9 in Table K3.2 and Table 9-2 in this Design Guide. Note that Equation K3-9 is a rewritten version of Equation 2-15 and contains an empirical term, B_e . Equation K3-9 contains terms associated with the four walls of a branch and is applicable directly to each branch member.
- Shearing of the chord cross section, in the gap region [Figure 9-2(e)] is described in Section 2.9 and is covered by AISC *Specification* Chapter G, as well as Table K3.2 and Table 9-2 in this Design Guide. Research has shown that this failure mode need only be checked if the chord is rectangular, not square, and oriented such that $H < B$. The available shear strength of the chord member, determined using Equation 2-24 with the relevant ϕ or Ω , is compared to the vertical components of the branch forces.
- The limit states of local yielding of the chord side walls (Section 2.6), local crippling of the chord side walls (Section 2.7), buckling of the chord side walls (Section 2.8), and local buckling of the chord connecting face (Section 2.10), have all been excluded by virtue of the limits of applicability in AISC *Specification* Table K3.2A. Furthermore, these limits of validity also encompass the range of validation, by tests, for the empirical terms mentioned previously. As noted in AISC *Specification* Chapter K Commentary, the AISC *Specification* does not prohibit using connections which fall outside the listed limits of applicability; however, the AISC *Specification* and Commentary do not provide connection capacities or guidance when doing so. A rational approach is left to the designer, and all possible failure modes must be considered.

For rectangular HSS overlapped K-connections meeting the limits of applicability in AISC *Specification* Table K3.2A, the sole limit state to be considered is local yielding of either branch due to uneven load distribution [similar to Figures 9-2(c) and 9-2(d)]. This is described in Section 2.5 and is represented by AISC *Specification* Equations K3-10 to K3-17 in AISC *Specification* Table K3.2 and Table 9-2 in this Design Guide. The design procedure presumes that one branch is welded solely to the chord and therefore has only a single cut at its end. Overlapped rectangular HSS K-connection strength calculations (AISC *Specification* Equations K3-10, K3-11, and K3-12) are performed initially just for the overlapping branch, regardless of whether it is in tension or compression, and then the resistance of the overlapped branch is determined from that. The equations for connection strength, expressed as a force in a branch, are based on the load-carrying contributions of the four side walls of the overlapping branch and follow the design recommendations of IIW (1989) and Packer and Henderson (1997). A maximum overlap of $O_v = 100\%$ occurs when one

branch sits completely on the other. In such cases, the overlapping branch is sometimes moved slightly up the overlapped branch so that the heel of the overlapping branch can be fillet-welded to the face of the overlapped branch. If the connection is fabricated in this manner, an overlap slightly exceeding 100% is created, which is still acceptable. (This is the reason for e/D and e/H being permitted to be as low as -0.55 in Tables K3.1A and K3.2A, respectively). In such cases, the rectangular HSS connection nominal strength can be calculated by AISC Specification Equation K3-12.

9.3 CONNECTION CLASSIFICATION

HSS-to-HSS truss connections consist of one or more branch members that are directly welded to a continuous chord that passes through the connection. The classification of these connections as K- (which includes N-), Y- (which includes T-), or cross- (also known as X-) connections is based on the method of force transfer in the connection, not on the physical appearance of the connection. Thus:

- When the punching load, $P\sin\theta$, in a branch member is equilibrated by beam shear in the chord member, the connection is classified as a T-connection when the branch is perpendicular to the chord and a Y-connection otherwise.
- When the punching load, $P\sin\theta$, in a branch member is essentially equilibrated (to within 20%) by a load in another branch member (or members) on the same side of the connection, the connection is classified as a K-connection. The relevant gap is between the primary branch members whose loads equilibrate.
- When the punching load, $P\sin\theta$, is transmitted through the chord member and is equilibrated by a load in a branch member (or members) on the opposite side of the chord member, the connection is classified as a cross- (or X-) connection.

Examples of these connection classifications are shown in Figure 9-3. If a branch member is unloaded, such as in Figures 9-3(d) and 9-3(f), it is conservative to just remove

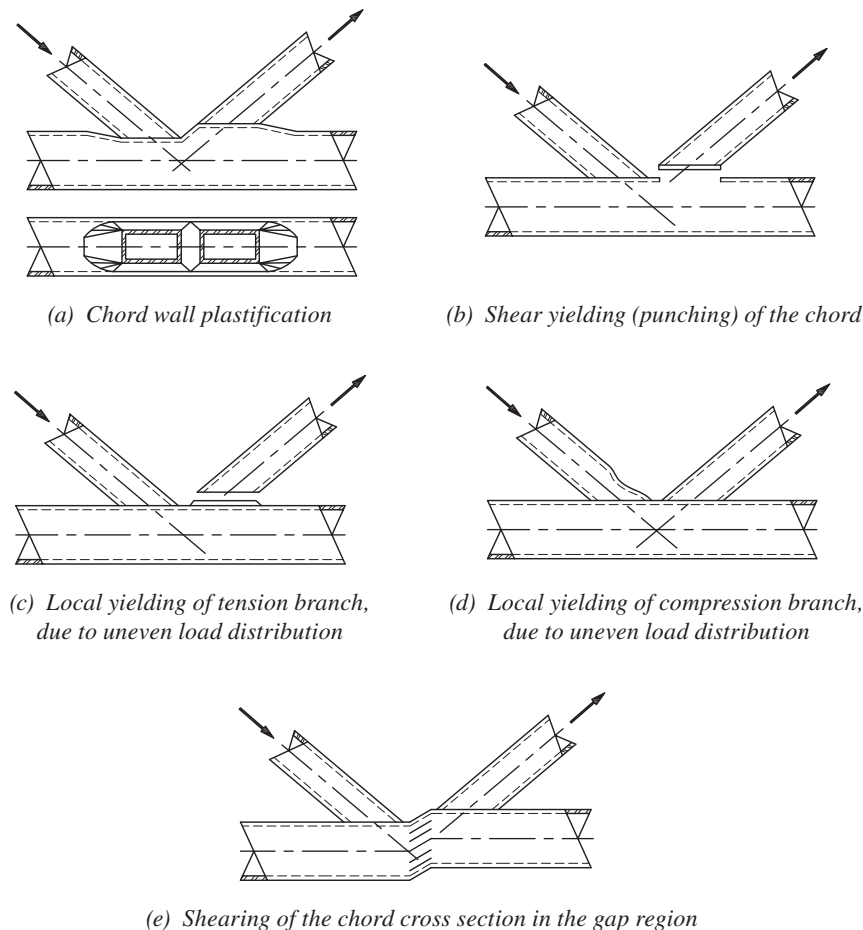


Fig. 9-2. Potential limit states for rectangular HSS-to-HSS gapped K-connections, within the AISC Specification limits of applicability for parameters.

this branch from consideration. Two K-connections, in Figures 9-3(b) and 9-3(i), illustrate that the branch force components normal to the chord member may differ by as much as 20% and still be deemed to exhibit K-connection behavior. This is to accommodate slight variations in branch member forces along a typical truss, caused by panel point loads. Note that Figure 9-3(e) has the appearance of a K-connection but is defined as a cross-connection because the branch forces are transferred through the chord member to the opposite side.

When branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the adequacy of each branch is determined by linear interaction of the proportion of the branch load involved in each type of load transfer. The N-connection in Figure 9-3(c) has a ratio of branch force components normal to the chord member of 2:1 that exceeds the tolerance of 1.2:1 for a K-connection. In the case shown in Figure 9-3(c), the connection needs to be analyzed as both a “pure” K-connection (with balanced branch forces) and a cross- (or X-) connection (because the remainder of the diagonal branch load is being transferred through the connection).

This can be done by drawing the free-body diagrams shown in Figure 9-4, whereby the original loading arrangement is split into a balanced K-connection and a cross-connection. Then, for this connection, the following checks are made for the two branches (letting P_r be the total required axial strength in the diagonal tension branch):

Compression branch:

$$\frac{0.5P_r \sin \theta}{\phi P_{n,Kv}} \leq 1.0 \quad (9-8)$$

Tension branch:

$$\frac{0.5P_r}{\phi P_{n,Kd}} + \frac{0.5P_r}{\phi P_{n,Xd}} \leq 1.0 \quad (9-9)$$

where

$\phi P_{n,Kv}$ = K-connection available strength in the vertical branch, kips

$\phi P_{n,Kd}$ = K-connection available strength in the diagonal branch, kips

$\phi P_{n,Xd}$ = Cross-connection available strength in the diagonal branch, kips

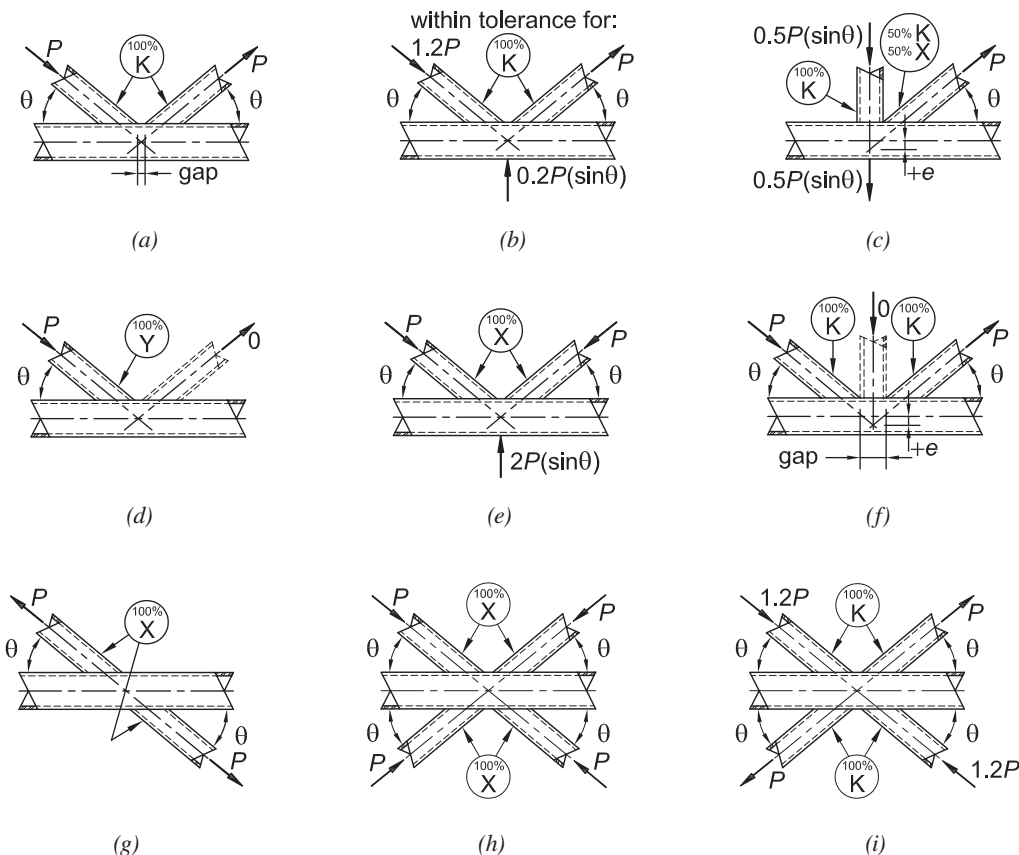


Fig. 9-3. Examples of connection classification.

Example 9.5 gives a demonstration of how design calculations are performed for such a connection. If the gap size in a gapped K- (or N-) connection [e.g., Figure 9-3(a)] becomes large and exceeds the value permitted by the eccentricity limit (in Table 9-1A or Table 9-2A, see Section 9.7), then the branches of the K-connection do not interact in a “push-pull mechanism” and hence should be treated as two independent Y-connections. In cross-connections, such as Figure 9-3(h), where the branches are close together or overlapping, the combined “footprint” of the two branches can be taken as the loaded area on the chord member. Example 9.3 demonstrates this technique for determining the bearing length on the chord. If Figure 9-3(i) has gapped K-connections, it can be seen that there is a high shear force on the cross section of the chord taken through the gap region. In this case, the chord shear strength should be checked (see Section 2.9 and Table 9-2 for rectangular HSS chords). If the double-K-connection shown in Figure 9-3(i) has the forces in the lower K-connection reversed in direction, there would be a significant equilibrating compression force in the chord member. In such cases, this total compression force should be used in computing Q_f (see Tables 9-1 and 9-2) for each K-connection.

9.4 TRUSS MODELING AND MEMBER DESIGN

This chapter, and similarly, AISC *Specification* Section K3, presumes that the branches of triangulated trusses are loaded only by axial forces. Within the constraints of the limits of applicability in Tables 9-1A and 9-2A, welded connections within an HSS truss will, however, be semi-rigid (or partially restrained) but branch member stiffnesses will be considerably less than the chord member stiffness. As a result, the actual bending moments in branch members will be much lower than the chord members. Various methods for modeling HSS trusses have been applied to actual tested trusses (Frater and Packer, 1992a), with the most-recent comprehensive evaluation (Tousignant and Packer, 2019a) considering frame analysis by four methods:

- Model P, in which member segments are all pin-jointed and connected concentrically at all joints regardless of centerline noding (work point) eccentricity.

- Model R, in which member segments are fixed at their ends and connected concentrically at all joints regardless of centerline noding eccentricity.
- Model PR, in which chord members are continuous and web members are pin-connected to a short, rigid link from the chord centerline, with the length of this link being the centerline noding eccentricity.
- Model RR, in which chord members are continuous and web members are joined rigidly to a short, rigid link from the chord centerline, with the length of this link being the centerline noding eccentricity.

For the ultimate limit state design of HSS truss members and their connections, all four analysis methods gave very good estimates of the member axial forces. Because web member moments tend to reduce to near-zero at the ultimate limit state due to connection plasticity and load redistribution, the prediction of chord member moments is of more significance. In that regard, models PR and RR give reasonable predictions, and both take the primary moments in the chords produced by connection noding eccentricity into account (unlike models P and R). If analysis is performed using model RR, the web moments generated can be ignored for both web member design and connection design. Thus, to avoid the unnecessary generation of web member moments, which are not required for use of Tables 9-1 and 9-2, model PR (shown in Figure 9-5) is recommended (Tousignant and Packer, 2019a).

For the serviceability limit state design of HSS trusses to conform to AISC *Specification* Chapter L and ASCE/SEI 7, a reliable method of analysis is required for predicting maximum truss deflections. At service-load levels, model PR generally errs slightly on the unconservative side for deflection predictions, especially with the more-flexible gapped connections. Thus, for analysis using model PR, a magnification factor of 1.1 on computed truss deflections is recommended for trusses with overlapped connections, and a magnification factor of 1.2 on computed truss deflections is recommended for trusses with gapped connections (Tousignant and Packer, 2019a).

When selecting members for an HSS truss, it must be reiterated that minimum weight does not equal minimum cost.

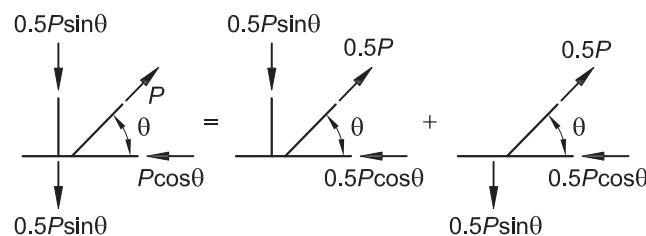


Fig. 9-4. Checking of a K-connection with imbalanced branch member loads.

The number of different member sizes should be small and the number of connections minimized to reduce fabrication cost. For fewer connections, Warren trusses can be a good choice. Because one is generally required to produce an HSS truss design without stiffened connections, it is common to find that member selection is governed by connection design criteria. Thus, as pointed out in a User Note in AISC *Specification* Section K1, connections need to be considered in the design of the members.

The most common cause for HSS connection design problems is the selection of chord members with a high D/t ratio or high B/t ratio, particularly for the compression chord, where a high buckling resistance is sought and, hence, a member with a high radius of gyration may have been initially chosen. Optimal chord members that produce efficient connections are relatively stocky; typical choices for round HSS chord members are in the range of $15 \leq D/t \leq 30$, and typical choices for square HSS chord members are in the range $15 \leq B/t \leq 25$. On the other hand, branch members should, in general, be chosen to have a high D_b/t_b or B_b/t_b ratio, but within the limits permitted by Tables 9-1A and 9-2A. However, branches should preferably still sit on the flat of the chord member for rectangular HSS trusses to simplify welding procedures. Thus, branch members are generally selected to be relatively wide and thin, whereas chord members are generally selected to be stocky and thick, resulting in $t_b < t$. Still, reference to Table 9-1A shows that the requirement for round HSS compression branches ($D_b/t_b \leq 0.05E/F_{yb}$) qualifies such web members as compact sections (see AISC *Specification* Table B4.1b). Similarly, reference to Table 9-2A shows that the requirement for rectangular HSS compression branches in overlapped K-connections (B_b/t_b and $H_b/t_b \leq 1.1\sqrt{E/F_{yb}}$) also qualifies such web members as compact sections (see AISC *Specification* Table B4.1b).

The package of connection design criteria in Tables 9-1 and 9-2, in conjunction with the limits of applicability in Tables 9-1A and 9-2A, permits the use of effective lengths, KL , less than L when designing planar, welded HSS truss compression members. For compression chords, $KL = 0.9L$ can be adopted, where L is the distance between chord panel points (for buckling in the plane of the truss) or L is the distance between points of lateral support for the chord (for buckling perpendicular to the truss). For compression web members, the use of $KL = 0.75L$ is permissible, where L is the distance between member nodes or work points (CIDECT, 1980; Mouty, 1981; Rondal et al., 1992).

Long, laterally unsupported compression chords can exist in U-frame [or “half-through” (Pony) truss] pedestrian bridges and in roof trusses subjected to large wind uplift (the bottom chord). The effective length of such laterally unsupported truss chords can be considerably less than the unsupported length, and design guidance is provided in Packer and Henderson (1997) and Ziemian (2010).

9.5 ROUND HSS BRANCHES TO A RECTANGULAR HSS CHORD

AISC *Specification* Chapters J and K cover round branch-to-round chord HSS welded connections, and rectangular branch-to-rectangular chord HSS welded connections, but hybrid trusses involving a mix of round and rectangular HSS members do occasionally occur. Pipe sections may potentially be readily sourced and used for branch members, yet square or rectangular HSS might be preferred for the chords to make fabrication easier. This combination of hollow sections has been advocated in design guides since the inception of the rectangular HSS (Abrahams, 1961). As shown in Figure 9-6(a), only straight cuts to the round branches are required for gapped K-connections (or T- and Y-connections). For overlapped K-connections, however,

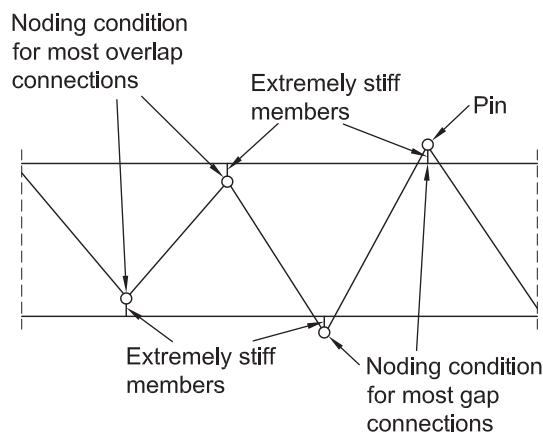


Fig. 9-5. Recommended method for modeling HSS trusses using web members pin-connected to continuous chord members (model PR).

some tube profiling will be required for the overlapping round HSS branch member [Figure 9-6(b)]; the overlapped round branch would be straight cut and welded directly to the flat chord face. Hybrid trusses using square or rectangular branches welded to round HSS (or pipe) chord members would be very impractical because of the profiling required for all branch member ends.

Experimental research has been performed on a considerable number of round-to-rectangular HSS N-connections, and it has been observed that the connection limit states resemble those of similar rectangular HSS connections. An inspection of contemporary design recommendations reveals that most failure modes for hollow section connections depend on the circumference or “footprint” of the branch member at the joint with the chord or depend on the cross-sectional area of the branch members. The perimeter of a round HSS of diameter D_b is πD_b , and that of a square HSS of width D_b is $4D_b$ (excluding the rounded corners), thus approximately in the ratio of $\pi:4$. Similarly, the cross-sectional area of a thin round HSS of diameter D_b and thickness t_b is approximately $\pi D_b t_b$, whereas the cross-sectional area of a thin square HSS of width D_b and thickness t_b is approximately $4D_b t_b$, again in the ratio of $\pi:4$. It was then surmised that round-to-rectangular HSS welded connections could potentially be treated as square-to-rectangular HSS connections by converting the round branch members to square sections, and this notion was validated against experimental results over a particular β range (Packer et al., 2007). Recent international design recommendations for HSS connections (Packer et al., 2009; ISO, 2013) have then assumed each round branch to be a square of width D_b , calculated the connection resistance using relevant rectangular HSS design

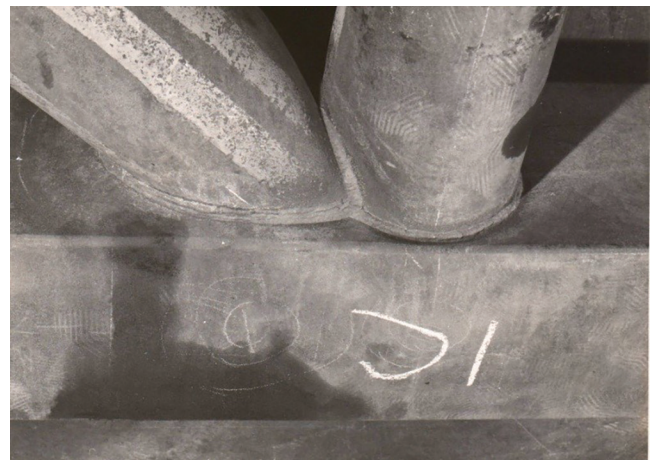
formulas, and then multiplied this connection capacity by $\pi/4$.

With regard to Table 9-2, round HSS branches with $0.25 \leq D_b/B \leq 0.80$ can be accommodated by the following:

- *For T-, Y-, and cross-connections*, replace both B_b and H_b by D_b in the equations and participating variables for the limit states of chord plastification, shear yielding (punching), and local yielding of the branch; then multiply the resulting connection available strength by $\pi/4$. No reduction by $\pi/4$ is required for the chord shear limit state (in the rare instances where this limit state applies). The limits of applicability are generally those in Table 9-2A, but round branches are also subject to the D_b/t_b limits in Table 9-1A.
- *For gapped K-connections*, replace both B_b and H_b by D_b in the equations and participating variables for the limit states of chord plastification, shear yielding (punching), and local yielding of the branches; then multiply the resulting connection available strength by $\pi/4$. No reduction by $\pi/4$ is required for the chord shear limit state. The limits of applicability are generally those in Table 9-2A, but round branches are also subject to the D_b/t_b limits in Table 9-1A and $D_b/B \geq 0.1 + \gamma/50$.
- *For overlapped K-connections*, replace both B_{bi} and H_{bi} by D_{bi} , plus B_{bj} by D_{bj} , in the relevant nominal strength equation and participating variables for the limit state of local yielding of the overlapping branch; then multiply the resulting connection available strength by $\pi/4$. The nominal strength equation for the overlapped branch requires no modification. The limits of applicability are generally those in Table 9-2A, but round branches are also subject to the D_b/t_b limits in Table 9-1A.



(a) Gapped K-connection



(b) Overlapped K-connection

Fig. 9-6. Round branch-to-rectangular/square chord connections.

9.6 KT-CONNECTIONS

HSS Warren trusses with K-connections, which have two diagonal branch members, are frequently modified by the introduction of a third vertical branch to form a so-called KT-connection, as shown in Figure 9-7. The vertical branch may be added to support an applied load between panel points or to reduce the effective length of a chord member but, in general, this vertical member is often lightly loaded. The design of statically loaded, planar, HSS KT-connections is beyond the scope of the *AISC Specification*, nor are such connections covered in the latest HSS design guidance from CIDECT (Wardenier et al., 2008a; Packer et al., 2009; Wardenier et al., 2010), IIW (2012), or ISO (2013). The reason for this lack of contemporary coverage is the realization that there are a very large number of possible configurations for members in KT-connections, combined with a large number of possible loading arrangements for the members. Very little research exists on HSS KT-connections, so a synthesis of “best practice” guidance is offered herein (Packer, 2014).

9.6.1 Methods of Analysis

Some of the possible load combinations on the three branch members of a KT-connection are shown in Figure 9-8. Figures 9-8(a) through 9-8(d) illustrate combinations where the three branch member forces are in vertical equilibrium (i.e.,

normal to the chord direction), with the two diagonals either having the opposite or the same force sense. Figures 9-8(e) through 9-8(h) have the same branch member force sense as in Figures 9-8(a) through 9-8(d), but some load is additionally transferred through the chord member. Figures 9-8(i) and 9-8(j) have all the load on one side of the connection transferred through the chord member to the other side; thus, these connections can be analyzed as cross- (or X-) connections.

Gapped KT-Connections

When all three branches have gaps between them, at the junction with the chord connecting face, a suggested method of connection analysis is as follows, for five cases:

1. In Figures 9-8(a) and 9-8(b), the force in branch 3 can be apportioned into two parts, each of which balances the vertical components of the forces in branches 1 and 2.
2. Thus, the total connection can be subdivided into two K-connections (bearing in mind that N-connections are a special case of the general K-connection), consisting of branches 1 and 3 and branches 2 and 3. The two K-connections can then be checked using the procedures in *AISC Specification* Section K3. The total utilization of branch 3 in each subconnection would also need to be checked in the manner outlined in Section 9.3. A calculation example of a K-connection, where a branch



Fig. 9-7. HSS KT-connection.

participates in two separate subconnections (or free-body diagrams), is given in Example 9.5.

2. In Figures 9-8(e) and 9-8(f), the procedure is similar to that in case 1, but the total-force, free-body diagram now needs to be split into separate free-body diagrams consisting of a cross-connection plus two K-connections, with the vertical branch 3 being checked for its utilization in three sub-connections, as shown in Figure 9-9(a).
3. In Figures 9-8(c) and 9-8(d), two neighboring branches have the same force sense, and these two branches could be possibly considered to have a “combined action.” This is the one case of a KT-connection covered in EN 1993-1-8 (CEN, 2005), where all the branches are illustrated with gaps between each other at the chord connecting face. In Eurocode 3, Table 7.15, the checking method (using AISC *Specification* LRFD terminology) is to confirm that [with reference to Figure 9-8(c)],

$$P_1 \sin \theta_1 + P_3 \sin \theta_3 \leq \phi P_{n1} \sin \theta_1 \quad (9-10)$$

$$P_2 \sin \theta_2 \leq \phi P_{n1} \sin \theta_1 \quad (9-11)$$

where P_1 , P_2 , and P_3 are the axial forces in branches 1, 2, and 3, respectively, and P_{n1} is the connection's nominal strength expressed as an axial force in branch 1, per AISC *Specification* Equation K3-7. Also, it is stipulated that the value of β_{eff} in AISC *Specification* Equation K3-16 be calculated by averaging over the three branches (b_1 , b_2 , and b_3) as follows:

$$\beta_{eff} = \frac{B_{b1} + H_{b1} + B_{b2} + H_{b2} + B_{b3} + H_{b3}}{6B} \quad (9-12)$$

This method has the following drawbacks: (1) It is tailored to the limit state of chord wall plastification, (2) it applies only to gapped KT-connections with this unique pattern of branch loads, and (3) it presumes that the diagonal branches and their forces dominate. The EN 1993 (CEN, 2005) procedure has also been applied to round-to-round, gapped, HSS KT-connections, pointing out that $(D_{b \text{ comp}}/D)$ in AISC *Specification* Equation K3-4 should be replaced by $(D_{b1} + D_{b2} + D_{b3})/3D$, where D_{b1} , D_{b2} , and D_{b3} are the outside diameters of branches 1, 2, and 3, respectively. Very similar methods to this EN 1993 technique have been cited earlier by Wardenier et al. (1991), Packer et al. (1992) and Packer and Henderson (1992, 1997), where the gap is recommended to be taken as “the largest gap between two [branches] having significant forces acting in the opposite sense” (Packer et al., 1992). Packer et al. (1992, 1997) also used a variant of Equation 9-11, still with reference to Figure 9-8(c), as given in Equation 9-13:

$$P_2 \sin \theta_2 \leq \phi P_{n2} \sin \theta_2 \quad (9-13)$$

where P_{n2} is the connection nominal strength measured as a force in branch 2, per AISC *Specification* Equation K3-7.

Despite all of the foregoing for case 3, it is much more logical, however, if the free-body diagram of KT-connection

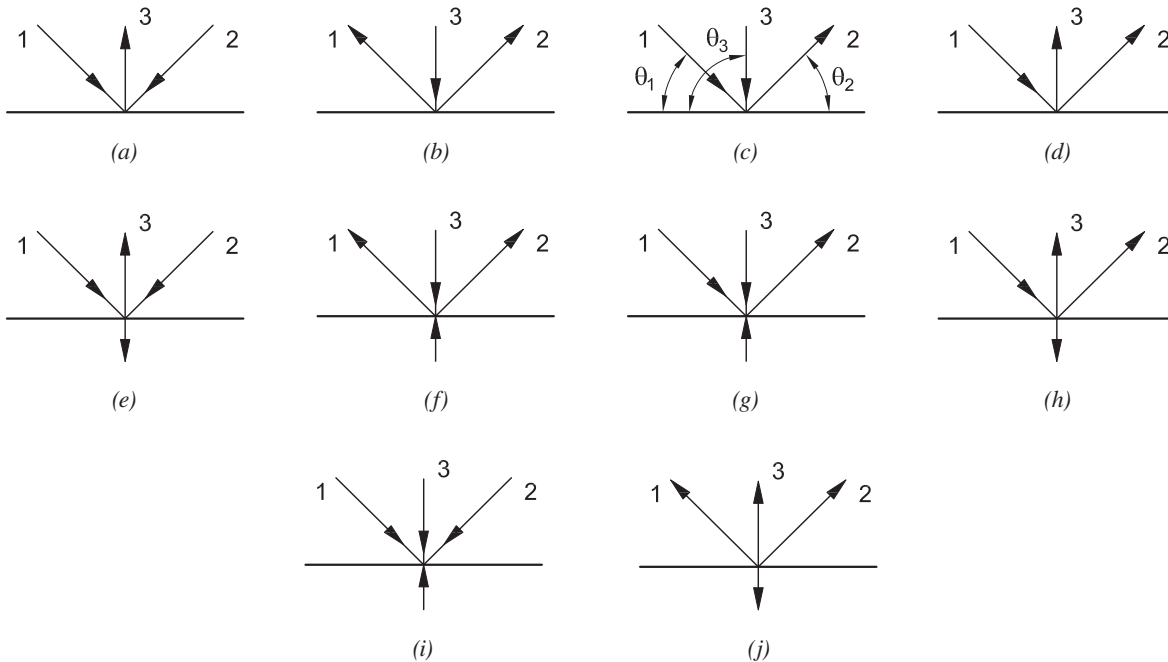


Fig. 9-8. Examples of load combinations on KT-connections (Tata Steel, 2011).

forces is again broken into its constituent subconnections, as illustrated in Figure 9-9(b), and analyzed in this preferred manner.

4. In Figures 9-8(g) and 9-8(h), the procedure is similar to Figure 9-9(b), except an additional cross-connection component will be introduced, thus making three separate subconnections (or free-body diagrams).
5. In Figures 9-8(i) and 9-8(j), the connection is a single cross-connection in both cases because all branch-force components normal to the chord member are transferred through the chord. A calculation example of a very similar cross-connection is given in Example 9.3.

If the vertical branch in a gapped KT-connection has zero (or near-zero) force in it, then it can be ignored, and the connection can be treated as a K-connection, with the gap taken as the distance between the toes of branches 1 and 2 in Figure 9-8. Also see Figure 9-3(f). This will be very conservative because the mere presence of additional steel (branch 3) welded to the gap region will stiffen the connection.

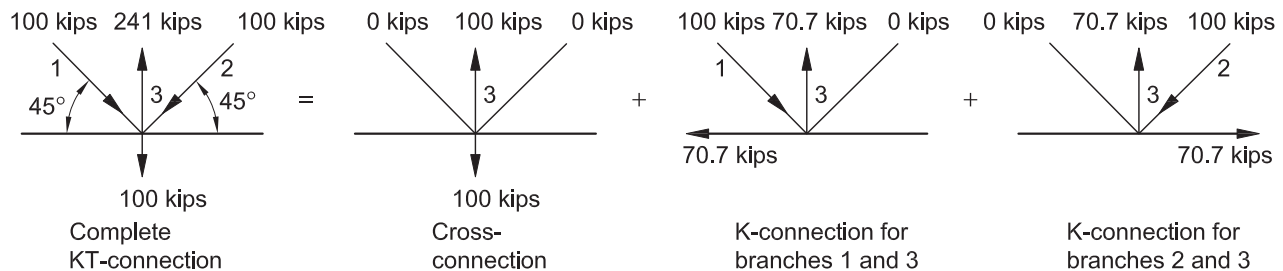
Overlapped KT-Connections

Overlapped KT-connections are much more probable than gapped KT-connections because the latter produces a large positive nodding eccentricity that is likely to violate the limit of applicability for connection eccentricity in AISC *Specification* Tables K3.1A or K3.2A. The common types of overlapped KT-connections are shown in Figure 9-10. The sequence of overlapping should follow the basic premise

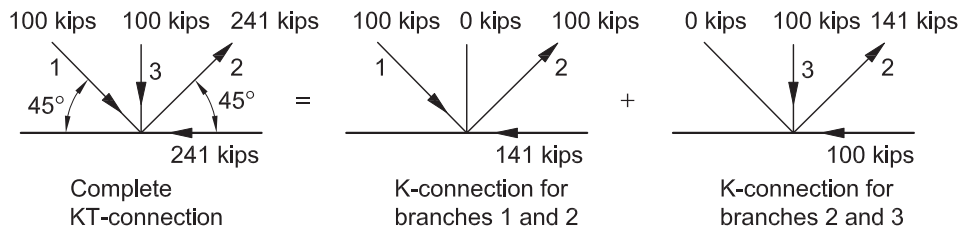
that narrower branch members “sit on” (or frame into) wider members. If two overlapping branch members have the same width, then the thinner should sit on the thicker branch (i.e., the thicker branch should be the through member). As with overlapped K-connections, overlapped KT-connections should have (at least) one branch welded directly to the chord.

The resistance of round HSS overlapped KT-connections can be handled in a similar way to cases 1 through 5 described for gapped KT-connections, which involves splitting the free-body diagram of connection axial loads into subconnections involving K- and cross-connection types. Branch members participating in multiple subconnection types need to have their total utilization checked to ensure that it is less than unity by linear addition of their respective utilizations in each subconnection. As noted previously, refer to Example 9.5. The available strength of overlapped K-connections between round HSS is based only on the limit state of chord plastification (AISC *Specification* Equations K3-4 and K3-5 in Table K3.1). The amount of overlap, O_v , or negative gap, g , to be used in AISC *Specification* Equation K3-6 pertains to the two branches under consideration in a particular subconnection. Again, if the vertical branch in an overlapped KT-connection has zero (or near-zero) force in it, then it can be ignored, and the connection treated as a K-connection.

The available strength of rectangular and square HSS overlapped KT-connections can be determined on a branch-by-branch basis, in a similar manner to overlapped K-connections, using AISC *Specification* Equations K3-10 to K3-17 in Table K3.2 and Table 9-2 of this Design Guide.



(a) Example of Fig. 9-8(e) broken into its constituent subconnections



(b) Example of Fig. 9-8(c) broken into its constituent subconnections

Fig. 9-9. Recommended analysis methods for gapped KT-connections.

This is demonstrated in Example 9.6. This method of checking overlapped KT- and K-connections is different for square and rectangular HSS connections compared to round HSS connections (described previously) in the *AISC Specification*. However, it is worth noting here that the most recent international design guidance for round HSS overlapped K-connections (Wardenier et al., 2008a; Wardenier et al., 2010; IIW, 2012; ISO, 2013) translates the round HSS branches into equivalent square HSS and then proceeds to use the square and rectangular HSS checking method.

9.7 HSS-TO-HSS TRUSS CONNECTION AVAILABLE STRENGTH TABLES

The available strength equations for round and rectangular HSS-to-HSS truss connections as well as their limits of applicability are presented in Tables 9-1, 9-1A, 9-2, and 9-2A.

9.8 MULTI-PLANAR CONNECTIONS

In practice, a great number of hollow section structures have not just planar welded connections but multi-planar connections. These arise in towers, offshore structures, crane gantries, bridges, canopies, and roof structures, some examples of which are shown in Figure 9-11.

Many space frames are designed as pin-connected systems, although fully welded 3D triangulated trusses can still be designed on the basis of a model using continuous chords with pin-connected web members. This results in web member axial forces only; thus, the design of many multi-planar welded connections (to a main or “through” member) can be related to their axially loaded planar connection counterparts. Terminology for multi-planar connections has been adopted such that if a second plane at a node contains a second X- (or cross-), T-, or K-connection, then the connection is termed an XX-, TT-, or KK-connection, respectively. Similarly, a T-branch in a plane orthogonal to

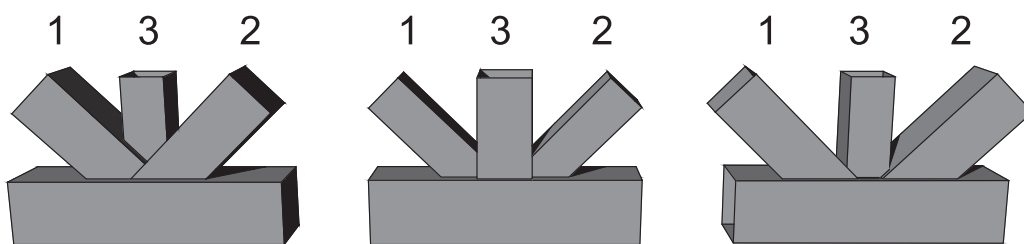


Fig. 9-10. Common types of overlapped KT-connections.



(a) Tower



(b) Triangular truss with KK-connections

Fig. 9-11. Examples of multi-planar round HSS connections.

Table 9-1. Available Strengths of Round HSS-to-HSS Truss Connections

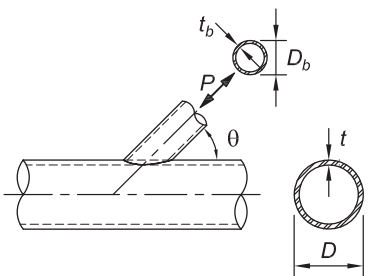
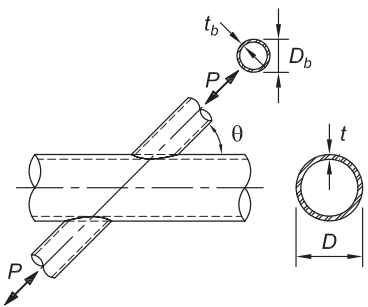
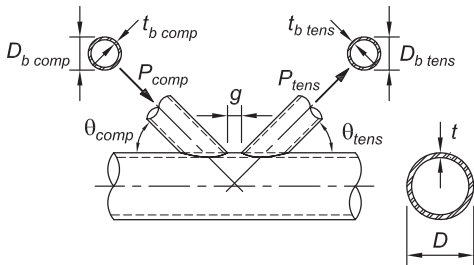
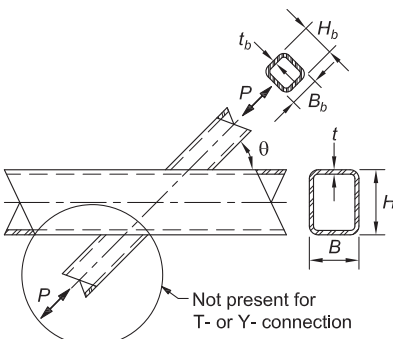
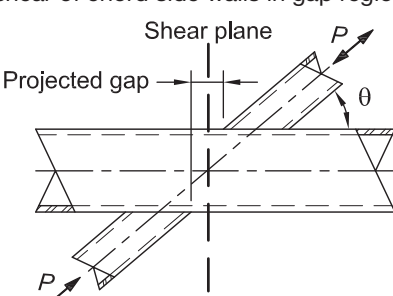
Connection Type	Connection Available Axial Strength ¹
General check For T-, Y-, cross-, and K-connections with gap, when $D_{b(tens/comp)} < (D - 2t)$	Limit State 1: Chord shear yielding (punching shear) $P_n = 0.6F_y t \pi D_b \left(\frac{1 + \sin \theta}{2 \sin^2 \theta} \right)$ (K3-1) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
T- and Y- connections 	Limit State 2: Chord plastification $P_n \sin \theta = F_y t^2 (3.1 + 15.6 \beta^2) \gamma^{0.2} Q_f$ (K3-2) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
Cross-connections 	Limit State 3: Chord plastification $P_n \sin \theta = F_y t^2 \left(\frac{5.7}{1 - 0.81 \beta} \right) Q_f$ (K3-3) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
K-connections with gap or overlap 	Limit State 4: Chord plastification $(P_n \sin \theta)_{compression\ branch} = F_y t^2 \left(2.0 + 11.33 \frac{D_{b\ comp}}{D} \right) Q_g Q_f$ (K3-4) $(P_n \sin \theta)_{tension\ branch} = (P_n \sin \theta)_{compression\ branch}$ (K3-5) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
Functions $Q_f = 1.0$ for chord connecting surface in tension $= 1.0 - 0.3U$ ($1 + U \leq 1.0$ for chord (connecting surface) in compression) $U = \left \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right \leq 1.0$ where P_{ro} and M_{ro} are determined on the side of the connection that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the chord: $P_{ro} = P_u$ for LRFD, P_a for ASD; $M_{ro} = M_u$ for LRFD, M_a for ASD. $Q_g = \gamma^{0.2} \left[1 + \frac{0.024 \gamma^{1.2}}{\exp \left(\frac{0.5g}{t} - 1.33 \right) + 1} \right]$ Note that $\exp(x)$ is equal to e^x, where $e = 2.71828$ is the base of the natural logarithm. ¹ Equation references are to the AISC Specification.	

Table 9-1A. Limits of Applicability for Limit States in Table 9-1

For Limit States 1 to 4 (per AISC Specification Table K3.1A):

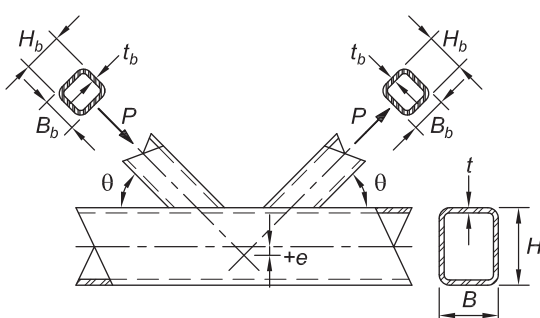
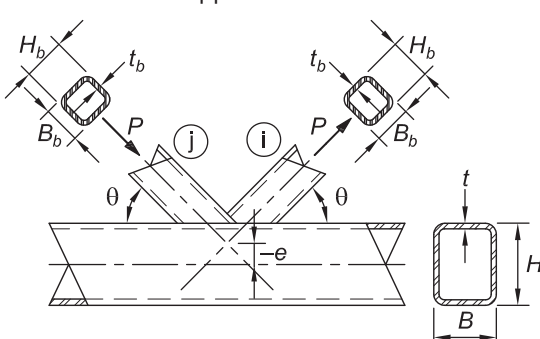
Connection eccentricity:	$-0.55 \leq e/D \leq 0.25$ for K-connections
Chord wall slenderness:	$D/t \leq 50$ for T-, Y-, and K-connections
	$D/t \leq 40$ for cross-connections
Branch wall slenderness:	$D_b/t_b \leq 50$ for tension and compression branch
	$D_b/t_b \leq 0.05E/F_{yb}$ for compression branch
Width ratio:	$0.2 \leq D_b/D \leq 1.0$ for T-, Y-, cross-, and overlapped K-connections
	$0.4 \leq D_b/D \leq 1.0$ for gapped K-connections
Gap:	$g \geq t_{b \text{ compression branch}} + t_{b \text{ tension branch}}$, for gapped K-connections (for welding at toes)
Overlap:	$25\% \leq O_v \leq 100\%$ for overlapped K-connections
Branch thickness:	$t_{b \text{ overlapping}} \leq t_{b \text{ overlapped}}$ for branches in overlapped K-connections
Material strength:	F_y and $F_{yb} \leq 52$ ksi
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \leq 0.8$ Note: ASTM A500/A500M Grade C is acceptable.
End distance:	$l_{end} \geq D \left(1.25 - \frac{\beta}{2} \right)$ for T-, Y-, cross-, and K-connections

Table 9-2. Available Strengths of Rectangular HSS-to-HSS Truss Connections

Connection Type	Connection Available Axial Strength ¹
T-, Y-, and cross-connections  Not present for T- or Y- connection Case for checking the limit state of shear of chord side walls in gap region 	Limit State 1: Chord plastification $P_n \sin \theta = F_y t^2 \left(\frac{2\eta}{1-\beta} + \frac{4}{\sqrt{1-\beta}} \right) Q_f$ (2-2)
	$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit State 2: Chord shear yielding (punching shear) $P_n \sin \theta = 0.6 F_y t B (2\eta + 2\beta_{eop})$ (2-10), (2-11)
	$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 3: Local yielding of the branch/branches due to uneven load distribution $P_n = F_{yb} t_b (2H_b + 2B_e - 4t_b)$ (K1-1), (2-14), (2-15)
	$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 4: Local yielding of the chord sidewalls $P_n \sin \theta = 2F_y t (7.5t + H_b / \sin \theta)$ (J10-2), (2-17)
	$\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit State 5: Local crippling of the chord sidewalls $P_n \sin \theta = 1.6t^2 \left(1 + \frac{3H_b / \sin \theta}{H} \right) \sqrt{EF_y} Q_f$ (J10-4), (2-19)
	$\phi = 0.75$ (LRFD) $\Omega = 2.00$ (ASD)
	Limit State 6: Buckling of the chord sidewalls $P_n \sin \theta = \left(\frac{48t^3}{H - 3t} \right) \sqrt{EF_y} Q_f$ (J10-8), (2-23)
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
	Limit State 7: Shear of the chord sidewalls in gap region $P_n \sin \theta = 0.6 F_y [2t(H - 3t)]$ (G4-1), (2-24)
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

(Table 9-2 continues on the next page)

Table 9-2. Available Strengths of Rectangular HSS-to-HSS Truss Connections (continued)

Connection Type	Connection Available Axial Strength ¹
Gapped K-connections 	Limit State 8: Chord plastification $P_n \sin \theta = F_y t^2 (9.8 \beta_{eff} \gamma^{0.5}) Q_f$ (K3-7)
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
	Limit State 9: Chord shear yielding (punching shear) $P_n \sin \theta = 0.6 F_y t B (2\eta + \beta + \beta_{eop})$ (K3-8)
	$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 10: Local yielding of the branch/branches due to uneven load distribution $P_n = F_{yb} t_b (2H_b + B_b + B_e - 4t_b)$ (K3-9)
	$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 11: Shear of the chord sidewalls in gap region $P_n \sin \theta = 0.6 F_y [2t(H - 3t)]$ (G4-1), (2-24)
Overlapped K-connections  Note that the force arrows shown for overlapped K-connections may be reversed; i and j control member identification.	Limit State 12: Local yielding of the branch/branches due to uneven load distribution When $25\% \leq O_v < 50\%$: $P_{n,i} = F_{yb_i} t_{b_i} \left[\frac{O_v}{50} (2H_{b_i} - 4t_{b_i}) + B_{eoi} + B_{eov} \right]$ (K3-10) When $50\% \leq O_v < 80\%$: $P_{n,i} = F_{yb_i} t_{b_i} (2H_{b_i} - 4t_{b_i} + B_{eoi} + B_{eov})$ (K3-11) When $80\% \leq O_v \leq 100\%$: $P_{n,i} = F_{yb_i} t_{b_i} (2H_{b_i} - 4t_{b_i} + B_{bi} + B_{eov})$ (K3-12) Subscript i refers to the overlapping branch Subscript j refers to the overlapped branch $P_{n,j} = P_{n,i} \left(\frac{F_{yb_j} A_{b_j}}{F_{yb_i} A_{b_i}} \right)$ (K3-15)
	$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

(Table 9-2 continues on the next page)

Table 9-2. Available Strengths of Rectangular HSS-to-HSS Truss Connections (continued)

Functions

$Q_f = 1.0$ for chord connecting surface in tension

$$= 0.4 \leq \left(1.3 - 0.4 \frac{U}{\beta} \right) \leq 1.0 \text{ for chord connecting surface in compression, for T-, Y-, and cross-connections} \quad (\text{K1-4})$$

$$= 0.4 \leq \left(1.3 - 0.4 \frac{U}{\beta_{eff}} \right) \leq 1.0 \text{ for chord connecting surface in compression for gapped K-connections} \quad (\text{K1-5})$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{K1-6})$$

where P_{ro} and M_{ro} are determined on the side of the connection that has the higher compression stress.

P_{ro} and M_{ro} refer to required strengths in the chord: $P_{ro} = P_u$ for LRFD, P_a for ASD; $M_{ro} = M_u$ for LRFD, M_a for ASD.

$$\beta_{eff} = \left[(B_b + H_b)_{compression\ branch} + (B_b + H_b)_{tension\ branch} \right] / 4B \quad (\text{K3-16})$$

$$\beta_{eop} = \frac{B_{ep}}{B} = \left(\frac{10}{B/t} \right) \left(\frac{B_b}{B} \right) = \frac{5\beta}{\gamma} \leq \beta \quad (\text{K3-17})$$

$$B_e = \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b \quad (\text{K1-1})$$

$$B_{eoi} = \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{ybi} t_{bi}} \right) B_{bi} \leq B_{bi} \quad (\text{K3-13})$$

$$B_{eov} = \left(\frac{10}{B_{bj}/t_{bj}} \right) \left(\frac{F_{ybj} t_{bj}}{F_{ybi} t_{bi}} \right) B_{bi} \leq B_{bi} \quad (\text{K3-14})$$

¹ Equation references are to the AISC *Specification* or in this Design Guide

Table 9-2A. Limits of Applicability for Limit States in Table 9-2

For Limit States 1 to 12:	
Limit State 1:	$\beta = B_b/B < 1.0$; plus restrictions on chord wall slenderness, width ratio, material strength and ductility (because of empirical Q_f), and end distance apply.
Limit State 2:	$B_b \leq (B - 2t)$; plus restrictions on chord connecting wall slenderness, B/t , width ratio, branch aspect ratio, material strength and ductility (because of empirical β_{eop}), and end distance apply. This limit state need not be checked if $B/t \geq 10$.
Limit State 3:	Restrictions on chord connecting wall slenderness, B/t , branch wall slenderness, width ratio, branch aspect ratio, material strength and ductility (because of empirical B_e), and end distance apply. This limit state need not be checked when $\beta < 0.85$.
Limit State 4:	$\beta \approx 1.0$; plus restriction on end distance.
Limit State 5:	Applicable for T- or Y-connections, branch is loaded in compression, and $\beta \approx 1.0$. Also, restrictions on chord wall slenderness, material strength and ductility (because of empirical Q_f), and end distance apply.
Limit State 6:	Applicable for cross-connections, branches are loaded in compression, and $\beta \approx 1.0$. Note that AISC <i>Specification</i> Equation J10-8 and Equation 2-23 apply only to bearing lengths less than the chord depth [i.e., $(H_b/H \sin \theta) < 1.0$]. For $(H_b/H \sin \theta) \geq 1.0$, use "column buckling" approach in Section 2.8. Also, restrictions on chord wall slenderness, material strength and ductility (because of empirical Q_f), and end distance apply.
Limit State 7:	Applicable for cross-connections, with $\theta < 90^\circ$ and a projected gap as shown in figure in Table 9-2. Also, restrictions on chord wall slenderness, width ratio, chord aspect ratio, and material strength and ductility apply.
Limit State 8:	Restrictions on connection eccentricity, chord wall slenderness, branch wall slenderness, width ratio, branch and chord aspect ratios, gap, branch size ratio, and material strength and ductility apply (because of empirical formula).
Limit State 9:	$B_b \leq (B - 2t)$; plus restrictions on connection eccentricity, chord connecting wall slenderness, B/t , width ratio, branch aspect ratio, gap, branch size ratio, and material strength and ductility apply (because of empirical β_{eop}). This limit state need not be checked if branches are square.
Limit State 10:	Restrictions on connection eccentricity, chord connecting wall slenderness, B/t , branch wall slenderness, width ratio, branch aspect ratio, gap, branch size ratio, and material strength and ductility apply (because of empirical B_e). This limit state need not be checked if branches are square or if $B/t \geq 15$.
Limit State 11:	$H < B$; plus restrictions on connection eccentricity, chord wall slenderness, width ratio, chord aspect ratio, gap, branch size ratio, and material strength and ductility apply.
Limit State 12:	Restrictions on connection eccentricity, chord connecting wall slenderness, B/t , branch wall slenderness, width ratio, branch and chord aspect ratios, overlap, branch size ratio and branch thickness ratio, and material strength and ductility apply.

(Table 9-2A continues on next page)

Table 9-2A. Limits of Applicability for Limit States in Table 9-2 (continued)

Parameter Limits (per AISC Specification Table K3.2A)	
Connection eccentricity: ¹	$-0.55 \leq e/H \leq 0.25$ for K-connections
Chord connecting wall slenderness:	$B/t \leq 35$ for T-, Y-, and cross-connections, and gapped K-connections $B/t \leq 30$ for overlapped K-connections
Chord side wall slenderness:	$H/t \leq 35$
Branch wall slenderness:	B_b/t_b and $H_b/t_b \leq 35$ for tension branch $\leq 1.25 \sqrt{\frac{E}{F_{yb}}}$ for compression branch of T-, Y-, and cross-connections, and gapped K-connections ≤ 35 for compression branch of T-, Y-, and cross-connections, and gapped K-connections $\leq 1.1 \sqrt{\frac{E}{F_{yb}}}$ for compression branch of overlapped K-connections
Width ratio:	B_b/B and $H_b/B \geq 0.25$ for T-, Y-, and cross-connections, and overlapped K-connections B_b/B and $H_b/B \geq (0.1 + \gamma/50)$ for gapped K-connections $\beta_{eff} \geq 0.35$ for gapped K-connections
Aspect ratio:	$0.5 \leq H_b/B_b \leq 2.0$ and $0.5 \leq H/B \leq 2.0$
Overlap:	$25\% \leq O_v \leq 100\%$ for overlapped K-connections
Gap: ¹	$0.5(1 - \beta_{eff}) \leq \xi = g/B \leq 1.5(1 - \beta_{eff})$ for gapped K-connections $g \geq t_{b \text{ compression branch}} + t_{b \text{ tension branch}}$, for gapped K-connections (for welding at toes)
Branch size ratio:	$B_{\text{overlapping branch}}/B_{\text{overlapped branch}} \geq 0.75$, for overlapped K-connections $B_{\text{smaller branch}}/B_{\text{larger branch}} \geq 0.63$, for gapped K-connections, if both branches are square
Branch thickness ratio:	$t_{\text{overlapping branch}}/t_{\text{overlapped branch}} \leq 1.0$, for overlapped K-connections
Material strength:	F_y and $F_{yb} \leq 52$ ksi
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \leq 0.8$ Note: ASTM A500/A500M Grade C is acceptable.
End distance:	$l_{end} \geq B\sqrt{1-\beta}$ for T-, Y-, and cross-connections with $\beta \leq 0.85$
¹ Note: When $e/H > 0.25$ or $g/B > 1.5(1 - \beta_{eff})$, treat as two separate Y-connections.	

a planar X-connection would produce a multi-planar TX-connection (or alternatively it could be viewed as an XT-connection), and so forth. However, such connection labels are still subject to the connection classification rules based on force transfer, as described in Section 9.3. These basic multi-planar connection types are shown, relative to their uniplanar counterparts, in Figure 9-12.

Until 2015, AWS D1.1/D1.1M provided a means of handling multi-planar effects for round HSS connections, but the design rules are relatively complex. They are developed mainly from elastic considerations and, depending on the configuration and loading of the connections, they may be unreliable for strength predictions. A great number of researchers, as summarized by Packer and Henderson (1997), Wardenier et al. (2008a), and Packer et al. (2009), have performed experimental and numerical studies on multi-planar, welded HSS connections. Research increasingly identified that the “multi-planar effect” is caused by both loading and geometric effects, the latter being a function especially of the width ratio, β , and the chord wall slenderness, 2γ , as for planar connections. The mere addition of more branches to a connection (i.e., welding on more steel) often serves to stiffen the connection region and provide an increase in strength.

The recommended approach by CIDECT (Wardenier et al., 2008a; Packer et al., 2009) for designing multi-planar connections is to consider a connection type by considering

each individual plane, on a plane-by-plane basis. While doing so, most limit state formulas need no adjustment. However, for the chord plastification limit state, to account for the effect of the branches lying out-of-plane, an adjustment by a correction factor, μ , is required. Considering the beneficial effect of welded-on branches, and ignoring the loading effect, this factor μ can be taken as 0.9 or 1.0 in many cases. This has the condition that the total chord force is used to calculate the planar connection Q_f value, using either the total “prestress” for round HSS chords, or the total “maximum stress” for rectangular HSS chords. The correction factors to the planar connection chord plastification limit state formulas (in Tables 9-1 and 9-2) are summarized for common connection types in Table 9-3 for round HSS and Table 9-4 for rectangular HSS. Particular remarks with regard to XX-connections and KK-connections follow.

9.8.1 XX-Connection Design

With a planar X- (or cross-) connection as shown in Figure 9-12, under equal and opposite branch compression forces, the round HSS chord flattens into an oval shape. For a rectangular HSS, the loaded chord faces bend inward and the two unloaded web walls curve outward, while the corners tend to approximately maintain their 90° angles. Thus, for both shapes the chord sides bulge outward. If compression-loaded, out-of-plane branches are now added (as in the XX-connection of Figure 9-12), one can see that such forces tend

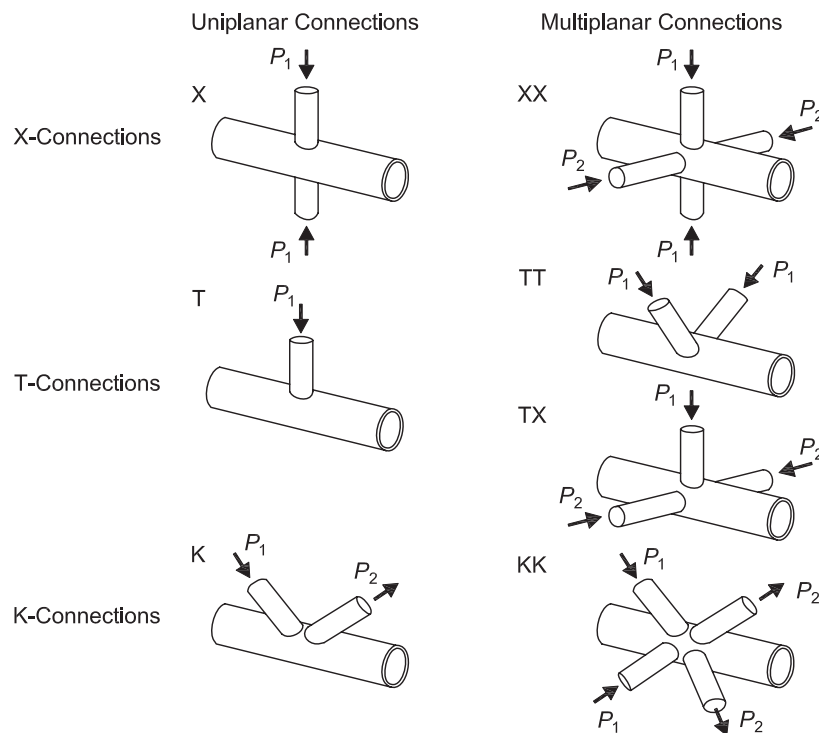
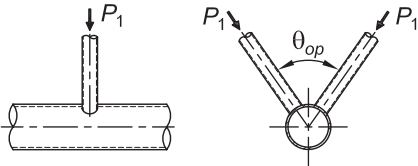
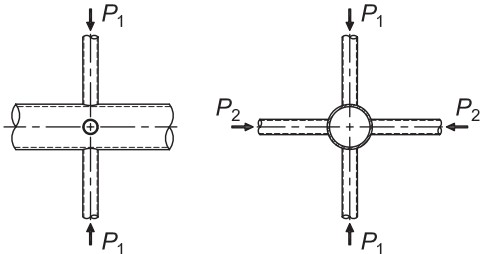
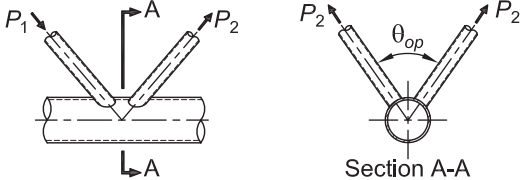


Fig. 9-12. Multi-planar welded connections, related to planar welded connections.

Table 9-3. Correction Factors for Multi-Planar Round HSS Connections

Connection Type	Correction Factor for Chord Plastification Available Strength in Table 9-1 ¹
TT-connections Members 1 may be either in tension or compression 	$\mu = 1.0$, applied to Equation (K3-2)
XX-connections Members 1 and 2 may be either in tension or compression 	$\mu = 1 + 0.35 \frac{P_2}{P_1}$, applied to Equation (K3-3) Notes: 1. Take account of the sign of P_1 and P_2, with $P_1 \geq P_2$. 2. P_2/P_1 is negative if the branches in one plane are in tension and in compression in the other plane.
KK-connections Members 1: compression Members 2: tension  Section A-A	$\mu = 0.9$, applied to Equations (K3-4) and (K3-5) Note: In a gapped connection, the chord cross section in the gap (Section A-A) must be checked for shear failure, using: $\left(\frac{P_{gap}}{\phi P_{pl}} \right)^2 + \left(\frac{V_{gap}}{\phi V_{pl}} \right)^2 \leq 1.0$ where P_{gap} = axial force in gap $P_{pl} = AF_y$ V_{gap} = shear force in gap $V_{pl} = 0.6F_y (2A/\pi)$ $\phi = 0.90$
Limits of applicability	Same as Table 9-1 $60^\circ \leq \theta_{op} \leq 90^\circ$

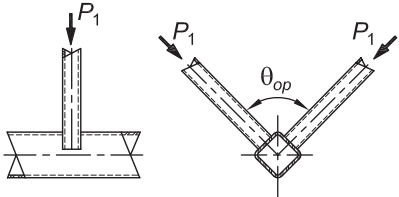
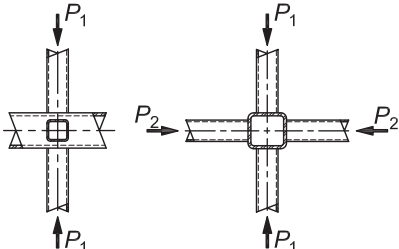
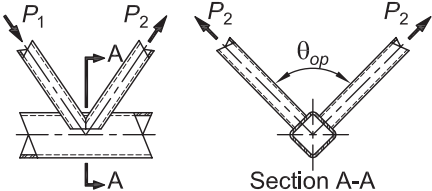
¹ Note: Each branch still needs to be checked individually for the chord shear yielding limit state.

to restore the chord section to its original shape and thus have a positive effect in opposing chord deformation. Conversely, tension-loaded, out-of-plane branches would have the opposite effect and would exacerbate the connection deformation. Thus, in Tables 9-3 and 9-4, the planar strength correction factor, μ , is highly dependent upon the relative sense of the forces in the pairs of X-branches. The XX-connection, with orthogonal planes of X-branches loaded in the opposite sense, is the one multi-planar connection case that can produce a severe reduction (up to 35%) relative to its uniplanar counterpart.

9.8.2 KK-Connection Design

For round HSS KK-connections, a simple reduction factor of $\mu = 0.9$ can be applied to account for multi-planar effects, as indicated in Table 9-3. This follows CIDECT Design Guide No. 1, 1st Ed. (Wardenier et al., 1991), because this particular CIDECT guide—like the AISC *Specification*—is based on a specific “chord pre-stress” (the lower chord stress) function for calculating the round HSS Q_f factor. (This can be seen in the Functions part of AISC *Specification* Table K3.1 and in Table 9-1). Also, the two recommendations (CIDECT Design Guide No. 1, 1st Ed., and the AISC *Specification*)

Table 9-4. Correction Factors for Multi-Planar Rectangular HSS Connections

Connection Type	Correction Factor for Chord Plastification Available Strength in Table 9-2 ¹
TT-connections Members 1 may be either in tension or compression 	$\mu = 1.0$, applied to Equation (2-2)
XX-connections Members 1 and 2 may be either in tension or compression 	$\mu = 1 + 0.35 \frac{P_2}{P_1}$, applied to Equation (2-2) Notes: 1. Take account of the sign of P_1 and P_2, with $P_1 \geq P_2$. 2. P_2/P_1 is negative if the branches in one plane are in tension and in compression in the other plane.
Gapped KK-connections Members 1: compression Members 2: tension  Section A-A	$\mu = 0.9$, applied to Equation (K3-7) Note: In a gapped connection, the chord cross section in the gap (Section A-A) must be checked for shear failure, using: $\left(\frac{P_{gap}}{\phi P_{pl}} \right)^2 + \left(\frac{0.71 V_{gap}}{\phi V_{pl}} \right)^2 \leq 1.0$ where P_{gap} = axial force in gap $P_{pl} = AF_y$ V_{gap} = shear force in gap $V_{pl} = 0.6F_y (0.5A)$ for a square chord $\phi = 0.90$
Limits of applicability	Same as Table 9-2 $\theta_{op} \approx 90^\circ$

¹ Note: Other limit states still need to be checked.

have common limit state checks for gapped and overlapped K-connections.

For rectangular HSS gapped KK-connections, a simple reduction factor of $\mu = 0.9$ can also be applied to account for multi-planar effects, as indicated in Table 9-4. This follows CIDECT Design Guide No. 3, 1st Ed. (Packer et al., 1992), because this particular CIDECT guide—like the AISC *Specification*—is based on a specific “chord maximum stress” function for calculating the rectangular HSS Q_f factor, and

both recommendations have common limit state checks for gapped and overlapped K-connections. This manner of determining Q_f is also incorporated in Table 9-2.

The force in a single chord member can be considerably higher in a KK-connection than in a K-connection because of the force components parallel to the chord from branches in two planes. Similarly, the force components normal to the chord from branches in two planes can be considerably higher in a KK-connection than in a K-connection. Thus, for

gapped KK-connections, it is recommended that a free-body diagram be produced showing the chord forces acting on a section taken through the gap region and then a check be made for failure in the gap region under combined normal and shear stress. This section A-A is shown in Table 9-3 for round HSS and Table 9-4 for rectangular HSS. For round HSS KK-connections, the total “vertical” shear force in the gap, V_{gap} , is applied to the circular cross section where the chord shear area is taken to be $(2/\pi)A$ (Wardenier et al., 2010). For the rectangular HSS gapped KK-connection, with an angle of $\theta_{op} = 90^\circ$ between the branch planes, the connection is subjected to a shear force of $(\sqrt{2}/2)V_{gap}$ in each plane. The shear force in each plane is resisted by two walls of the rectangular HSS chord. The horizontal force components from the two planes equilibrate. For both round and rectangular HSS, the shear yield stress is taken as $0.6F_y$ in accordance with the AISC *Specification*.

9.8.3 Fabrication Aspects with KK-Connections

Round HSS

A common initial layout for 3D modeling is to have a single work point for multiple branches on the chord centerline, but designers should be aware that this may involve more complicated fabrication procedures. In a single plane, two branch members in a K-connection may overlap each other, but in a KK-connection, the branch members from one plane may also overlap those from another plane. An increasing number of overlapping branches leads to more branch-end profiling, more complex fit-up, and more welding issues. To avoid overlapping branch members from one side plane onto those of the other, it may be necessary to introduce a “transverse gap” between neighboring planes, but this may introduce a work-point offset perpendicular to the chord axis.

This work-point (or noding) eccentricity should not exceed $0.25D$ in order for the bending moment generated to be ignored in terms of its primary effect on connection design. As noted in Section 9.2.1, the eccentricity moment has some influence on the connection strength because it affects the value of Q_f , the chord-stress interaction parameter. The eccentricity moment will always influence the design of the chord members because they will need to be designed for combined axial force and flexure.

In a KK-connection, if the two compression web members overlap each other and the two tension web members overlap each other, but there is still a gap, g , along the chord between the compression and tension web members (under symmetrical loading), then the combined two-branch “foot-print” can be treated as one unit with the KK-connection analyzed as a K-connection.

Rectangular HSS

Triangular or “Delta” (Δ) trusses are frequently arranged in the form of a “V” that combines one bottom chord with two top chords, as illustrated in Figure 9-13 and Figure 9-11(b). In Figure 9-13, the bottom chord is rotated so that the web members can be welded to the chord flat sides, with just straight cuts to the web member ends.

Under gravity loading, this Delta truss has the advantage of increased lateral stability offered by the twin, separated, but connected compression chords. These trusses are frequently used as exposed structures and are considered equivalent in appearance, but less expensive, than space frames.

Unrotated square HSS chords have also been used with web members framing into the corners of the HSS chord section, by shaping the web member ends to form so-called “bird beak” connections. For example, this fabrication technique was used for the Minneapolis Convention Center roof,

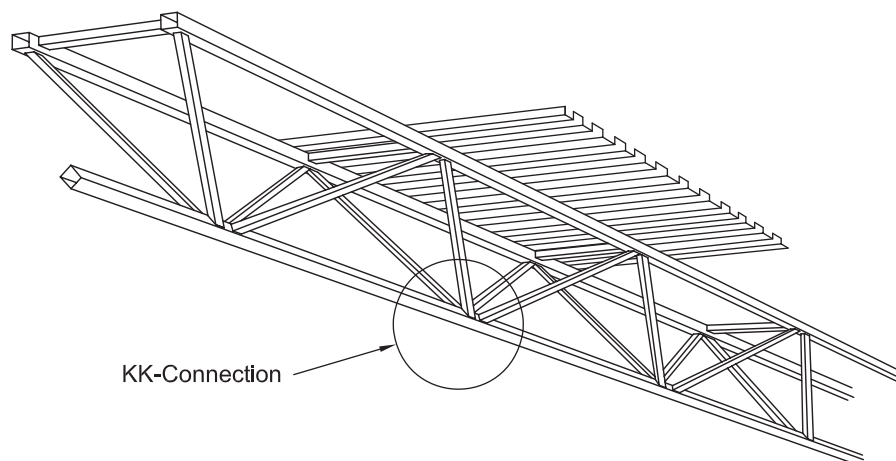


Fig. 9-13. HSS triangular truss with two compression chords and a rotated tension chord.

as shown in Figure 9-14. However, it is less expensive to rotate the bottom chord by 45° and to bevel cut the web ends so that they land on the flat chord faces (as shown for KK-connections in Table 9-4). When serving as a Delta roof

truss, purlins may not be necessary, because the top chords can be spaced at a suitable distance apart for the roof deck to span between. The decking can then be fastened directly to the flat surfaces of the HSS top chord members.

9.9 TRUSS CONNECTION DESIGN EXAMPLES

EXAMPLE 9.1—Y-Connection with Round HSS

Given:

Determine the adequacy of the welded HSS Y-connection under the member loads shown in Figure 9-15. Loads consist of 25% dead load, P_D , and 75% live load, P_L . Assume the welds have sufficient strength to develop the yield strength of the connected branch wall at all locations around the branch. All HSS members are ASTM A500/A500M Grade C.

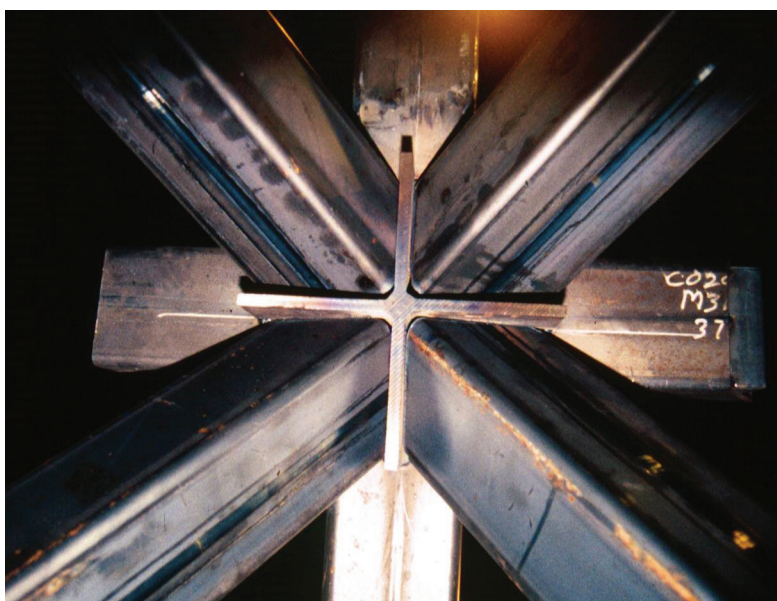


Fig. 9-14. Minneapolis Convention Center roof connection, with web members framing into the chord corners (photo courtesy of Mr. Larry Kloiber and LeJeune Steel).

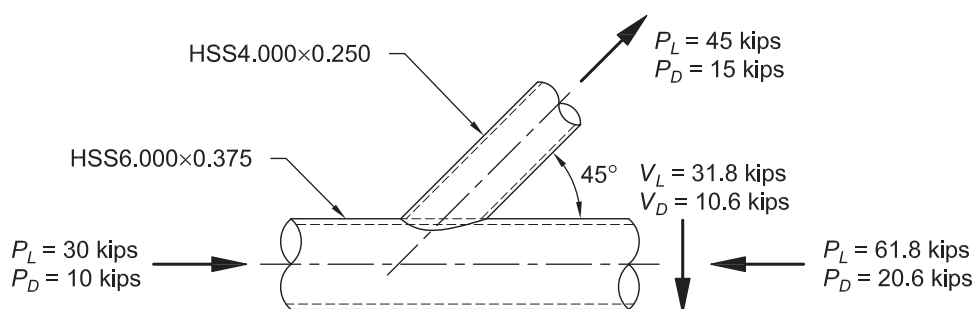


Fig. 9-15. Welded round HSS Y-connection for Example 9.1.

From AISC *Manual* Table 2-4, the material properties are as follows:

All members

ASTM A500/A500M Grade C

$$F_y = 50 \text{ ksi}$$

$$F_u = 62 \text{ ksi}$$

From AISC *Manual* Table 1-13, the geometric properties are as follows:

Chord

HSS6.000×0.375

$$D = 6.00 \text{ in.}$$

$$t = 0.349 \text{ in.}$$

$$A = 6.20 \text{ in.}^2$$

Branch

HSS4.000×0.250

$$D_b = 4.00 \text{ in.}$$

$$t_b = 0.233 \text{ in.}$$

$$A_b = 2.76 \text{ in.}^2$$

Solution:

From AISC *Specification* Table K3.1 and Table 9-1, there are two potential limit states for round HSS-to-HSS Y-connections.

$$\begin{aligned} D - 2t &= 6.00 \text{ in.} - 2(0.349 \text{ in.}) \\ &= 5.30 \text{ in.} \end{aligned}$$

$$D_b = 4.00 \text{ in.} < 5.30 \text{ in.}$$

Chord shear yielding applies.

Limits of applicability

From Table 9-1A, for a Y-connection, the limits of applicability are:

Chord wall slenderness:

$$\begin{aligned} D/t &= 6.00 \text{ in.}/0.349 \text{ in.} \leq 50 \\ &= 17.2 < 50 \quad \mathbf{o.k.} \end{aligned}$$

Branch wall slenderness:

$$\begin{aligned} D_b/t_b &= 4.00 \text{ in.}/0.233 \text{ in.} \leq 50 \\ &= 17.2 < 50 \quad \mathbf{o.k.} \end{aligned}$$

Width ratio:

$$\begin{aligned} \beta &= D_b/D \\ &= 4.00 \text{ in.}/6.00 \text{ in.} \\ &= 0.667 \end{aligned}$$

$$0.2 \leq 0.667 \leq 1.0 \quad \mathbf{o.k.}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

$$F_{yb} = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$F_y / F_u = 50 \text{ ksi} / 62 \text{ ksi} \leq 0.8$$

$$= 0.806 > 0.8$$

$$F_{yb} / F_{ub} = 50 \text{ ksi} / 62 \text{ ksi} \leq 0.8$$

$$= 0.806 > 0.8$$

However, the note in Table 9-1A indicates ASTM A500/A500M Grade C is acceptable. **o.k.**

End distance:

$$l_{end} \geq D \left(1.25 - \frac{\beta}{2} \right)$$

$$\geq (6.00 \text{ in.}) \left(1.25 - \frac{0.667}{2} \right)$$

$$\geq 5.50 \text{ in.}$$

Thus, the toe and heel of the branch must be located a minimum of 5.50 in. from an uncapped chord end.

Required strength (expressed as a force in the branch)

From ASCE/SEI 7, Chapter 2, the required tensile strength of the branch is:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(15 \text{ kips}) + 1.6(45 \text{ kips})$ $= 90.0 \text{ kips}$	$P_a = P_D + P_L$ $= 15 \text{ kips} + 45 \text{ kips}$ $= 60.0 \text{ kips}$

Chord shear yielding (punching shear)

From AISC *Specification* Table K3.1 and Table 9-1 (Limit State 1), the nominal strength is:

$$P_n = 0.6F_y t \pi D_b \left(\frac{1 + \sin \theta}{2 \sin^2 \theta} \right) \quad (\text{Spec. Eq. K3-1})$$

$$= 0.6(50 \text{ ksi})(0.349 \text{ in.})(\pi)(4.00 \text{ in.}) \left(\frac{1 + \sin 45^\circ}{2 \sin^2 45^\circ} \right)$$

$$= 225 \text{ kips}$$

The available strength for the chord shear yielding limit state is:

LRFD	ASD
$\phi = 0.95$ $\phi P_n = 0.95(225 \text{ kips})$ $= 214 \text{ kips} > 90.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.58$ $\frac{P_n}{\Omega} = \frac{225 \text{ kips}}{1.58}$ $= 142 \text{ kips} > 60.0 \text{ kips} \quad \mathbf{o.k.}$

Chord plastification

From AISC *Specification* Table K3.1 and Table 9-1 (Limit State 2), the nominal strength is:

$$P_n \sin \theta = F_y t^2 (3.1 + 15.6\beta^2) \gamma^{0.2} Q_f \quad (\text{Spec. Eq. K3-2})$$

where

$$\gamma = D/2t$$

$$= \frac{6.00 \text{ in.}}{2(0.349 \text{ in.})}$$

$$= 8.60$$

$$Q_f = 1.0 - 0.3U(1+U) \leq 1.0$$

(Spec. Eq. K1-3)

For a chord connecting surface in compression:

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0$$

(Spec. Eq. K1-6)

From ASCE/SEI 7, Chapter 2, the required strength in the chord on the side of the connection with the lower compression stress is:

LRFD	ASD
$P_{ro} = P_u$ $= 1.2P_D + 1.6P_L$ $= 1.2(10 \text{ kips}) + 1.6(30 \text{ kips})$ $= 60.0 \text{ kips}$ $M_{ro} = 0 \text{ kip-in.}$ $F_c = F_y$ $= 50 \text{ ksi}$ $U = \left \frac{60.0 \text{ kips}}{(50 \text{ ksi})(6.20 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.194 < 1.0$ $= 0.194$ $Q_f = 1.0 - 0.3(0.194)(1+0.194) \leq 1.0$ $= 0.931 < 1.0$ $= 0.931$	$P_{ro} = P_u$ $= P_D + P_L$ $= 10 \text{ kips} + 30 \text{ kips}$ $= 40.0 \text{ kips}$ $M_{ro} = 0 \text{ kip-in.}$ $F_c = 0.6F_y$ $= 0.6(50 \text{ ksi})$ $= 30.0 \text{ ksi}$ $U = \left \frac{40.0 \text{ kips}}{(30 \text{ ksi})(6.20 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.215 < 1.0$ $= 0.215$ $Q_f = 1.0 - 0.3(0.215)(1+0.215) \leq 1.0$ $= 0.922 < 1.0$ $= 0.922$

The available strength for the chord plastification limit state is:

LRFD	ASD
$\phi = 0.90$ $P_n = (50 \text{ ksi})(0.349 \text{ in.})^2 [3.1 + 15.6(0.667)^2]$ $\times (8.60)^{0.2} (0.931) / \sin 45^\circ$ $= 124 \text{ kips}$ $\phi P_n = 0.90(124 \text{ kips})$ $= 112 \text{ kips} > 90.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.67$ $P_n = (50 \text{ ksi})(0.349 \text{ in.})^2 [3.1 + 15.6(0.667)^2]$ $\times (8.60)^{0.2} (0.922) / \sin 45^\circ$ $= 123 \text{ kips}$ $\frac{P_n}{\Omega} = \frac{123 \text{ kips}}{1.67}$ $= 73.7 \text{ kips} > 60.0 \text{ kips} \quad \mathbf{o.k.}$

EXAMPLE 9.2—Overlapped K-Connection with Round HSS

Given:

Determine the adequacy of the welded N-connection (a particular case of K-connection) shown in Figure 9-16. Note that the chord moment is necessary for equilibrium because of the noding (or work point) eccentricity. The connection is a balanced K-connection because the load in the vertical compression branch member is equilibrated (within 20%) by the vertical component of the tension branch member [see AISC *Specification* Section K3(b) and Section 9.3 in this Design Guide]. Note that the thicker branch member is the through branch. The loads shown consist of live and dead loads in the ratio of 2.5:1. All HSS members are ASTM A500/A500M Grade C. Assume the welds are strong enough to develop the yield strength of the connected branch wall at all locations around the branch.

From AISC *Manual* Table 2-4, the material properties are as follows:

All members
ASTM A500/A500M Grade C
 $F_y = 50$ ksi
 $F_u = 62$ ksi

From AISC *Manual* Table 1-13, the geometric properties are as follows:

Chord
HSS10.000×0.500
 $D = 10.0$ in.
 $t = 0.465$ in.
 $A = 13.9$ in.²

Branch (compression)
HSS6.625×0.375
 $D_b = 6.625$ in.
 $t_b = 0.349$ in.
 $A_b = 6.88$ in.²

Branch (tension)
HSS6.625×0.250
 $D_b = 6.625$ in.
 $t_b = 0.233$ in.
 $A_b = 4.68$ in.²

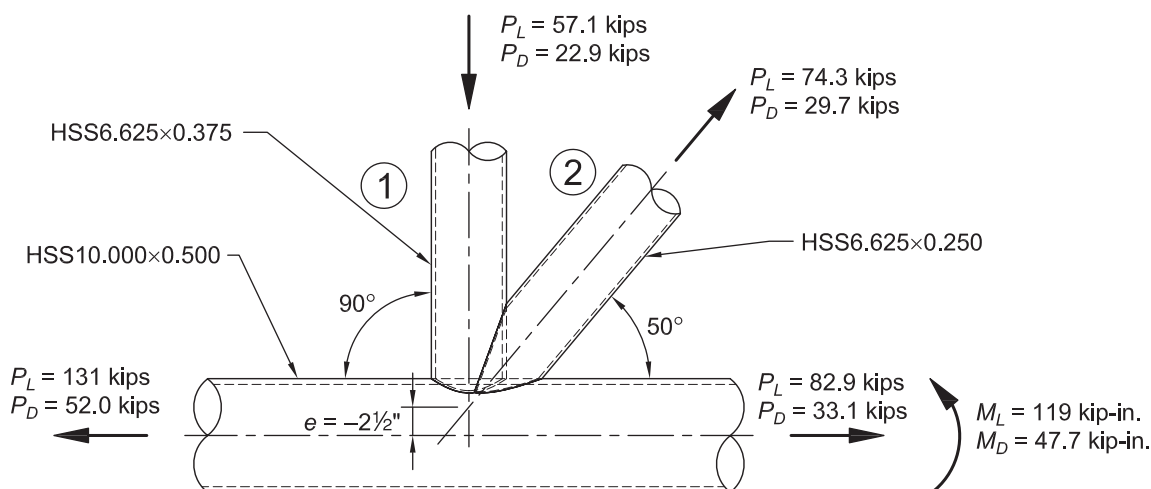


Fig. 9-16. Welded overlap K-connection with round HSS for Example 9.2.

Solution:

From AISC *Specification* Table K3.1 and Table 9-1, there is only one limit state for round HSS-to-HSS overlapped K-connections.

Limits of applicability

From Table 9-1A, for an overlapped K-connection, the limits of applicability are:

Connection eccentricity:

Note that the nodding eccentricity, e , is negative because the branch centerlines intersect toward the inside of the truss, relative to the chord centerline

$$e/D = -2.50 \text{ in.}/10.0 \text{ in.}$$

$$= -0.250$$

$$-0.55 \leq -0.250 \leq 0.25 \quad \text{o.k.}$$

Chord wall slenderness:

$$D/t = 10.0 \text{ in.}/0.465 \text{ in.} \leq 50$$

$$= 21.5 < 50 \quad \text{o.k.}$$

Branch wall slenderness:

For the tension branch:

$$D_b/t_b = 6.625 \text{ in.}/0.233 \text{ in.} \leq 50$$

$$= 28.4 < 50 \quad \text{o.k.}$$

For the compression branch:

$$D_b/t_b = 6.625 \text{ in.}/0.349 \text{ in.} \leq 50$$

$$= 19.0 < 50 \quad \text{o.k.}$$

$$0.05E/F_{yb} = 0.05(29,000 \text{ ksi})/50 \text{ ksi}$$

$$= 29.0 \geq D_b/t_b = 19.0 \quad \text{o.k.}$$

Width ratio:

$$\beta = D_b/D$$

$$= 6.625 \text{ in.}/10.0 \text{ in.}$$

$$= 0.663$$

$$0.2 \leq 0.663 \leq 1.0 \quad \text{o.k.}$$

Overlap:

Overlap length measured along the connecting face of the chord beneath the two branches:

$$l_{ov} = -x$$

(from Eq. 9-1)

$$= (A + B) - \left(\frac{e + E}{C} \right)$$

$$= \left(\frac{D_{b1}}{2 \sin \theta_{b1}} + \frac{D_{b2}}{2 \sin \theta_{b2}} \right) - \left[\frac{\frac{e + D/2}{\sin \theta_{b1} \sin \theta_{b2}}}{\sin(\theta_{b1} + \theta_{b2})} \right]$$

$$= \left(\frac{6.625 \text{ in.}}{2 \sin 90^\circ} + \frac{6.625 \text{ in.}}{2 \sin 50^\circ} \right) - \left[\frac{-2\frac{1}{2} \text{ in.} + 5.00 \text{ in.}}{\left(\frac{\sin 90^\circ \sin 50^\circ}{\sin 140^\circ} \right)} \right]$$

$$= 5.54 \text{ in.}$$

Projected length of the overlapping branch on the chord:

$$\begin{aligned}l_p &= D_{b2}/\sin \theta_{b2} \\&= 6.625 \text{ in.}/\sin 50^\circ \\&= 8.65 \text{ in.}\end{aligned}$$

$$\begin{aligned}O_v &= (l_{ov}/l_p) \times 100\% \\&= \frac{5.54 \text{ in.}}{8.65 \text{ in.}} \times 100\% \\&= 64.0\%\end{aligned}$$

$$25\% \leq 64.0\% \leq 100\% \quad \mathbf{o.k.}$$

Branch thickness:

$$t_{bi} = 0.233 \text{ in.} < t_{bj} = 0.349 \text{ in.} \quad \mathbf{o.k.}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

$$F_{yb} = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$\begin{aligned}F_y/F_u &= 50 \text{ ksi}/62 \text{ ksi} \\&= 0.806 > 0.8\end{aligned}$$

$$\begin{aligned}F_{yb}/F_{ub} &= 50 \text{ ksi}/62 \text{ ksi} \\&= 0.806 > 0.8\end{aligned}$$

However, the note in Table 9-1A indicates ASTM A500/A500M Grade C is acceptable. **o.k.**

End distance:

$$\begin{aligned}l_{end} &\geq D \left(1.25 - \frac{\beta}{2} \right) \\&\geq (10.0 \text{ in.}) \left(1.25 - \frac{0.663}{2} \right) \\&\geq 9.19 \text{ in.}\end{aligned}$$

Therefore, the outside of the vertical branch and the heel of the diagonal branch must be located a minimum of 9.19 in. from an uncapped chord end.

Required strength (expressed as a force in the branch)

From ASCE/SEI 7, Chapter 2, the required strengths are:

LRFD	ASD
For the compression branch: $P_u = 1.2P_D + 1.6P_L$ $= 1.2(22.9 \text{ kips}) + 1.6(57.1 \text{ kips})$ $= 119 \text{ kips}$ For the tension branch: $P_u = 1.2P_D + 1.6P_L$ $= 1.2(29.7 \text{ kips}) + 1.6(74.3 \text{ kips})$ $= 155 \text{ kips}$	For the compression branch: $P_a = P_D + P_L$ $= 22.9 \text{ kips} + 57.1 \text{ kips}$ $= 80.0 \text{ kips}$ For the tension branch: $P_a = P_D + P_L$ $= 29.7 \text{ kips} + 74.3 \text{ kips}$ $= 104 \text{ kips}$

Chord plastification

From AISC *Specification* Table K3.1 and Table 9-1 (Limit State 4), the nominal strength is:

$$(P_n \sin \theta)_{\text{compression branch}} = F_y t^2 \left(2.0 + 11.33 \frac{D_{b \text{ comp}}}{D} \right) Q_g Q_f \quad (\text{Spec. Eq. K3-4})$$

where

$Q_f = 1.0$ because the chord connecting surface is in tension

$$Q_g = \gamma^{0.2} \left[1 + \frac{0.024 \gamma^{1.2}}{\exp \left(\frac{0.5g}{t} - 1.33 \right) + 1} \right] \quad (\text{Spec. Eq. K3-6})$$

$$g = -l_{ov}$$

$$= -5.54 \text{ in.}$$

$$\gamma = D/2t$$

$$= \frac{10.0 \text{ in.}}{2(0.465 \text{ in.})}$$

$$= 10.8$$

Therefore,

$$Q_g = (10.8)^{0.2} \left\{ 1 + \frac{0.024(10.8)^{1.2}}{\exp \left[\frac{0.5(-5.54 \text{ in.})}{0.465 \text{ in.}} - 1.33 \right] + 1} \right\}$$

$$= 2.28$$

and

$$(P_n \sin \theta)_{\text{compression branch}} = (50 \text{ ksi})(0.465 \text{ in.})^2 \left[2.0 + 11.33 \left(\frac{6.625 \text{ in.}}{10.0 \text{ in.}} \right) \right] (2.28)(1.0)$$

$$= 234 \text{ kips}$$

$$P_{n \text{ compression branch}} = \frac{234 \text{ kips}}{\sin 90^\circ}$$

$$= 234 \text{ kips}$$

$$(P_n \sin \theta)_{\text{tension branch}} = (P_n \sin \theta)_{\text{compression branch}} \quad (\text{Spec. Eq. K3-5})$$

$$= 234 \text{ kips}$$

$$P_{n \text{ tension branch}} = \frac{234 \text{ kips}}{\sin 50^\circ}$$

$$= 305 \text{ kips}$$

The available strengths for the chord plastification limit state, expressed as forces in the tension and compression branches are:

LRFD	ASD
$\phi = 0.90$ For the compression branch: $\phi P_n = 0.90(234 \text{ kips})$ $= 211 \text{ kips} > 119 \text{ kips} \quad \text{o.k.}$ For the tension branch: $\phi P_n = 0.90(305 \text{ kips})$ $= 275 \text{ kips} > 155 \text{ kips} \quad \text{o.k.}$	$\Omega = 1.67$ For the compression branch: $\frac{P_n}{\Omega} = \frac{234 \text{ kips}}{1.67}$ $= 140 \text{ kips} > 80.0 \text{ kips} \quad \text{o.k.}$ For the tension branch: $\frac{P_n}{\Omega} = \frac{305 \text{ kips}}{1.67}$ $= 183 \text{ kips} > 104 \text{ kips} \quad \text{o.k.}$

EXAMPLE 9.3—Cross-Connection with Rectangular HSS

Given:

Determine the adequacy of the welded HSS connection illustrated in Figure 9-17 subject to the loads indicated. The connection behaves as a planar cross-connection because the punching load is transmitted through the chord member and is equilibrated by branch member forces on the opposite side of the chord. The two branches on one side of the chord, having a gap at the branch toes of $g = 0.960$ in. (just sufficient for welding), can be considered to have a composite effect on the chord. Assume that the welds are adequately designed and that the connection is not near an open chord end. All HSS members are ASTM A1085/A1085M.

From the AISC *Manual* Table 2-4, the material properties are as follows:

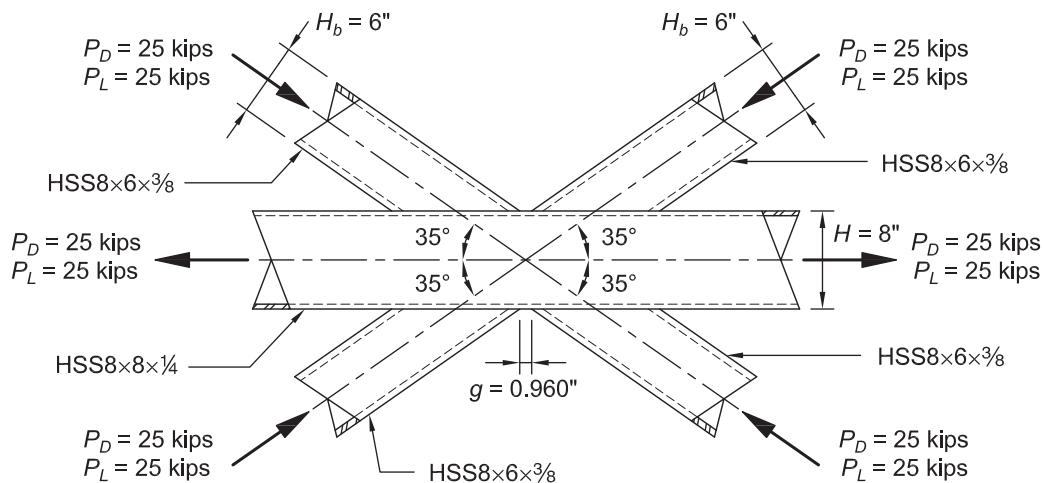


Fig. 9-17. Welded cross-connection with rectangular HSS for Example 9.3.

All members
ASTM A1085/A1085M
 $F_y = 50$ ksi
 $F_u = 65$ ksi

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the geometric properties are as follows:

HSS8×8×¼
 $B = 8.00$ in.
 $H = 8.00$ in.
 $t = 0.250$ in.
 $A = 7.59$ in.²

HSS8×6×⅜
 $B_b = 8.00$ in.
 $H_b = 6.00$ in.
 $t_b = 0.375$ in.
 $A_b = 9.58$ in.²

Solution:

Because the punching load on one side of the chord member is transmitted through the chord member and is equilibrated by branch member forces on the opposite side of the chord member, this is classified as a cross-connection. This type of connection is not explicitly covered by AISC *Specification* Table K3.2 and must be inferred from the *Specification* Chapter J. However, Table 9-2 of this Design Guide summarizes the seven potential limit states for rectangular HSS-to-HSS cross-connections. Limit State 1 (chord plastification) does not apply because this is a matched-width connection and $\beta = B_b/B = 1.0$ (see Table 9-2A). Limit State 2 (chord shear yielding or punching shear) does not apply because $B_b > (B - 2t)$ (see Table 9-2A). Limit State 5 (local crippling of the chord sidewalls) does not apply because this is not a T- or Y-connection (see Table 9-2A). Limit State 7 (shear of the chord sidewalls in gap region) does not apply because, even though there is a vertical plane through the center of the connection that could serve as a shear plane, there is no net shearing force on the other side of this plane.

Limits of applicability

From Table 9-2A for a cross-connection subject to Limit State 3 (local yielding of the branches due to uneven load distribution), Limit State 4 (local yielding of the chord sidewalls), and Limit State 6 (buckling of the chord sidewalls), the limits of applicability are:

Chord wall slenderness:

$$\begin{aligned} B/t &= H/t \leq 35 \\ &= 8.00 \text{ in.}/0.250 \text{ in.} \leq 35 \\ &= 32.0 < 35 \quad \text{o.k.} \end{aligned}$$

Branch wall slenderness:

$$\begin{aligned} B_b/t_b &= 8.00 \text{ in.}/0.375 \leq 35 \\ &= 21.3 < 35 \quad \text{o.k.} \end{aligned}$$

$$\begin{aligned} H_b/t_b &= 6.00 \text{ in.}/0.375 \leq 35 \\ &= 16.0 < 35 \quad \text{o.k.} \end{aligned}$$

$$\begin{aligned} 1.25 \sqrt{\frac{E}{F_{yb}}} &= 1.25 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 30.1 \end{aligned}$$

$$B_b/t_b = 21.3 < 30.1 \quad \text{o.k.}$$

$$H_b/t_b = 16.0 < 30.1 \quad \text{o.k.}$$

Width ratio:

$$B_b/B = 8.00 \text{ in.}/8.00 \text{ in.} \geq 0.25 \\ = 1.00 > 0.25 \quad \mathbf{o.k.}$$

$$H_b/B = 6.00 \text{ in.}/8.00 \text{ in.} \geq 0.25 \\ = 0.750 > 0.25 \quad \mathbf{o.k.}$$

Aspect ratio for branches:

$$H_b/B_b = 6.00 \text{ in.}/8.00 \text{ in.} \\ = 0.750$$

$$0.5 \leq 0.750 \leq 2.0 \quad \mathbf{o.k.}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

$$F_{yb} = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$F_y/F_u = 50 \text{ ksi}/65 \text{ ksi} \\ = 0.769 < 0.8 \quad \mathbf{o.k.}$$

$$F_{yb}/F_{ub} = 50 \text{ ksi}/65 \text{ ksi} \\ = 0.769 < 0.8 \quad \mathbf{o.k.}$$

Required strength (expressed as a force in one branch)

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(25 \text{ kips}) + 1.6(25 \text{ kips})$ $= 70.0 \text{ kips}$	$P_a = P_D + P_L$ $= 25 \text{ kips} + 25 \text{ kips}$ $= 50.0 \text{ kips}$

Local yielding of the branch(es) due to uneven load distribution

From Table 9-2 (Limit State 3), the nominal strength is:

$$P_n = F_{yb}t_b(2H_b + 2B_e - 4t_b) \quad (2-15)$$

where B_e is the effective width of the branch face transverse to the chord; it will be conservative to assume this reduced effective width for all branch transverse walls, B_b , even though the inner transverse wall may be stiffened by the neighboring branch.

$$B_e = \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b \quad (\text{Spec. Eq. K1-1})$$

$$= \left[\frac{10}{(8.00 \text{ in.}/0.250 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.250 \text{ in.})}{(50 \text{ ksi})(0.375 \text{ in.})} \right] (8.00 \text{ in.}) < 8.00 \text{ in.}$$

$$= 1.67 \text{ in.} < 8.00 \text{ in.}$$

$$= 1.67 \text{ in.}$$

Therefore

$$P_n = (50 \text{ ksi})(0.375 \text{ in.})[2(6.00 \text{ in.}) + 2(1.67 \text{ in.}) - 4(0.375 \text{ in.})]$$

$$= 260 \text{ kips/branch}$$

The available strength for the branch local yielding limit state is:

LRFD	ASD
$\phi = 0.95$ $\phi P_n = 0.95(260 \text{ kips})$ $= 247 \text{ kips} > 70.0 \text{ kips} \quad \text{o.k.}$	$\Omega = 1.58$ $\frac{P_n}{\Omega} = \frac{260 \text{ kips}}{1.58}$ $= 165 \text{ kips} > 50.0 \text{ kips} \quad \text{o.k.}$

Local yielding of the chord sidewalls

From Table 9-2 (Limit State 4), the nominal strength is:

$$P_n \sin \theta = 2F_y t (7.5t + l_b) \quad (\text{from Eq. 2-17})$$

where l_b is the combined bearing length on the chord beneath two branches:

$$l_b = 2(H_b / \sin \theta) + g$$

$$= 2\left(\frac{6.00 \text{ in.}}{\sin 35^\circ}\right) + 0.960 \text{ in.}$$

$$= 21.9 \text{ in.}$$

Therefore,

$$P_n \sin \theta = 2(50 \text{ ksi})(0.250 \text{ in.})[7.5(0.250 \text{ in.}) + 21.9 \text{ in.}]$$

$$= 594 \text{ kips}$$

$$P_n = \frac{594 \text{ kips}}{2(\sin 35^\circ)}$$

$$= 518 \text{ kips/branch}$$

The available strength for the chord sidewall yielding limit state is:

LRFD	ASD
$\phi = 1.00$ $\phi P_n = 1.00(518 \text{ kips})$ $= 518 \text{ kips} > 70.0 \text{ kips} \quad \text{o.k.}$	$\Omega = 1.50$ $\frac{P_n}{\Omega} = \frac{518 \text{ kips}}{1.50}$ $= 345 \text{ kips} > 50.0 \text{ kips} \quad \text{o.k.}$

Buckling of the chord sidewalls

As for the limit state of local yielding of the chord sidewalls, the total bearing length is $l_b = 21.9 \text{ in.}$ In Table 9-2A, for Limit State 6 it is noted that Equation J10-8 and Equation 2-23, which are listed in Table 9-2, only apply to bearing lengths less than the chord depth, H . For this connection, $l_b > H = 8.00 \text{ in.}$ Thus, the “column buckling” procedure given in Section 2.8 applies.

$$\begin{aligned}
\frac{KL}{r} &= 3.46 \left(\frac{H}{t} - 3 \right) \sqrt{\frac{1}{\sin \theta}} \\
&= 3.46 \left(\frac{8.00 \text{ in.}}{0.250 \text{ in.}} - 3 \right) \sqrt{\frac{1}{\sin 90^\circ}} \\
&= 100
\end{aligned} \tag{2-22}$$

The symmetry of the loading resembles a 90° cross-connection, so θ is taken as 90° for the wall slenderness calculation because the buckled wall panel will be rectangular, not rhomboidal. The “column” cross-sectional area (for one sidewall) is:

$$\begin{aligned}
A_g &= t(7.5t + l_b) \\
&= (0.250 \text{ in.})[7.5(0.250 \text{ in.}) + 21.9 \text{ in.}] \\
&= 5.94 \text{ in.}^2
\end{aligned}$$

$$\begin{aligned}
4.71 \sqrt{\frac{E}{F_y}} &= 4.71 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\
&= 113
\end{aligned}$$

$$\begin{aligned}
\frac{L_c}{r} &= \frac{KL}{r} \\
&= 100 < 113
\end{aligned}$$

$$\begin{aligned}
F_e &= \frac{\pi^2 E}{\left(\frac{L_c}{r} \right)^2} && (\text{Spec. Eq. E3-4}) \\
&= \frac{\pi^2 (29,000 \text{ ksi})}{(100)^2} \\
&= 28.6 \text{ ksi}
\end{aligned}$$

$$\begin{aligned}
F_{cr} &= \left(0.658^{\frac{F_y}{F_e}} \right) F_y && (\text{Spec. Eq. E3-2}) \\
&= \left(0.658^{\frac{50 \text{ ksi}}{28.6 \text{ ksi}}} \right) (50 \text{ ksi}) \\
&= 24.1 \text{ ksi}
\end{aligned}$$

Alternatively, from the AISC *Manual* Table 4-14, $\phi_c F_{cr} = 21.7 \text{ ksi}$ and $F_{cr}/\Omega_c = 14.4$, for $L_c/r = 100$.

$$\begin{aligned}
P_n &= F_{cr} A_g && (\text{Spec. Eq. E3-1}) \\
&= (24.1 \text{ ksi})(5.94 \text{ in.}^2) \\
&= 143 \text{ kips/sidewall}
\end{aligned}$$

This needs to be multiplied by the chord-stress interaction parameter, Q_f (see Section 2.8). However, $Q_f = 1.0$ for a chord connecting surface in tension, in accordance with the Functions part of Table 9-2. Therefore, $P_n = 286 \text{ kips}$ for two sidewalls. This vertical force can be equated to the sum of two vertical force components from two branches. Expressing the nominal strength as a force per branch then gives:

$$\begin{aligned}
P_n &= \frac{286 \text{ kips}}{2(\sin 35^\circ)} \\
&= 249 \text{ kips/branch}
\end{aligned}$$

The available strength for the chord sidewall buckling limit state is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(249 \text{ kips})$ $= 224 \text{ kips} > 70.0 \text{ kips} \quad \text{o.k.}$	$\Omega_c = 1.67$ $\frac{P_n}{\Omega_c} = \frac{249 \text{ kips}}{1.67}$ $= 149 \text{ kips} > 50.0 \text{ kips} \quad \text{o.k.}$

EXAMPLE 9.4—Overlapped K-Connection with Rectangular HSS

Given:

A planar roof truss contains the welded rectangular HSS 60° overlapped K-connection shown in Figure 9-18. Note that the chord moment is necessary for equilibrium because of the nodding (work point) eccentricity. The connection is a balanced K-connection because the vertical component of the compression branch member force is equilibrated (within 20%) by the vertical component of the tension branch member force [see AISC *Specification* Section K3(b) and Section 9.3 of this Design Guide]. The through (overlapped) branch is the wider and thicker branch member. For fabrication the compression (through) branch is fully welded (overlapped/hidden toe included) to the chord, the diagonal (overlapping) branch is then tacked into place, and finally the whole connection is welded together. All HSS members are ASTM A500/A500M Grade C. The loads shown consist of live load and dead load in the ratio of 3:1. Determine the adequacy of the connection under the given loads and the required weld throat, for each of the branches, using the weld effective length approach in AISC *Specification* Table K5.1. Assume matched electrodes with a specified ultimate strength of $F_{EXX} = 70 \text{ ksi}$.

From the AISC *Manual* Table 2-4, the material properties are as follows:

All members
 ASTM A500/A500M Grade C
 $F_y = 50 \text{ ksi}$
 $F_u = 62 \text{ ksi}$

From AISC *Manual* Tables 1-11 and 1-12, the HSS geometric properties are as follows:

Chord
 HSS8×8×½
 $H = 8.00 \text{ in.}$
 $B = 8.00 \text{ in.}$
 $t = 0.465 \text{ in.}$
 $A = 13.5 \text{ in.}^2$

 HSS6×4×⅝
 $H_{bj} = 6.00 \text{ in.}$
 $B_{bj} = 4.00 \text{ in.}$
 $t_{bj} = 0.291 \text{ in.}$
 $A_{bj} = 5.26 \text{ in.}^2$

 HSS5×3×¼
 $H_{bi} = 5.00 \text{ in.}$
 $B_{bi} = 3.00 \text{ in.}$
 $t_{bi} = 0.233 \text{ in.}$
 $A_{bi} = 3.37 \text{ in.}^2$

Solution:

From AISC *Specification* Table K3.2 and Table 9-2, there is only one limit state for rectangular HSS-to-HSS overlap K-connections—local yielding of the branch/branches due to uneven load distribution.

Limits of applicability

From Table 9-2A, for an overlapped K-connection (subject to Limit State 12), the limits of applicability are:

Connection eccentricity:

Note that the noding eccentricity, e , is negative because the branch centerlines intersect toward the inside of the truss, relative to the chord centerline.

$$\begin{aligned} e/H &= -1 \text{ in.}/8.00 \text{ in.} \\ &= -0.125 \end{aligned}$$

$$-0.55 \leq -0.125 \leq 0.25 \quad \text{o.k.}$$

Chord wall slenderness:

$$\begin{aligned} B/t &= 8.00 \text{ in.}/0.465 \text{ in.} \leq 30 \\ &= 17.2 < 30 \quad \text{o.k.} \end{aligned}$$

$$\begin{aligned} H/t &= 8.00 \text{ in.}/0.465 \text{ in.} \leq 35 \\ &= 17.2 < 35 \quad \text{o.k.} \end{aligned}$$

Wall slenderness for tension branch:

$$\begin{aligned} B_{bi}/t_{bi} &= 3.00 \text{ in.}/0.233 \text{ in.} \leq 35 \\ &= 12.9 < 35 \quad \text{o.k.} \end{aligned}$$

$$\begin{aligned} H_{bi}/t_{bi} &= 5.00 \text{ in.}/0.233 \text{ in.} \leq 35 \\ &= 21.5 < 35 \quad \text{o.k.} \end{aligned}$$

Wall slenderness for compression branch:

$$\begin{aligned} 1.1 \sqrt{\frac{E}{F_{yb}}} &= 1.1 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 26.5 \end{aligned}$$

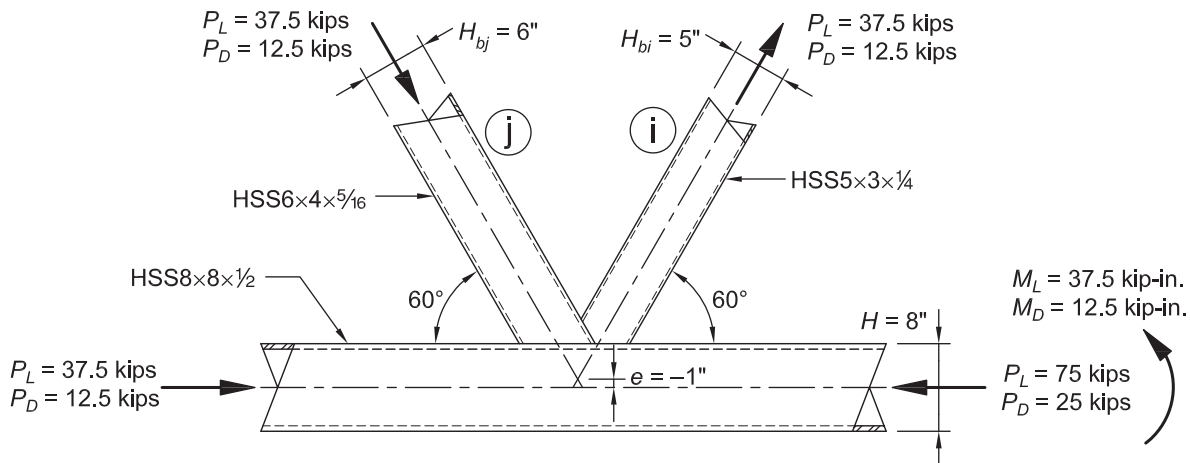


Fig. 9-18. Welded overlap K-connection with rectangular HSS for Example 9.4.

$$B_{bj}/t_{bj} = 4.00 \text{ in.}/0.291 \text{ in.}$$

$$= 13.7 < 26.5 \quad \text{o.k.}$$

$$H_{bj}/t_{bj} = 6.00 \text{ in.}/0.291 \text{ in.}$$

$$= 20.6 < 26.5 \quad \text{o.k.}$$

Width ratio for tension branch:

$$B_{bi}/B = 3.00 \text{ in.}/8.00 \text{ in.} \geq 0.25$$

$$= 0.375 > 0.25 \quad \text{o.k.}$$

$$H_{bi}/B = 5.00 \text{ in.}/8.00 \text{ in.} \geq 0.25$$

$$= 0.625 > 0.25 \quad \text{o.k.}$$

Width ratio for compression branch:

$$B_{bj}/B = 4.00 \text{ in.}/8.00 \text{ in.} \geq 0.25$$

$$= 0.500 > 0.25 \quad \text{o.k.}$$

$$H_{bj}/B = 6.00 \text{ in.}/8.00 \text{ in.} \geq 0.25$$

$$= 0.750 > 0.25 \quad \text{o.k.}$$

Aspect ratio for chord:

$$H/B = 8.00 \text{ in.}/8.00 \text{ in.}$$

$$= 1.00$$

$$0.5 \leq 1.00 \leq 2.0 \quad \text{o.k.}$$

Aspect ratio for tension branch:

$$H_{bi}/B_{bi} = 5.00 \text{ in.}/3.00 \text{ in.}$$

$$= 1.67$$

$$0.5 \leq 1.67 \leq 2.0 \quad \text{o.k.}$$

Aspect ratio for compression branch:

$$H_{bi}/B_{bi} = 6.00 \text{ in.}/4.00 \text{ in.}$$

$$= 1.50$$

$$0.5 \leq 1.50 \leq 2.0 \quad \text{o.k.}$$

Overlap:

Overlap length measured along the connecting face of the chord beneath the two branches:

$$l_{ov} = -x \quad \text{(from Eq. 9-1)}$$

$$= (A + B) - \left(\frac{e + E}{C} \right)$$

$$= \left(\frac{H_{b1}}{2 \sin \theta_{b1}} + \frac{H_{b2}}{2 \sin \theta_{b2}} \right) - \left[\frac{e + H/2}{\sin(\theta_{b1} + \theta_{b2})} \right]$$

$$= \left[\frac{6.00 \text{ in.}}{2(\sin 60^\circ)} + \frac{5.00 \text{ in.}}{2(\sin 60^\circ)} \right] - \left[\frac{-1 \text{ in.} + 4.00 \text{ in.}}{\frac{(\sin 60^\circ)^2}{\sin 120^\circ}} \right]$$

$$= 2.89 \text{ in.}$$

Projected length of the overlapping branch on the chord:

$$\begin{aligned} l_p &= H_{bi} / \sin \theta_{bi} \\ &= 5.00 \text{ in.} / \sin 60^\circ \\ &= 5.77 \text{ in.} \end{aligned}$$

$$\begin{aligned} O_v &= (l_{ov} / l_p) \times 100\% \\ &= \frac{2.89 \text{ in.}}{5.77 \text{ in.}} \times 100\% \\ &= 50.1\% \end{aligned}$$

$$25\% \leq 50.1\% \leq 100\% \quad \mathbf{o.k.}$$

Branch size ratio:

$$\begin{aligned} B_{bi} / B_{bj} &= 3.00 \text{ in.} / 4.00 \text{ in.} \geq 0.75 \\ &= 0.750 \geq 0.75 \quad \mathbf{o.k.} \end{aligned}$$

Branch thickness ratio:

$$\begin{aligned} t_{bi} / t_{bj} &= 0.233 \text{ in.} / 0.291 \text{ in.} \leq 1.0 \\ &= 0.801 < 1.0 \quad \mathbf{o.k.} \end{aligned}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

$$F_{yb} = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$\begin{aligned} F_y / F_u &= 50 \text{ ksi} / 62 \text{ ksi} \\ &= 0.806 > 0.8 \end{aligned}$$

$$\begin{aligned} F_{yb} / F_{ub} &= 50 \text{ ksi} / 62 \text{ ksi} \\ &= 0.806 > 0.8 \end{aligned}$$

However, the note in Table 9-2A indicates ASTM A500/A500M Grade C is acceptable. **o.k.**

Required strength (expressed as a force in a branch)

From ASCE/SEI 7, Chapter 2, the required strength for the compression and tension branch is:

LRFD	ASD
$\begin{aligned} P_u &= 1.2P_D + 1.6P_L \\ &= 1.2(12.5 \text{ kips}) + 1.6(37.5 \text{ kips}) \\ &= 75.0 \text{ kips} \end{aligned}$	$\begin{aligned} P_a &= P_D + P_L \\ &= 12.5 \text{ kips} + 37.5 \text{ kips} \\ &= 50.0 \text{ kips} \end{aligned}$

Local yielding of the branch(es) due to uneven load distribution

From AISC *Specification* Table K3.2 and Table 9-2 (Limit State 12), the nominal strength expressed as a force in the overlapping branch, *i*, for $50\% \leq O_v < 80\%$ is:

$$P_{n,i} = F_{ybi} t_{bi} (2H_{bi} - 4t_{bi} + B_{eoi} + B_{eov}) \quad (\text{Spec. Eq. K3-11})$$

where B_{eoi} is the effective width of the overlapping branch member transverse wall landing on the chord:

$$\begin{aligned}
 B_{eoi} &= \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{ybi} t_{bi}} \right) B_{bi} \leq B_{bi} & (\text{Spec. Eq. K3-13}) \\
 &= \left[\frac{10}{(8.00 \text{ in.}/0.465 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.465 \text{ in.})}{(50 \text{ ksi})(0.233 \text{ in.})} \right] (3.00 \text{ in.}) \leq 3.00 \text{ in.} \\
 &= 3.48 \text{ in.} > 3.00 \text{ in.} \\
 &= 3.00 \text{ in.}
 \end{aligned}$$

and B_{eov} is the effective width of the overlapping branch member transverse wall landing on the overlapped branch:

$$\begin{aligned}
 B_{eov} &= \left(\frac{10}{B_{bj}/t_{bj}} \right) \left(\frac{F_{ybj} t_{bj}}{F_{ybi} t_{bi}} \right) B_{bi} \leq B_{bi} & (\text{Spec. Eq. K3-14}) \\
 &= \left[\frac{10}{(4.00 \text{ in.}/0.291 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.291 \text{ in.})}{(50 \text{ ksi})(0.233 \text{ in.})} \right] (3.00 \text{ in.}) \leq 3.00 \text{ in.} \\
 &= 2.73 \text{ in.} < 3.00 \text{ in.} \\
 &= 2.73 \text{ in.}
 \end{aligned}$$

Therefore, the nominal strength of the overlapping branch is:

$$\begin{aligned}
 P_{n,i} &= (50 \text{ ksi})(0.233 \text{ in.}) [2(5.00 \text{ in.}) - 4(0.233 \text{ in.}) + 3.00 \text{ in.} + 2.73 \text{ in.}] \\
 &= 172 \text{ kips}
 \end{aligned}$$

The nominal strength of the overlapped branch is then:

$$\begin{aligned}
 P_{n,j} &= P_{n,i} \left(\frac{F_{ybj} A_{bj}}{F_{ybi} A_{bi}} \right) & (\text{Spec. Eq. K3-15}) \\
 &= (172 \text{ kips}) \left[\frac{(50 \text{ ksi})(5.26 \text{ in.}^2)}{(50 \text{ ksi})(3.37 \text{ in.}^2)} \right] \\
 &= 268 \text{ kips}
 \end{aligned}$$

The available strength for the branch local yielding limit state, expressed as forces in the tension (overlapping) and compression (overlapped) branches, is:

LRFD	ASD
$\phi = 0.95$ For the tension (overlapping) branch: $\phi P_{n,i} = 0.95(172 \text{ kips})$ $= 163 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$ For the compression (overlapped) branch: $\phi P_{n,j} = 0.95(268 \text{ kips})$ $= 255 \text{ kips} > 75.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.58$ For the tension (overlapping) branch: $\frac{P_{n,i}}{\Omega} = \frac{172 \text{ kips}}{1.58}$ $= 109 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$ For the compression (overlapped) branch: $\frac{P_{n,j}}{\Omega} = \frac{268 \text{ kips}}{1.58}$ $= 170 \text{ kips} > 50.0 \text{ kips} \quad \mathbf{o.k.}$

Determine the required weld throat

A continuous weld effective throat will be provided for each branch. Both branches will be welded around the entire perimeter, including the hidden toe of the overlapped (through) branch.

From AISC *Specification* Table K5.1, the overlapping member weld effective length for $50\% \leq O_v < 80\%$ is:

$$l_{e,i} = 2 \left\{ \left(1 - \frac{O_v}{100} \right) \left(\frac{H_{bi}}{\sin \theta_i} \right) + \frac{O_v}{100} \left[\frac{H_{bi}}{\sin(\theta_i + \theta_j)} \right] \right\} + B_{eoi} + B_{eov} \quad (\text{Spec. Eq. K5-11})$$

where $B_{eoi} = 3.00$ in., as determined earlier.

However, AISC *Specification* Table K5.1 also states that when $B_{bi}/B > 0.85$ or $\theta_i > 50^\circ$ (which is the case in this example because $\theta_i = 60^\circ$), $B_{eoi}/2$ should not exceed $B_{bi}/4$. Thus, $B_{eoi} = 1.50$ in. $B_{eov} = 2.73$ in. as calculated earlier, but AISC *Specification* Table K5.1 also states that when $B_{bi}/B_{bj} > 0.85$ or $(180 - \theta_i - \theta_j) > 50^\circ$ (which is the case in this example because $\theta_i = \theta_j = 60^\circ$), $B_{eov}/2$ should not exceed $B_{bi}/4$. Thus, $B_{eov} = 1.50$ in. also.

Therefore, the effective length of the weld to the overlapping branch, i , is:

$$\begin{aligned} l_{e,i} &= 2 \left[\left(1 - \frac{50.1}{100} \right) \left(\frac{5.00 \text{ in.}}{\sin 60^\circ} \right) + \frac{50.1}{100} \left(\frac{5.00 \text{ in.}}{\sin 120^\circ} \right) \right] + 1.50 \text{ in.} + 1.50 \text{ in.} \\ &= 14.6 \text{ in.} \end{aligned}$$

The effective length of the weld to the overlapped (through) branch, j , is:

$$l_{e,j} = \frac{2H_{bj}}{\sin \theta_j} + 2B_{ej} \quad (\text{Spec. Eq. K5-13})$$

where B_{ej} is given in this case by Equation 3-11 and AISC *Specification* Equation K5-14 because both transverse welds are attached to the chord. However, AISC *Specification* Table K5.1 also states that when $B_{bj}/B > 0.85$ or $\theta_j > 50^\circ$ (which is the case in this example), the transverse welds are neglected and $l_{e,j}$ is given by:

$$\begin{aligned} l_{e,j} &= 2(H_{bj} - 1.2t_{bj}) / \sin \theta_j \quad (\text{Spec. Eq. K5-15}) \\ &= \frac{2[6.00 \text{ in.} - 1.2(0.291 \text{ in.})]}{\sin 60^\circ} \\ &= 13.0 \text{ in.} \end{aligned}$$

Note that a weld would be provided across the full widths of the overlapped branch, B_{bj} , at the toe and the heel locations, transverse to the chord, even though these transverse welds are not considered to be at all effective.

The required weld throat, assuming fillet welds all around, for the tension (overlapping) branch is as follows. This assumes that the flat width of the overlapped branch is adequate for fillet welding.

$$\begin{aligned} R_n &= F_{nw} t_w l_e \quad (\text{Spec. Eq. K5-1}) \\ &= 0.60(70 \text{ ksi})(t_w)(14.6 \text{ in.}) \\ &= (613 \text{ kips/in.}) t_w \end{aligned}$$

LRFD	ASD
$\phi_w = 0.75$ (Spec. Table J2.5) $\phi_w R_n \geq P_u$ $0.75(613 \text{ kips/in.})t_w \geq 75.0 \text{ kips}$ $t_w \geq 0.163 \text{ in.}$	$\Omega_w = 2.00$ (Spec. Table J2.5) $\frac{R_n}{\Omega_w} \geq P_a$ $\frac{(613 \text{ kips/in.})t_w}{2.00} \geq 50.0 \text{ kips}$ $t_w \geq 0.163 \text{ in.}$

The required weld throat, assuming fillet welds all around, for the compression (overlapped/through) branch is as follows. In this case the fillet weld directional strength increase factor may be used because the member is in axial compression (see Section 3.1.1); however, this is not utilized because of local secondary bending moments in the overlapped region of the connection.

$$R_n = F_{nw} t_w l_e \quad (\text{Spec. Eq. K5-1})$$

$$= 0.60(70 \text{ ksi})(t_w)(13.0 \text{ in.})$$

$$= (546 \text{ kips/in.})t_w$$

LRFD	ASD
$\phi_w = 0.75$ (Spec. Table J2.5) $\phi_w R_n \geq P_u$ $0.75(546 \text{ kips/in.})t_w \geq 75.0 \text{ kips}$ $t_w \geq 0.183 \text{ in.}$	$\Omega_w = 2.00$ (Spec. Table J2.5) $\frac{R_n}{\Omega_w} \geq P_a$ $\frac{(546 \text{ kips/in.})t_w}{2.00} \geq 50.0 \text{ kips}$ $t_w \geq 0.183 \text{ in.}$

These weld calculations are repeated in Example 3.4 of Chapter 3, except the ASTM A500/A500M Grade C HSS material in this example is replaced by ASTM A1085/A1085M material in Example 3.4. Note that the same required weld throats are required, despite differences in HSS design wall thickness.

EXAMPLE 9.5—Gapped K-Connection with Square HSS and Unbalanced Branch Loads

Given:

A planar roof truss contains the welded rectangular HSS 45° gapped K-connection shown in Figure 9-19. Note that the chord moment is necessary for equilibrium because of the nodding (work point) eccentricity. Because the vertical components of the branch member forces differ by more than 20%, the connection must be treated as a combination of two types—a K-connection and a cross-connection [see AISC *Specification* Section K3(b) and Section 9.3 of this Guide], as will be demonstrated. All HSS members are ASTM A1085/A1085M. The loads shown consist of live load and dead load in the ratio 3:1. Determine the adequacy of the HSS connection and also consider the design of the welds.

From the AISC *Manual* Table 2-4, the material properties are as follows:

All members

ASTM A1085/A1085M

$F_y = 50 \text{ ksi}$

$F_u = 65 \text{ ksi}$

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the HSS geometric properties are as follows:

Chord
HSS12×12×½
 $B = 12.0$ in.
 $H = 12.0$ in.
 $t = 0.500$ in.
 $A = 22.4$ in.²

Branches
HSS8×8×¾
 $B_b = 8.00$ in.
 $H_b = 8.00$ in.
 $t_b = 0.375$ in.
 $A_b = 11.1$ in.²

Solution:

The total applied loads on each branch are:

$$\begin{aligned} P_{\text{compression branch}} &= P_D + P_L \\ &= 40 \text{ kips} + 120 \text{ kips} \\ &= 160 \text{ kips} \end{aligned}$$

$$\begin{aligned} P_{\text{tension branch}} &= P_D + P_L \\ &= 21.2 \text{ kips} + 63.4 \text{ kips} \\ &= 84.6 \text{ kips} \end{aligned}$$

From AISC *Specification* Section K3(b) and Section 9.3 of this Design Guide, when the punching load, $P_r \sin \theta$, in a branch member is essentially equilibrated (within 20%) by loads in other branch member(s) on the same side of the connection, the connection is classified as a K-connection. The relevant gap is between the primary branch members whose loads equilibrate. When the punching load, $P_r \sin \theta$, is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection is classified as a cross-connection. When branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the adequacy of each branch is determined by linear interaction of the proportion of the branch load involved in each type of load transfer.

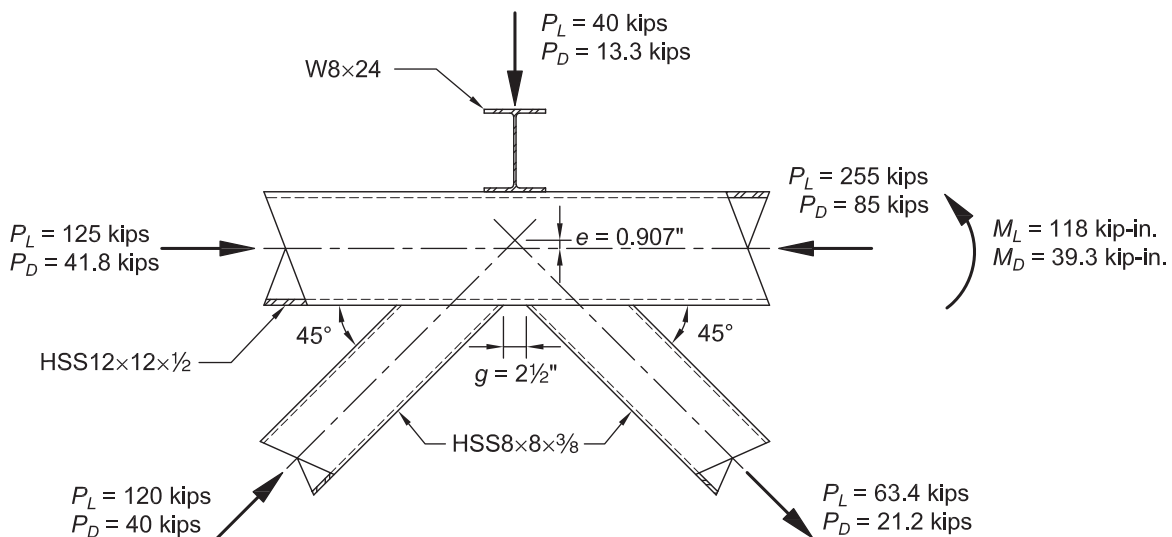


Fig. 9-19. Welded gap K-connection with square HSS for Example 9.5.

Connection classification

Based on the previous discussion, the classification of a connection as a K-connection is dependent on the following inequality:

$$0.8 \leq \frac{P_{\text{compression branch}} \sin \theta_{\text{compression branch}}}{P_{\text{tension branch}} \sin \theta_{\text{tension branch}}} \leq 1.2$$

$$\frac{P_{\text{compression branch}} \sin \theta_{\text{compression branch}}}{P_{\text{tension branch}} \sin \theta_{\text{tension branch}}} = \frac{(160 \text{ kips})(\sin 45^\circ)}{(84.6 \text{ kips})(\sin 45^\circ)} = 1.89 > 1.2$$

Therefore, treat the connection as a combination of a K-connection and a cross-connection, as shown in Figure 9-20.

Analysis of balanced K-connection [see Figure 9-20(a)]

From AISC *Specification* Table K3.2 and Table 9-2, there are four potential limit states for rectangular HSS-to-HSS gapped K-connections. Limit State 9 (chord shear yielding or punching shear) does not need to be checked because the branches are square, as noted in Table K3.2 and Table 9-2A. Limit State 10 (local yielding of the branch/branches due to uneven load distribution) does not need to be checked because the branches are square, as noted in Table K3.2 and Table 9-2A. Limit State 11 (shear of the chord sidewalls in gap region) also does not need to be checked because the chord is square, as noted in Table K3.2 and Table 9-2A. Thus, only the limit state of chord plastification (Limit State 8 in Table 9-2) is applicable.

Limits of applicability

From Table 9-2A, for a gapped K-connection (subject to Limit State 8), the limits of applicability are:

Connection eccentricity:

The connection is detailed with a gap, $g = 2\frac{1}{2}$ in., between the toes of the branch members, excluding any weld dimensions. From this, the connection nodding eccentricity, e , can be obtained:

$$e = \frac{\sin \theta_{b1} \sin \theta_{b2}}{\sin(\theta_{b1} + \theta_{b2})} \left(\frac{H_{b1}}{2 \sin \theta_{b1}} + \frac{H_{b2}}{2 \sin \theta_{b2}} + g \right) - \frac{H}{2} \quad (\text{from Eq. 9-2})$$

$$= \frac{(\sin 45^\circ)^2}{\sin 90^\circ} \left(\frac{8.00 \text{ in.}}{2 \sin 45^\circ} + \frac{8.00 \text{ in.}}{2 \sin 45^\circ} + 2\frac{1}{2} \text{ in.} \right) - \frac{12.0 \text{ in.}}{2}$$

$$= 0.907 \text{ in.}$$

The eccentricity, $e = 0.907$ in., is also shown on Figure 9-19. A positive value of e indicates that the branch centerlines intersect toward the outside of the truss relative to the chord centerline.

$$\frac{e}{H} = 0.907 \text{ in.} / 12.0 \text{ in.}$$

$$= 0.0756$$

$$-0.55 \leq 0.0756 \leq 0.25 \quad \text{o.k.}$$

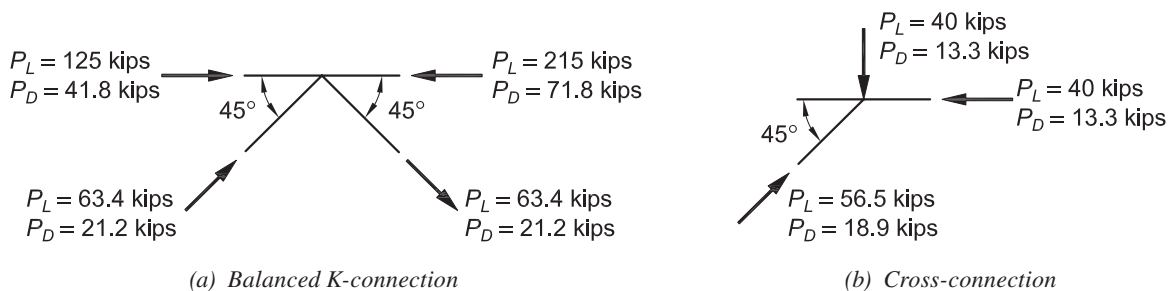


Fig. 9-20. Loads for balanced K-connection and cross-connection.

Because the nodding eccentricity satisfies this limit, the resulting eccentricity moment that it produces ($M_L + M_D = 118 \text{ kip-in.} + 39.3 \text{ kip-in.} = 157 \text{ kip-in.}$, as shown on Figure 9-19) can be neglected with regard to its effect on connection design. For this reason, it is not considered when calculating U later, when determining the required strength. However, it should be noted that the eccentricity moment is still a primary bending moment (necessary for equilibrium) and should be taken into account for chord member design.

Chord wall slenderness:

$$\begin{aligned} B/t &= H/t \\ &= 12.0 \text{ in.}/0.500 \text{ in.} \leq 35 \\ &= 24.0 < 35 \quad \mathbf{o.k.} \end{aligned}$$

Wall slenderness for tension branch:

$$\begin{aligned} B_b/t_b &= H_b/t_b \\ &= 8.00 \text{ in.}/0.375 \text{ in.} \leq 35 \\ &= 21.3 < 35 \quad \mathbf{o.k.} \end{aligned}$$

Wall slenderness for compression branch:

$$\begin{aligned} B_b/t_b &= H_b/t_b \\ &= 8.00 \text{ in.}/0.375 \text{ in.} \leq 35 \\ &= 21.3 < 35 \quad \mathbf{o.k.} \end{aligned}$$

$$\begin{aligned} 1.25 \sqrt{\frac{E}{F_{yb}}} &= 1.25 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 30.1 \end{aligned}$$

$$\begin{aligned} B_b/t_b &= H_b/t_b \\ &= 21.3 < 30.1 \quad \mathbf{o.k.} \end{aligned}$$

Width ratio:

$$\begin{aligned} \gamma &= B/2t \\ &= \frac{12.0 \text{ in.}}{2(0.500 \text{ in.})} \\ &= 12.0 \end{aligned}$$

$$\begin{aligned} B_b/B &= H_b/B \geq (0.1 + \gamma/50) \\ &= 8.00 \text{ in.}/12.0 \text{ in.} \geq (0.1 + 12.0/50) \\ &= 0.667 > 0.340 \quad \mathbf{o.k.} \end{aligned}$$

$$\begin{aligned} \beta_{eff} &= \frac{(B_b + H_b)_{compression \text{ branch}} + (B_b + H_b)_{tension \text{ branch}}}{4B} \geq 0.35 \\ &= \frac{(8.00 \text{ in.} + 8.00 \text{ in.}) + (8.00 \text{ in.} + 8.00 \text{ in.})}{4(12.0 \text{ in.})} \geq 0.35 \\ &= 0.667 > 0.35 \quad \mathbf{o.k.} \end{aligned}$$

Aspect ratio for chord:

$$\begin{aligned} H/B &= 12.0 \text{ in.}/12.0 \text{ in.} \\ &= 1.00 \\ 0.5 &\leq 1.00 \leq 2.0 \quad \mathbf{o.k.} \end{aligned}$$

Aspect ratio for the branches:

$$H_b/B_b = 8.00 \text{ in.}/8.00 \text{ in.} \\ = 1.00$$

$$0.5 \leq 1.00 \leq 2.0 \quad \mathbf{o.k.}$$

Gap:

$$\zeta = g/B \\ = 2\frac{1}{2} \text{ in.}/12.0 \text{ in.} \\ = 0.208$$

$$0.5(1 - \beta_{eff}) \leq \zeta \leq 1.5(1 - \beta_{eff}) \\ 0.5(1 - 0.667) \leq 0.208 \leq 1.5(1 - 0.667) \\ 0.167 \leq 0.208 \leq 0.500 \quad \mathbf{o.k.}$$

$$g \geq t_{b \text{ compression branch}} + t_{b \text{ tension branch}} \\ \geq 0.375 \text{ in.} + 0.375 \text{ in.} \\ \geq 0.750 \text{ in.}$$

$$2\frac{1}{2} \text{ in.} > 0.750 \text{ in.} \quad \mathbf{o.k.}$$

Branch size ratio:

$$B_{\text{smaller branch}}/B_{\text{larger branch}} = 8.00 \text{ in.}/8.00 \text{ in.} \geq 0.63 \\ = 1.00 > 0.63 \quad \mathbf{o.k.}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

$$F_{yb} = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$F_y/F_u = 50 \text{ ksi}/65 \text{ ksi} \leq 0.8 \\ = 0.769 < 0.8 \quad \mathbf{o.k.}$$

$$F_{yb}/F_{ub} = 50 \text{ ksi}/65 \text{ ksi} \leq 0.8 \\ = 0.769 < 0.8 \quad \mathbf{o.k.}$$

Required strength (expressed as a force in a branch)

From ASCE/SEI 7, Chapter 2, the required strength for the compression branch and the tension branch for the K-connection portion [see Figure 9-20(a)] is:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(21.2 \text{ kips}) + 1.6(63.4 \text{ kips})$ $= 127 \text{ kips}$	$P_a = P_D + P_L$ $= 21.2 \text{ kips} + 63.4 \text{ kips}$ $= 84.6 \text{ kips}$

Chord plastification

From AISC *Specification* Table K3.2 and Table 9-2 (Limit State 8), the nominal strength, expressed as a force in a branch, is:

$$P_n \sin \theta = F_y t^2 (9.8 \beta_{eff} \gamma^{0.5}) Q_f \quad (\text{Spec. Eq. K3-7})$$

where for a chord connecting surface in compression:

$$0.4 \leq Q_f = \left(1.3 - 0.4 \frac{U}{\beta_{eff}} \right) \leq 1.0 \quad (\text{Spec. Eq. K1-5})$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{Spec. Eq. K1-6})$$

From ASCE/SEI 7, Chapter 2, the required strength in the chord, on the side of the connection with the higher compression stress [see Figure 9-20(a)], is:

LRFD	ASD
$P_{ro} = P_u$ $= 1.2P_D + 1.6P_L$ $= 1.2(71.8 \text{ kips}) + 1.6(215 \text{ kips})$ $= 430 \text{ kips}$ $M_{ro} = 0$ $F_c = F_y$ $= 50 \text{ ksi}$ $U = \left \frac{430 \text{ kips}}{(50 \text{ ksi})(22.4 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.384 < 1.0$ $Q_f = 1.3 - 0.4 \left(\frac{0.384}{0.667} \right) \leq 1.0$ $= 1.07 > 1.0$ $= 1.0$	$P_{ro} = P_a$ $= P_D + P_L$ $= 71.8 \text{ kips} + 215 \text{ kips}$ $= 287 \text{ kips}$ $M_{ro} = 0$ $F_c = 0.6F_y$ $= 0.6(50 \text{ ksi})$ $= 30.0 \text{ kips}$ $U = \left \frac{287 \text{ kips}}{(30.0 \text{ ksi})(22.4 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.427 < 1.0$ $Q_f = 1.3 - 0.4 \left(\frac{0.427}{0.667} \right) \leq 1.0$ $= 1.04 > 1.0$ $= 1.0$

Therefore, the available strength for the balanced, gapped K-connection is:

$$P_n = \frac{(50 \text{ ksi})(0.500 \text{ in.})^2 [9.8(0.667)(12.0)^{0.5}](1.0)}{\sin 45^\circ}$$

$$= 400 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi P_n = 0.90(400 \text{ kips})$ $= 360 \text{ kips}$ For both branches, because θ_b is the same, the utilization is: $\frac{P_u}{\phi P_n} = \frac{127 \text{ kips}}{360 \text{ kips}}$ $= 0.353$	$\Omega = 1.67$ $\frac{P_n}{\Omega} = \frac{400 \text{ kips}}{1.67}$ $= 240 \text{ kips}$ For both branches, because θ_b is the same, the utilization is: $\frac{P_a}{P_n/\Omega} = \frac{84.6 \text{ kips}}{240 \text{ kips}}$ $= 0.353$

Analysis of cross-connection [see Figure 9-20(b)]

From Table 9-2, there are seven potential limit states for rectangular HSS-to-HSS cross-connections. Limit State 2 (chord shear yielding or punching shear) does not need to be checked because $B/t = 24.0 \geq 10$, as noted in Table 9-2A. Limit State 3 (local yielding of the branch/branches due to uneven load distribution) does not need to be checked because $\beta = 0.667 < 0.85$, as noted in Table 9-2A. Limit States 4, 5, and 6 do not need to be checked because $\beta < 1.0$. Limit State 7 (shear of the chord sidewalls in gap region) also does not need to be checked because there is no shear plane evident, as noted in Table 9-2A. (The flange of the W8×24 is wider than the connection gap, $g = 2\frac{1}{2}$ in.). Thus, only the limit state of chord plastification (Limit State 1 in Table 9-2) is applicable.

Limits of applicability

From Table 9-2A, the limits of applicability for a cross-connection (subject to Limit State 1) are:

Note that only the compression branch needs to be checked because this is the only branch that participates in the cross-connection force transfer. The applicable limits are on chord wall slenderness, width ratio, material strength, ductility, and end distance (Table 9-2A). Most have been validated previously for the balanced K-connection, except for the following:

Width ratio for compression branch:

$$\begin{aligned}\beta &= B_b/B = H_b/B \geq 0.25 \\ &= 8.00 \text{ in.}/12.0 \text{ in.} \geq 0.25 \\ &= 0.667 > 0.25 \quad \text{o.k.}\end{aligned}$$

End distance:

$$\begin{aligned}\beta &= 0.667 \leq 0.85 \\ l_{end} &\geq B\sqrt{1-\beta} \\ &\geq (12.0 \text{ in.})\sqrt{1-0.667} \\ &\geq 6.92\end{aligned}$$

Thus, the heel of the compression branch must be located a minimum of 6.92 in. from an uncapped HSS end.

Required strength (expressed as a force in the compression branch)

From ASCE/SEI 7, Chapter 2, the required strength for the compression branch of the cross-connection portion [see Figure 9-20(b)] is:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $= 1.2(18.9 \text{ kips}) + 1.6(56.5 \text{ kips})$ $= 113 \text{ kips}$	$P_a = P_D + P_L$ $= 18.9 \text{ kips} + 56.5 \text{ kips}$ $= 75.4 \text{ kips}$

Chord plastification

From Table 9-2 (Limit State 1), the nominal strength, expressed as a force in the branch, is:

$$P_n \sin \theta = F_y t^2 \left[\frac{2\eta}{(1-\beta)} + \frac{4}{\sqrt{1-\beta}} \right] Q_f \quad (2-2)$$

where

$$\begin{aligned}\eta &= \frac{H_b}{B \sin \theta} \\ &= \frac{8.00 \text{ in.}}{(12.0 \text{ in.})(\sin 45^\circ)} \\ &= 0.943\end{aligned}$$

for a chord connecting surface in compression

$$0.4 \leq Q_f = \left(1.3 - 0.4 \frac{U}{\beta} \right) \leq 1.0 \quad (\text{Spec. Eq. K1-4})$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{Spec. Eq. K1-6})$$

From ASCE/SEI 7, Chapter 2, the required strength in the chord, on the side of the connection with the higher compression stress [see Figure 9-20(b)], is:

LRFD	ASD
$\begin{aligned}P_{ro} &= P_u \\ &= 1.2P_D + 1.6P_L \\ &= 1.2(13.3 \text{ kips}) + 1.6(40 \text{ kips}) \\ &= 80.0 \text{ kips} \\ M_{ro} &= 0 \\ F_c &= F_y \\ &= 50 \text{ ksi} \\ U &= \left \frac{80.0 \text{ kips}}{(50 \text{ ksi})(22.4 \text{ in.}^2)} + 0 \right \leq 1.0 \\ &= 0.0714 < 1.0 \\ Q_f &= 1.3 - 0.4 \left(\frac{0.0714}{0.667} \right) \leq 1.0 \\ &= 1.26 > 1.0 \\ &= 1.0\end{aligned}$	$\begin{aligned}P_{ro} &= P_a \\ &= P_D + P_L \\ &= 13.3 \text{ kips} + 40 \text{ kips} \\ &= 53.3 \text{ kips} \\ M_{ro} &= 0 \\ F_c &= 0.6F_y \\ &= 0.6(50 \text{ ksi}) \\ &= 30.0 \text{ ksi} \\ U &= \left \frac{53.3 \text{ kips}}{(30.0 \text{ ksi})(22.4 \text{ in.}^2)} + 0 \right \leq 1.0 \\ &= 0.0793 < 1.0 \\ Q_f &= 1.3 - 0.4 \left(\frac{0.0793}{0.667} \right) \leq 1.0 \\ &= 1.25 > 1.0 \\ &= 1.0\end{aligned}$

Therefore, the available strength for the cross-connection is:

$$\begin{aligned}P_n &= \frac{(50 \text{ ksi})(0.500 \text{ in.})^2 \left[\frac{2(0.943)}{1-0.667} + \frac{4}{\sqrt{1-0.667}} \right] (1.0)}{\sin 45^\circ} \\ &= 223 \text{ kips}\end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi P_n = 1.00(223 \text{ kips})$ $= 223 \text{ kips}$ Compression branch utilization: $\frac{P_u}{\phi P_n} = \frac{113 \text{ kips}}{223 \text{ kips}}$ $= 0.507$	$\Omega = 1.50$ $\frac{P_n}{\Omega} = \frac{223 \text{ kips}}{1.50}$ $= 149 \text{ kips}$ Compression branch utilization: $\frac{P_a}{P_n/\Omega} = \frac{75.4 \text{ kips}}{149 \text{ kips}}$ $= 0.506$

Total utilization of each branch member

For the compression branch:

$$\begin{aligned}
 \text{Utilization} &= \text{balanced K} + \text{cross} \\
 &= 0.353 + 0.507 \\
 &= 0.860 \leq 1.0 \quad \mathbf{o.k.}
 \end{aligned}$$

For the tension branch:

$$\begin{aligned}
 \text{Utilization} &= \text{balanced K} \\
 &= 0.353 \leq 1.0 \quad \mathbf{o.k.}
 \end{aligned}$$

These summations use LRFD values; however, the ASD values are virtually the same.

Weld design considerations

The compression branch, inclined at 45° , behaves partly as a gapped K-connection branch and partly as a cross-connection branch. For $\theta \leq 50^\circ$, all four sides of a gapped K-connection perimeter weld are effective (AISC *Specification* Equation K5-8), whereas less of the perimeter weld around a cross-connection is effective (AISC *Specification* Equation K5-5). A conservative approach would be to treat both branches as cross-connections, and thus the total weld effective length around one branch would be:

$$l_e = \frac{2H_b}{\sin \theta} + 2B_e \quad (\text{Spec. Eq. K5-5})$$

where

$$\begin{aligned}
 B_e &= \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b & (\text{Spec. Eq. K1-1}) \\
 &= \left[\frac{10}{(12.0 \text{ in.}/0.500 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.500 \text{ in.})}{(50 \text{ ksi})(0.375 \text{ in.})} \right] (8.00 \text{ in.}) < 8.00 \text{ in.} \\
 &= 4.44 \text{ in.} < 8.00 \text{ in.} \\
 &= 4.44 \text{ in.}
 \end{aligned}$$

Thus,

$$\begin{aligned}
 l_e &= \frac{2(8.00 \text{ in.})}{\sin 45^\circ} + 2(4.44 \text{ in.}) \\
 &= 31.5 \text{ in.}
 \end{aligned}$$

The weld can then be designed in accordance with procedures given in Chapter 3 of this Design Guide and in AISC Design Guide 21 (Miller, 2017). Alternatively, as noted in Chapter 3 and also in the AISC *Specification* Section K5 Commentary, welds may be proportioned to develop the capacity of the connected branch wall at all points along the weld length. Welds sized in this latter manner represent an upper limit on the required weld size for any loading condition.

EXAMPLE 9.6—Overlapped KT-Connection with Square HSS

Given:

Determine the adequacy of the welded KT-connection shown in Figure 9-21. The branch member force arrangement is similar to Figure 9-9(b), with live load and dead load in a 3:1 ratio. Of the two diagonal members, which are the largest branches and which are also the same size, the branch with the largest force is welded directly to the chord member. The branches labeled 1 and 2 in Figure 9-21 have an overlap, O_v , of 50% at the chord connecting face. Thus, $l_{ov} = 0.50l_p = 0.50(5.00 \text{ in.}/\sin 60^\circ) = 2.89 \text{ in.}$ All HSS members are ASTM A1085/A1085M.

From the AISC *Manual* Table 2-4, the material properties are as follows:

All members
 ASTM A1085/A1085M
 $F_y = 50 \text{ ksi}$
 $F_u = 65 \text{ ksi}$

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the geometric properties are as follows:

Chord
 HSS10×10× $\frac{3}{8}$
 $B = 10.0 \text{ in.}$
 $H = 10.0 \text{ in.}$
 $t = 0.375 \text{ in.}$
 $A = 14.1 \text{ in.}^2$

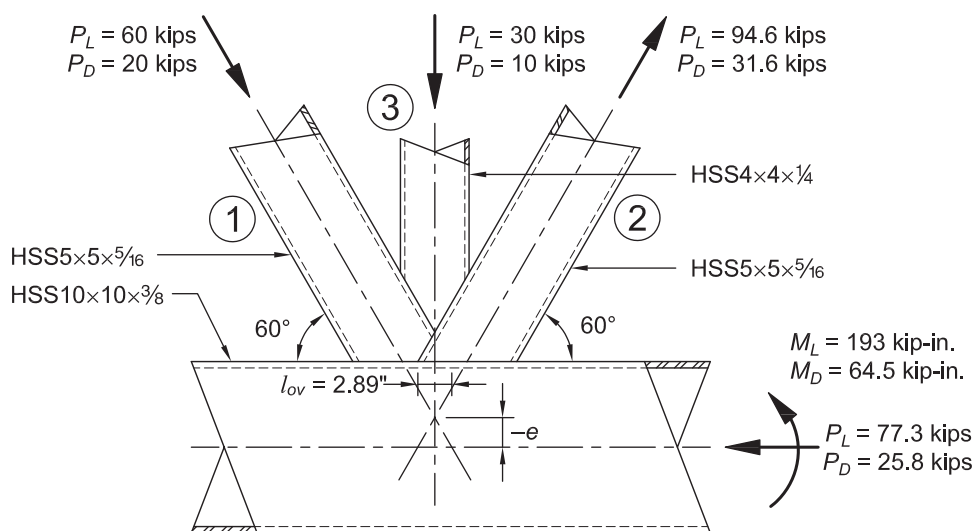


Fig. 9-21. Overlapped KT-connection for Example 9.6.

Branches (diagonals)

HSS5×5×⁵/₁₆

$B_b = 5.00$ in.

$H_b = 5.00$ in.

$t_b = 0.313$ in.

$A_b = 5.62$ in.²

Branch (vertical)

HSS4×4×¹/₄

$B_b = 4.00$ in.

$H_b = 4.00$ in.

$t_b = 0.250$ in.

$A_b = 3.59$ in.²

Solution:

As mentioned in Section 9.6.1, for overlapped KT-connections composed of rectangular and square HSS, the connection may be checked on a branch-by-branch basis in a similar manner to overlapped K-connections, using AISC *Specification* Table K3.2 and Table 9-2 of this Design Guide. These tables confirm that, subject to meeting the relevant Limits of Applicability in Table K3.2A and Table 9-2A, there is only one limit state for rectangular HSS-to-HSS overlap K-connections—local yielding of the branch/branches due to uneven load distribution.

Limits of applicability

From Table 9-2A for an overlapped K-connection (subject to Limit State 12), the limits of applicability are:

Connection eccentricity:

The connection is detailed with an overlap, $l_{ov} = 2.89$ in., between branches 1 and 2. This can be considered a “negative gap”; that is, the gap, $g = -2.89$ in. From this, the connection nodding eccentricity, e , can be obtained:

$$\begin{aligned} e &= \frac{\sin \theta_{b1} \sin \theta_{b2}}{\sin(\theta_{b1} + \theta_{b2})} \left(\frac{H_{b1}}{2 \sin \theta_{b1}} + \frac{H_{b2}}{2 \sin \theta_{b2}} - l_{ov} \right) - \frac{H}{2} && \text{(from Eq. 9-2)} \\ &= \frac{(\sin 60^\circ)^2}{\sin 120^\circ} \left(\frac{5.00 \text{ in.}}{2 \sin 60^\circ} + \frac{5.00 \text{ in.}}{2 \sin 60^\circ} - 2.89 \text{ in.} \right) - \frac{10.0 \text{ in.}}{2} \\ &= -2.50 \text{ in.} \end{aligned}$$

A negative value of e indicates that the branch centerlines intersect toward the inside of the truss, relative to the chord centerline.

$$e/H = -2.50 \text{ in.}/10.0 \text{ in.}$$

$$= -0.250$$

$$-0.55 \leq -0.250 \leq 0.25 \quad \text{o.k.}$$

Chord wall slenderness:

$$B/t = 10.0 \text{ in.}/0.375 \text{ in.} \leq 30$$

$$= 26.7 < 30 \quad \text{o.k.}$$

$$H/t = 10.0 \text{ in.}/0.375 \text{ in.} \leq 35$$

$$= 26.7 < 35 \quad \text{o.k.}$$

Wall slenderness for tension branch 2:

$$B_b/t_b = H_b/t_b \leq 35$$

$$= 5.00 \text{ in.}/0.313 \text{ in.} \leq 35$$

$$= 16.0 < 35 \quad \text{o.k.}$$

Wall slenderness for compression branch 1:

$$1.1\sqrt{\frac{E}{F_{yb}}} = 1.1\sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ = 26.5$$

$$B_b/t_b = H_b/t_b \leq 26.5 \\ = 5.00 \text{ in.}/0.313 \text{ in.} \leq 26.5 \\ = 16.0 < 26.5 \quad \mathbf{o.k.}$$

Wall slenderness for compression branch 3:

$$1.1\sqrt{\frac{E}{F_{yb}}} = 1.1\sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ = 26.5$$

$$B_b/t_b = H_b/t_b \leq 26.5 \\ = 4.00 \text{ in.}/0.250 \text{ in.} \leq 26.5 \\ = 16.0 < 26.5 \quad \mathbf{o.k.}$$

Width ratio for branches 1 and 2:

$$B_b/B = H_b/B \geq 0.25 \\ = 5.00 \text{ in.}/10.0 \text{ in.} \geq 0.25 \\ = 0.500 > 0.25 \quad \mathbf{o.k.}$$

Width ratio for branch 3:

$$B_b/B = H_b/B \geq 0.25 \\ = 4.00 \text{ in.}/10.0 \text{ in.} \geq 0.25 \\ = 0.400 > 0.25 \quad \mathbf{o.k.}$$

Aspect ratio for chord:

$$H/B = 10.0 \text{ in.}/10.0 \text{ in.} \\ = 1.00$$

$$0.5 \leq 1.00 \leq 2.0 \quad \mathbf{o.k.}$$

Aspect ratio for branches 1 and 2:

$$H_b/B_b = 5.00 \text{ in.}/5.00 \text{ in.} \\ = 1.00$$

$$0.5 \leq 1.00 \leq 2.0 \quad \mathbf{o.k.}$$

Aspect ratio for branch 3:

$$H_b/B_b = 4.00 \text{ in.}/4.00 \text{ in.} \\ = 1.00$$

$$0.5 \leq 1.00 \leq 2.0 \quad \mathbf{o.k.}$$

Overlap between branches 1 and 2 only:

$$25\% \leq 50\% \leq 100\% \quad \mathbf{o.k.}$$

Overlap between branch 3 and the two diagonal branches, treated as 100%:

$$25\% \leq 100\% \leq 100\% \quad \mathbf{o.k.}$$

Branch size ratio between branches 1 and 2:

$$\begin{aligned} B_{bi}/B_{bj} &= 5.00 \text{ in.}/5.00 \text{ in.} \geq 0.75 \\ &= 1.00 > 0.75 \quad \mathbf{o.k.} \end{aligned}$$

Branch size ratio between branches 3 and 1 or branches 3 and 2:

$$\begin{aligned} B_{bi}/B_{bj} &= 4.00 \text{ in.}/5.00 \text{ in.} \geq 0.75 \\ &= 0.800 > 0.75 \quad \mathbf{o.k.} \end{aligned}$$

Branch thickness ratio between branches 1 and 2:

$$\begin{aligned} t_{bi}/t_{bj} &= 0.313 \text{ in.}/0.313 \text{ in.} \leq 1.0 \\ &= 1.00 \leq 1.0 \quad \mathbf{o.k.} \end{aligned}$$

Branch thickness ratio between branches 3 and 1 or branches 3 and 2:

$$\begin{aligned} t_{bi}/t_{bj} &= 0.250 \text{ in.}/0.313 \text{ in.} \leq 1.0 \\ &= 0.799 < 1.0 \quad \mathbf{o.k.} \end{aligned}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

$$F_{yb} = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$\begin{aligned} F_y/F_u &= 50 \text{ ksi}/65 \text{ ksi} \leq 0.8 \\ &= 0.769 < 0.8 \quad \mathbf{o.k.} \end{aligned}$$

$$\begin{aligned} F_{yb}/F_{ub} &= 50 \text{ ksi}/65 \text{ ksi} < 0.8 \\ &= 0.769 < 0.8 \quad \mathbf{o.k.} \end{aligned}$$

Required strength (expressed as a force in a branch)

From ASCE/SEI 7, Chapter 2, the required strengths for Branches 1, 2, and 3 are:

LRFD	ASD
$P_u = 1.2P_D + 1.6P_L$ $P_{u1} = 1.2(20 \text{ kips}) + 1.6(60 \text{ kips})$ $= 120 \text{ kips}$ $P_{u2} = 1.2(31.6 \text{ kips}) + 1.6(94.6 \text{ kips})$ $= 189 \text{ kips}$ $P_{u3} = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips})$ $= 60.0 \text{ kips}$	$P_a = P_D + P_L$ $P_{a1} = 20 \text{ kips} + 60 \text{ kips}$ $= 80.0 \text{ kips}$ $P_{a2} = 31.6 \text{ kips} + 94.6 \text{ kips}$ $= 126 \text{ kips}$ $P_{a3} = 10 \text{ kips} + 30 \text{ kips}$ $= 40.0 \text{ kips}$

Local yielding of the branch(es) due to uneven load distribution

From AISC *Specification* Table K3.2 and Table 9-2 (Limit State 12), the nominal strength, expressed as a force in each branch is as follows:

Branch 3

For Branch 3, which overlaps both branches 1 and 2, one needs to recognize that both transverse faces are represented by B_{eov} terms because both are welded to overlapped branches and not to the chord. Hence, AISC *Specification* Equation K3-12 needs to be modified to:

$$P_{n,i} = F_{ybi}t_{bi}(2H_{bi} - 4t_{bi} + 2B_{eov}) \quad (\text{from Spec. Eq. K3-12})$$

where

$$B_{eov} = \left(\frac{10}{B_{bj}/t_{bj}} \right) \left(\frac{F_{ybj}t_{bj}}{F_{ybi}t_{bi}} \right) B_{bi} \leq B_{bi} \quad (\text{Spec. Eq. K3-14})$$

and the subscript i refers to the overlapping branch (3) and the subscript j refers to the overlapped branch (1 or 2).

Thus:

$$\begin{aligned} B_{eov} &= \left[\frac{10}{(5.00 \text{ in.}/0.313 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.313 \text{ in.})}{(50 \text{ ksi})(0.250 \text{ in.})} \right] (4.00 \text{ in.}) \leq 4.00 \text{ in.} \\ &= 3.14 \text{ in.} < 4.00 \text{ in.} \\ &= 3.14 \text{ in.} \end{aligned}$$

and

$$\begin{aligned} P_{n3} &= (50 \text{ ksi})(0.250 \text{ in.}) [2(4.00 \text{ in.}) - 4(0.250 \text{ in.}) + 2(3.14 \text{ in.})] \\ &= 166 \text{ kips} \end{aligned}$$

As a check, this is less than the yield strength of the branch:

$$\begin{aligned} P_n &= F_{yb3}A_{b3} \\ &= (50 \text{ ksi})(3.59 \text{ in.}^2) \\ &= 180 \text{ kips} > 166 \text{ kips} \end{aligned}$$

The available strength for the branch local yielding limit state, expressed as a force in branch 3 is:

LRFD	ASD
$\phi = 0.95$ $\phi P_{n3} = 0.95(166 \text{ kips})$ $= 158 \text{ kips} > 60.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.58$ $\frac{P_{n3}}{\Omega} = \frac{166 \text{ kips}}{1.58}$ $= 105 \text{ kips} > 40.0 \text{ kips} \quad \mathbf{o.k.}$

Branch 1

For Branch 1, checking can be performed as an overlapped K-connection with branch 1 overlapping branch 2.

$$P_{n,i} = F_{ybi}t_{bi}(2H_{bi} - 4t_{bi} + B_{eoi} + B_{eov}) \quad (\text{Spec. Eq. K3-11})$$

where

$$B_{eoi} = \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yb_i} t_{b_i}} \right) B_{bi} \leq B_{bi} \quad (\text{Spec. Eq. K3-13})$$

$$= \left[\frac{10}{(10.0 \text{ in.}/0.375 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.375 \text{ in.})}{(50 \text{ ksi})(0.313 \text{ in.})} \right] (5.00 \text{ in.}) \leq 5.00 \text{ in.}$$

$$= 2.25 \text{ in.} < 5.00 \text{ in.}$$

$$= 2.25 \text{ in.}$$

$$B_{eov} = \left(\frac{10}{B_{bj}/t_{bj}} \right) \left(\frac{F_{yb_j} t_{b_j}}{F_{yb_i} t_{b_i}} \right) B_{bi} \leq B_{bi} \quad (\text{Spec. Eq. K3-14})$$

$$= \left[\frac{10}{(5.00 \text{ in.}/0.313 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.313 \text{ in.})}{(50 \text{ ksi})(0.313 \text{ in.})} \right] (5.00 \text{ in.}) \leq 5.00 \text{ in.}$$

$$= 3.13 \text{ in.} < 5.00 \text{ in.} \quad \mathbf{o.k.}$$

and

$$P_{n1} = (50 \text{ ksi})(0.313 \text{ in.}) [2(5.00 \text{ in.}) - 4(0.313 \text{ in.}) + 2.25 \text{ in.} + 3.13 \text{ in.}]$$

$$= 221 \text{ kips}$$

As a check, this is less than the yield strength of the branch:

$$P_n = F_{yb1} A_{b1}$$

$$= (50 \text{ ksi})(5.62 \text{ in.}^2)$$

$$= 281 \text{ kips} > 221 \text{ kips}$$

The available strength for the branch local yielding limit state, expressed as a force in branch 1 is:

LRFD	ASD
$\phi = 0.95$ $\phi P_{n1} = 0.95(221 \text{ kips})$ $= 210 \text{ kips} > 120 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.58$ $\frac{P_{n1}}{\Omega} = \frac{221 \text{ kips}}{1.58}$ $= 140 \text{ kips} > 80.0 \text{ kips} \quad \mathbf{o.k.}$

Branch 2

For Branch 2, which is an overlapped member, the nominal axial strength of this branch—as a proportion of its yield strength (i.e., efficiency)—is not to exceed the nominal axial strength of the overlapping branch, as a proportion of its yield strength, which is the basis of the AISC *Specification* Equation K3-15. Because branch 2 is overlapped by two members, the efficiency of branch 2 is limited by both members 1 and 3.

$$\frac{P_{n2}}{F_{yb2} A_{b2}} \leq \frac{P_{n1}}{F_{yb1} A_{b1}} \text{ and } \frac{P_{n3}}{F_{yb3} A_{b3}}$$

$$P_{n2} \leq \frac{P_{n1} F_{yb2} A_{b2}}{F_{yb1} A_{b1}} \text{ and } \frac{P_{n3} F_{yb2} A_{b2}}{F_{yb3} A_{b3}}$$

$$\leq \frac{(221 \text{ kips})(50 \text{ ksi})(5.62 \text{ in.}^2)}{(50 \text{ ksi})(5.62 \text{ in.}^2)} \text{ and } \frac{(166 \text{ kips})(50 \text{ ksi})(5.62 \text{ in.}^2)}{(50 \text{ ksi})(3.59 \text{ in.}^2)}$$

$$\leq 221 \text{ kips and } 260 \text{ kips}$$

Therefore, $P_{n2} = 221$ kips.

The available strength for the branch local yielding limit state, expressed as a force in branch 2 is:

LRFD	ASD
$\phi = 0.95$ $\phi P_{n2} = 0.95(221 \text{ kips})$ $= 210 \text{ kips} > 189 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.58$ $\frac{P_{n2}}{\Omega} = \frac{221 \text{ kips}}{1.58}$ $= 140 \text{ kips} > 126 \text{ kips} \quad \mathbf{o.k.}$

Chapter 10

HSS-to-HSS Moment Connections

10.1 SCOPE AND BASIS

The scope of this chapter follows, and expands upon, AISC *Specification* Section K4 and Chapter J. It is limited to planar connections between HSS (or box-section members) welded to one another, in the form of T-, Y-, and cross-connections. T- and Y-connections have one branch welded to a continuous chord (or through member), whereas cross-connections have a branch on each (opposite) side of a continuous chord. For the design criteria herein, the centerlines of the branch(es) and the chord member all lie in a common plane. Round HSS-to-round HSS and rectangular HSS-to-rectangular HSS are the only combinations of HSS shapes considered. Branches loaded by bending moments acting in the plane of the connection, and out of the plane of the connection, are covered. Out-of-plane bending moments are not very common in planar structures; this type of loading generally appears more frequently in multi-planar structures. Cases where there is a combination of branch in-plane bending, branch out-of-plane bending, and branch axial load (the latter being covered in Chapter 9) may be handled by checking the connection utilization by means of an interaction between these effects. For round-to-round HSS moment connections, this interaction is given by AISC *Specification* Equation K4-5 in Table K4.1 and in Table 10-1 of this Design Guide (see Section 10.3). The exponents in these equations are a result of investigations that have shown that in-plane bending is less severe than out-of-plane bending, with a simplified lower bound interaction function given by Hoadley and Yura (1985). For rectangular-to-rectangular HSS moment connections, the bending moment and axial force interaction is given by AISC *Specification* Equation K4-9 in Table K4.2 and in Table 10-2 of this Design Guide (see Section 10.3). It can be seen that linear interaction in this case is used in the absence of sufficient alternative evidence. These interactions for combined actions are also in accord with current CIDECT design recommendations (Wardenier et al., 2008a; Packer et al., 2009; ISO, 2013).

The provisions in this chapter are not generally applicable to typical planar triangulated trusses (which are covered in Chapter 9) because these should be analyzed in a manner that results in no bending moments in the web members (see Section 9.4). When K-connection branches are subjected to bending, as well as axial load—for example, due to fluid pressures on branches of submerged offshore K-connections—there is research evidence for gapped K-connections (Pecknold et al., 2007; Wardenier et al., 2008a) to suggest that the connection may be validated by checking that the branch utilization due to bending plus the

branch utilization due to axial load does not exceed 0.8. The interaction summation of 0.8, rather than 1.0, serves as a lower bound for a variety of possible moment loading directions on the K-connection branches, such as branches bending in-plane in the same direction versus in opposite directions, and so forth.

This chapter is applicable to frames with partially rigid or PR moment connections, and fully rigid or FR moment connections which are required, for example, in Vierendeel girders. If the branch-to-chord width ratio, β , is less than unity, the connection will generally not be FR, hence only PR. A PR connection will transfer moment, but the rotation between the connected members will not be negligible, and in the analysis of the structure, the force-deformation response characteristics of the connection need to be included [AISC *Specification* Section B3.4b(b)]. In addition to the strength limit state (moment strength), the connection stiffness and deformation capacity need to be considered with PR connections. This is a topic beyond the scope of this Design Guide, and such characteristics are best obtained by analytical or numerical (e.g., finite element analysis) means, or from published literature (e.g., Mäkeläinen et al., 1988; Grotmann and Sedlacek, 1998; Garifullin et al., 2017).

Connections considered in this chapter are presumed to be remote from an open chord end. An appropriate minimum chord end distance to an open (uncapped) end, to cover various failure modes, would be $l_{end} \geq B$ and H (for rectangular HSS) and $l_{end} \geq D$ (for round HSS).

10.1.1 Vierendeel Connections

Vierendeel girder connections as shown in Figure 10-1 have branch members subjected to substantial bending moments, as well as axial and shear forces. Most of the testing on HSS T-connections loaded by in-plane branch bending has been on isolated connection specimens with a lateral load applied to the vertical branch member. Thus, the connection strength and moment-rotation behavior have been assessed mainly by researchers under moment plus shear loading. Researchers concur that both the strength and flexural rigidity of an unstiffened connection decrease as the chord wall slenderness ratio, B/t , increases, and as the branch-to-chord width ratio, β , decreases. The connection rotational stiffness may considerably influence the moment distribution in statically indeterminate structures. If fully rigid connections are required, such as for Vierendeel frames that are typically analyzed as rigid-jointed, it is recommended to choose matched-width connections with a low chord wall slenderness and a low branch-to-chord thickness ratio. For square

HSS connections with $\beta = 1.0$, it has been shown that $B/t \leq 16$ and $t_b/t \leq 0.5$ will produce a connection with a moment capacity equal to the plastic moment capacity of a compact branch (Wardenier, 1982; Packer, 1993; Packer and Henderson, 1997). These requirements are also substantiated by Korol et al. (1977). A calculated connection moment resistance, in a Vierendeel frame, still needs to be reduced for the effect of branch axial force using the interaction effect described previously.

If the criteria for full rigidity of a connection are not met, the connection is deemed semi-rigid or partially restrained. A variety of means of stiffening have been used to achieve full rigidity, but the most common is a chord doubler plate as shown in Figure 10-2 (Korol et al., 1982). This plate would generally be approximately the same width as the chord member, welded all around its perimeter and with a thickness, t_p , much greater than that of the chord member, t . Under moment loading, the stiffening plate and the chord connecting face cannot be assumed to act compositely; in the tension region these two elements separate (Packer, 1986). Thus, the stiffening plate can conservatively be assumed to substitute for the chord connecting face and, if the plate is

full-width and long enough to include the complete connection deformation pattern, its required thickness can be determined from Table 10-2.

10.2 LIMIT STATES

Typical failure modes (and hence limit states) for axially loaded, truss-type connections are discussed in Chapter 2. Although testing of such connections has been much greater than for HSS-to-HSS moment connections, considerable similarity in connection failure modes between the two loading cases has been reported in international research. The limit states for axially loaded connections have thus been used for the possible limit states in moment-loaded connections. Similarly, the limits of applicability of the available strength equations for HSS-to-HSS moment connections are predominantly replicates of those for counterpart axially loaded connections (Chapter 9).

10.2.1 In-Plane Bending of Branch(es)

The potential failure modes for HSS moment connections, illustrated for a rectangular HSS T-connection under branch



Fig. 10-1. Vierendeel frames using rectangular HSS.

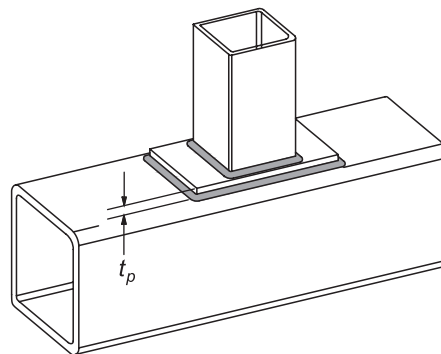


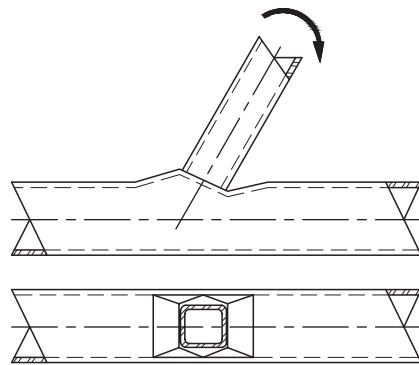
Fig. 10-2. Plate-stiffened Vierendeel frame T-connection.

in-plane bending, are given in Figure 10-3. These limit states are then summarized for this particular connection type, positioned remotely from a chord end.

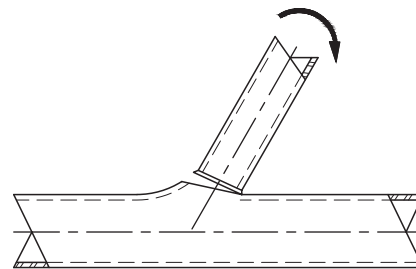
- Chord wall plastification [Figure 10-3(a)] is described in Section 2.3 and can be represented by the yield-line mechanism shown in Figure 10-4. This limit state is covered by AISC *Specification* Section J10.10 and AISC *Manual* Part 9, where Figure 9-5(b) also illustrates the flexural yield-line mechanism. Neglecting the influence of membrane effects and strain hardening, for small deformations the flexural yield load can be determined using the Principle of Virtual Work, with optimization to achieve the minimum yield moment. AISC *Manual* Equation 9-46

gives the solution in a generic form. For a centrally positioned 90° branch with $\beta \leq 0.85$, the in-plane moment capacity simplifies to Equation 2-6. This is also given in Table 10-2, adjusted for Y- and cross-connections with a branch angle less than 90° . A relevant resistance factor of $\phi = 1.00$ for LRFD (or $\Omega = 1.50$ for ASD) is assigned, consistent with other yield line equations in the AISC *Specification*, along with the chord-stress interaction parameter, Q_f , for rectangular HSS.

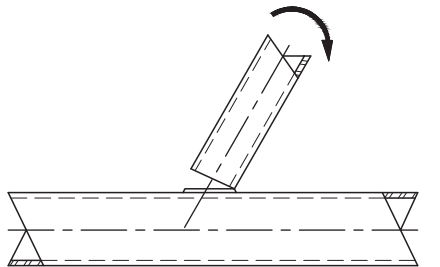
- Shear yielding (punching) of the chord [Figure 10-3(b)] is described in Section 2.4. This failure mode, shown in Figure 10-5, will often be preceded by other failure modes, such as chord wall plastification. Due to non-uniform



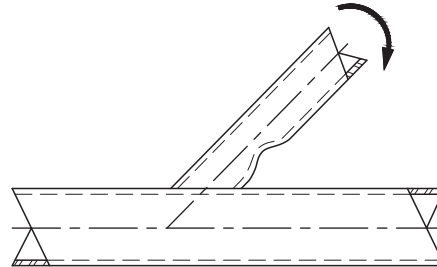
(a) Chord wall plastification



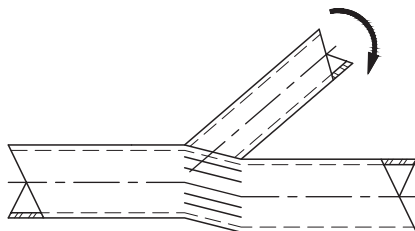
(b) Shear yielding (punching) of the chord



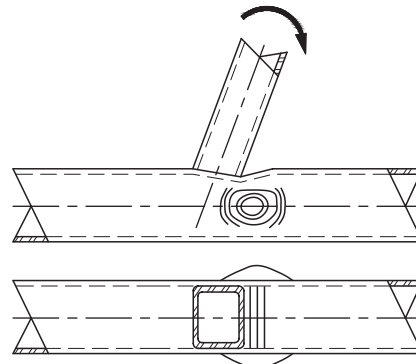
(c) Local yielding of tension side of branch, due to uneven load distribution



(d) Local yielding of compression side of branch, due to uneven load distribution



(e) Shear of the chord sidewall



(f) Chord sidewall failure

Fig. 10-3. Potential limit states for HSS-to-HSS moment connections.

loading around the cross section of the branch member adjacent to the chord connecting face, only part of each transverse branch wall will be effective, shown by the shaded parts of the branch in Figure 10-6. Regardless of branch loading (axial, in-plane bending, or out-of-plane bending), the longitudinal branch walls will be fully effective (because they are adjacent and parallel to the stiff chord sidewalls), while the transverse branch walls will have varying degrees of effectiveness (because they are supported by the flexible chord face). The welds and chord face immediately adjacent to the highly loaded parts of the branch will also be highly loaded. Hence, if chord punching shear occurs it will take place along the two U-shaped segments, each of length $H_b + B_{ep}$ (see Figure 10-6), for a T-connection. The chord effective width for punching shear (considered to be the same on the tension and compression sides of the connection), B_{ep} , is determined according to Equation 2-11. By simplifying the perimeter of an inclined branch to a box shape, and equating the external work done to the internal work done by the shear yield forces, the nominal moment strength (using a shear yield stress of $0.6F_y$) is given by:

$$M_{n-ip} = \frac{0.6F_y t H_b}{\sin \theta} \left(\frac{H_b}{2 \sin \theta} + B_{ep} \right) \quad (10-1)$$

Equation 10-1 is given in Table 10-2 with a resistance factor of $\phi = 0.95$ for LRFD (or $\Omega = 1.58$ for ASD), which is consistent with the same limit state for an axially loaded branch (Limit State 2 in Table 9-2). The chord-stress interaction parameter, Q_f , is also not included, for consistency.

- Local yielding of the branch due to uneven load distribution

[Figures 10-3(c) and 10-3(d)] is described in Section 2.5 and is a failure mode associated with moderate to high β values. An effective width (considered to be the same for the tension and compression “flanges” of the branch member), B_e , according to Equation 2-14 and AISC *Specification* Equation K1-1, is used for the transverse walls of the branch member. The total effective portions of the branch are then as shaded in Figure 10-6 for a T-connection, but B_{ep} must be replaced by B_e . These two terms are different because the branch member effective width, B_e , is influenced by the relative strengths of the branch and the chord to which it is attached, whereas the chord punching shear effective width, B_{ep} , is independent of the branch properties. Assuming a compact branch member, the branch nominal moment strength may be calculated by subtracting the moment contribution of the non-effective flange (transverse wall) portions from the branch (in-plane) plastic moment strength. Thus, for an inclined branch,

$$M_{n-ip} = F_{yb} Z_b - F_{yb} t_b (B_b - B_e) \left(\frac{H_b - t_b}{\sin \theta} \right) \quad (10-2)$$

where Z_b is the plastic section modulus of the branch about the axis of bending. Rearranging terms and simplifying the moment arm to $(H_b/\sin \theta)$ one obtains:

$$M_{n-ip} = F_{yb} \left[Z_b - \left(1 - \frac{B_e}{B_b} \right) \frac{B_b H_b t_b}{\sin \theta} \right] \quad (10-3)$$

Equation 10-3 is a rearrangement of the equations given in AISC *Specification* Commentary Table C-K1.1 and is given in Table 10-2 with a resistance factor of $\phi = 0.95$ for

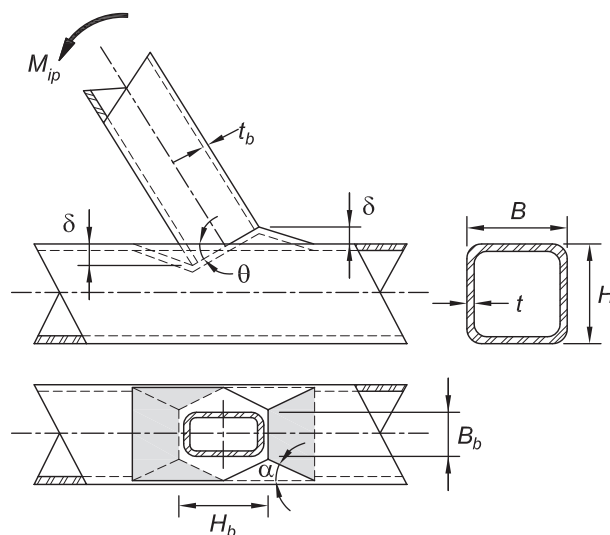


Fig. 10-4. Yield-line mechanism for chord wall plastification for a rectangular HSS-to-HSS T- or Y-connection under in-plane bending.

LRFD (or $\Omega = 1.58$ for ASD), which is consistent with the same limit state for an axially loaded branch (Limit State 3 in Table 9-2). The chord-stress interaction parameter, Q_f , is not included because this failure mode occurs in the branch.

- Shearing of the chord cross section [Figure 10-3(e)] is described in Section 2.9. This can be considered a member failure, covered by AISC *Specification* Chapter G, and is therefore not included in Table 10-2.
- Chord sidewall failure [Figure 10-3(f)] is described in Section 2.6 (for web local yielding), Section 2.7 (for

web crippling), and Section 2.8 (for web buckling). For the limit state of local yielding of the chord side walls, the nominal moment capacity may be derived from stress blocks in each of the chord side walls, acting as a couple at centers of $(0.5H_b + 2.5t)$, as shown in Figure 10-7. This assumes a load dispersion at a slope of 2.5:1 to a depth of t (rather than the chord member “k-line”), which is conservative (Packer and Henderson, 1997). T-connection tests by Mang et al. (1983) and de Koning and Wardenier (1984) have shown that chord sidewall local crippling in the compression region is not critical. Thus, the nominal moment strength for the limit state of local yielding of the



Fig. 10-5. Laboratory test of a rectangular HSS-to-HSS T-connection under branch in-plane bending (McFadden and Packer, 2014): failure by chord wall plastification followed by chord punching shear at ultimate moment ($\beta = 0.5$).

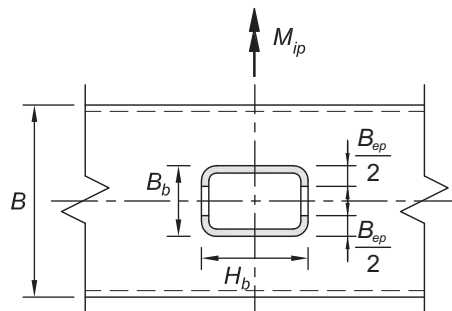


Fig. 10-6. Chord punching shear failure model for a rectangular HSS-to-HSS T-connection under branch in-plane bending (similar for local yielding of the branch).

chord sidewalls in T- and Y-connections, allowing for an inclined branch, is:

$$M_{n-ip} = 0.5F_y t \left(\frac{H_b}{\sin \theta} + 5t \right)^2 \quad (10-4)$$

Equation 10-4 is given in Table 10-2 for Limit State 4 with a resistance factor of $\phi = 1.00$ for LRFD (or $\Omega = 1.50$ for ASD) and Q_f is not included, which is consistent with the similar limit state for an axially loaded branch (Limit State 4 in Table 9-2) and AISC *Specification* Section J10.2. Equation 10-4 is also given in AISC *Specification* Table K4.2 as Equation K4-8.

Buckling of the chord sidewalls in the compression region is a potential limit state for cross-connections with $\beta \approx 1.0$ when the connection is subjected to equilibrating branch moments (in the opposite sense). However, the connection moment strength will be enhanced by yielding of the chord sidewalls in the nearby tension region. To account for this hybrid (compression buckling and tension yielding) behavior, it has been suggested that the failure stress in the model of Figure 10-7 be reduced to $0.8F_y$ (Packer et al., 1992) and treated as local yielding. Thus,

$$M_{n-ip} = 0.4F_y t \left(\frac{H_b}{\sin \theta} + 5t \right)^2 \quad (10-5)$$

Equation 10-5 is given in Table 10-2 for Limit State 5 with a resistance factor of $\phi = 1.00$ for LRFD (or $\Omega = 1.50$ for ASD) and Q_f is excluded, to be consistent with the similar limit state for T- and Y-connections (Limit State 4 in Table 10-2) and AISC *Specification* Section J10.2. Equation 10-5 is also given in AISC *Specification* Table K4.2 as Equation K4-8 but uses the definition $F_y^* = 0.8F_y$.

The nominal moment strengths of round HSS-to-HSS moment connections under branch in-plane bending are

given in AISC *Specification* Table K4.1 and in Table 10-1, based on the two observed failure modes of chord plastification and chord punching shear. These connection capacities are predominantly empirical and are in accord with the CIDECT 1st Ed. Design Guide (Wardenier et al., 1991), with the limits of applicability based on those for axially loaded branches. The resistance factors in AISC *Specification* Table K4.1 and in Table 10-1, in combination with the constants in the nominal moment strength equations, give the same available strengths as in the CIDECT 1st Ed. Design Guide.

10.2.2 Out-of-Plane Bending of Branch(es)

For rectangular HSS T- and cross-connections under branch out-of-plane bending, there is very little test evidence available to support design models, so analogous failure modes to those described for in-plane bending of the branch(es) are postulated. The limit states for rectangular HSS T- and cross-connections, remote from the chord end, under branch out-of-plane bending, are as follows.

- Chord wall plastification, described in Section 2.3, is covered by AISC *Specification* Section J10.10 and AISC *Manual* Part 9, where Figure 9-5(c) illustrates the flexural yield-line mechanism. Neglecting the influence of membrane effects and strain hardening, for small deformations the flexural yield load is given by AISC *Manual* Equation 9-48. For a centrally positioned 90° branch with $\beta \leq 0.85$, the yield-line mechanism is also shown in Figure 10-8, and the corresponding out-of-plane moment strength is given by Equation 10-6. This is also given in Table 10-2 for Limit State 6, with a resistance factor of $\phi = 1.00$ for LRFD (or $\Omega = 1.50$ for ASD), consistent with other yield line equations in the AISC *Specification*, along with the chord-stress interaction parameter, Q_f , for rectangular HSS.

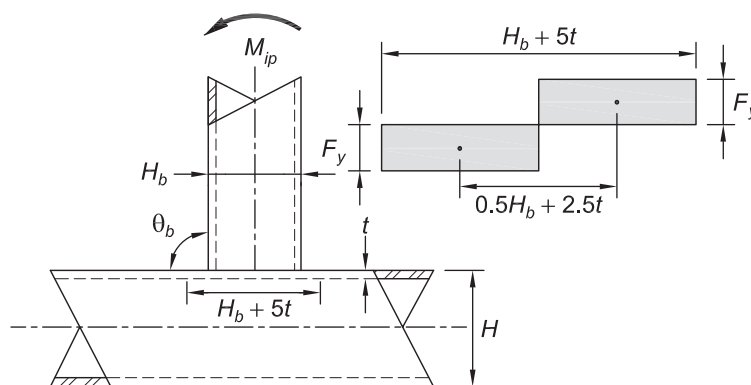


Fig. 10-7. Chord side wall local yielding model for a rectangular HSS-to-HSS T-connection under branch in-plane bending (similar for chord sidewall buckling).

$$M_{n-op} = F_y t^2 \left[\frac{0.5 H_b (1 + \beta)}{1 - \beta} + \sqrt{\frac{2 B B_b (1 + \beta)}{1 - \beta}} \right] Q_f \quad (10-6)$$

- Shear yielding (punching) of the chord, described in Section 2.4, will often be preceded by other failure modes such as chord wall plastification. As explained previously, if chord punching shear occurs it will take place along two U-shaped segments, each of length $H_b + B_{ep}$, even under branch out-of-plane bending. B_{ep} is calculated according to Equation 2-11 and AISC *Specification* Equation K1-2. Again, the branch can be imagined punching through the chord thickness at the furthest point from the axis of rotation, with the punching tapering to zero at the axis of rotation. The total punching shear effective length in the chord connecting face is adjacent to the shaded regions of Figure 10-9, with B_e replaced by B_{ep} . By simplifying the branch perimeter to a box shape, and equating the external work done to the internal work done by the shear yield forces, the nominal moment strength is given by:

$$M_{n-op} = 0.6 F_y t B_b \left[H_b + B_{ep} \left(1 - \frac{B_{ep}}{2 B_b} \right) \right] \quad (10-7)$$

Equation 10-7 is given in Table 10-2 for Limit State 7 with a resistance factor of $\phi = 0.95$ for LRFD (or $\Omega = 1.58$ for ASD), which is consistent with the same limit state for in-plane bending (Table 10-2, Limit State 2). The chord-stress interaction parameter, Q_f , is also not included, for consistency.

- Local yielding of the branch due to uneven load distribution, described in Section 2.5, is a failure mode associated with moderate to high β values. The effective portions of the branch transverse walls, $B_e/2$, are shown in Figure 10-9, with B_e defined by Equation 2-14 and AISC *Specification* Equation K1-1. Assuming a compact branch member, the branch nominal moment strength may be derived by subtracting the moment contribution of the non-effective portions of the branch transverse walls from the branch (out-of-plane) plastic moment strength. Thus,

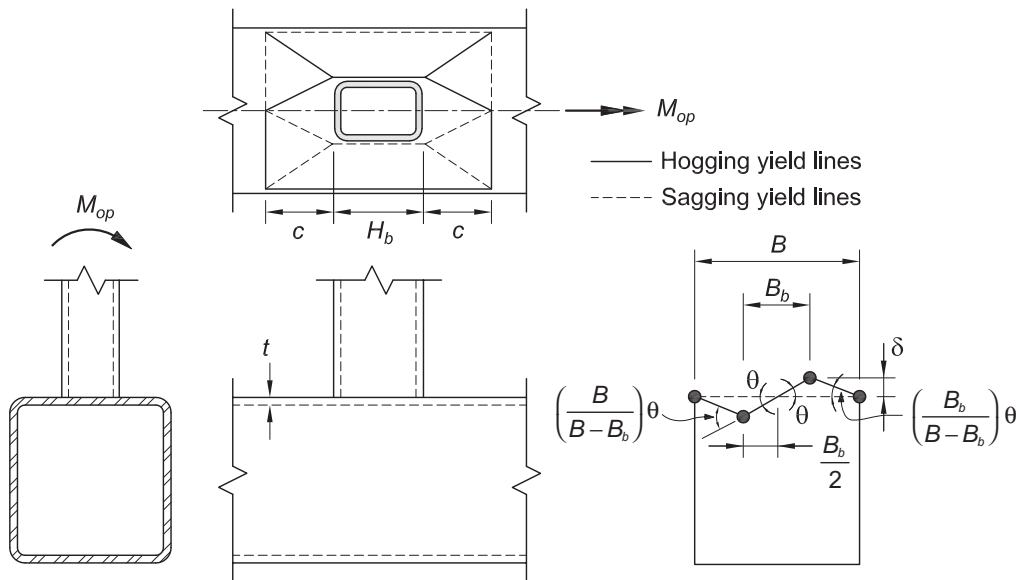


Fig. 10-8. Chord face plastification model for a rectangular HSS-to-HSS T-connection under branch out-of-plane bending.

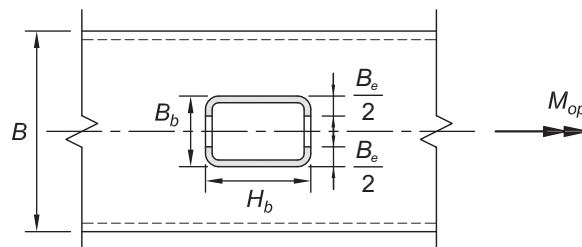


Fig. 10-9. Effective portions of the branch for a rectangular HSS-to-HSS T-connection under branch out-of-plane bending.

$$M_{n-op} = F_{yb}Z_b - \frac{2F_{yb}t_b}{4}(B_b - B_e)^2 \quad (10-8)$$

where Z_b is the plastic section modulus of the branch about the axis of bending. Rearranging, one obtains:

$$M_{n-op} = F_{yb} \left[Z_b - 0.5 \left(1 - \frac{B_e}{B_b} \right)^2 B_b^2 t_b \right] \quad (10-9)$$

Equation 10-9 appears in the AISC *Specification* Commentary Table C-K1.1 and is given in Table 10-2 with a resistance factor of $\phi = 0.95$ for LRFD (or $\Omega = 1.58$ for ASD), which is consistent with the same limit state for an axially loaded branch (Limit State 3 in Table 9-2). The chord-stress interaction parameter, Q_f , is not included because this failure mode occurs in the branch.

- Chord sidewall failure is described in Section 2.6 (for web local yielding), Section 2.7 (for web crippling), and Section 2.8 (for web buckling). For the limit state of local yielding of the chord sidewalls, the moment strength may be derived from the force couple shown in Figure 10-10, acting with a moment arm of $(B - t)$. The nominal moment strength for the limit state of local yielding of the chord sidewalls in T-connections is:

$$M_{n-op} = F_y t (H_b + 5t)(B - t) \quad (10-10)$$

As with the in-plane bending case, the load dispersion is at a slope of 2.5:1 to a depth of the chord thickness, t , which is conservative. Equation 10-10 is given in Table 10-2 with a resistance factor of $\phi = 1.00$ for LRFD (or $\Omega = 1.50$ for ASD) and Q_f is not included, which is consistent with the similar limit state for an axially loaded branch (Limit

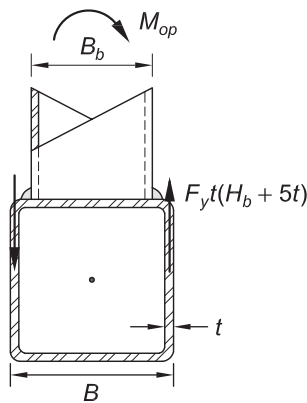


Fig. 10-10. Chord sidewall local yielding model for a rectangular HSS-to-HSS T-connection under branch out-of-plane bending (similar for chord sidewall buckling).

State 4 in Table 9-2) and AISC *Specification* Section J10.2. Equation 10-10 is also given in AISC *Specification* Table K4.2 as Equation K4-6.

Buckling of the chord sidewall is a potential limit state for cross-connections with $\beta \approx 1.0$ when the connection is subjected to equilibrating branch moments (in the opposite sense). Although buckling takes place in one wall, the opposite chord wall is in tension and has a higher capacity. To account for this hybrid behavior, it has been suggested that the failure stress in the model of Figure 10-10 be reduced to $0.8F_y$ (Packer et al., 1992) and treated as local yielding. Thus,

$$M_{n-op} = 0.8F_y t (H_b + 5t)(B - t) \quad (10-11)$$

Equation 10-11 is given in Table 10-2 for Limit State 10 with a resistance factor of $\phi = 1.00$ for LRFD (or $\Omega = 1.50$ for ASD) and Q_f is excluded, to be consistent with the similar limit state for T-connections (Limit State 9 in Table 9-2) and AISC *Specification* Section J10.2. Equation 10-11 is also given in AISC *Specification* Table K4.2 as Equation K4-6 but uses the definition $F_y^* = 0.8F_y$.

- Chord distortional failure is another potential limit state for T-connections and for unbalanced cross-connections (where the branch moments are not equilibrating). This failure mode involves rhomboidal distortion of the chord cross section, with the torsional strength given by (CEN, 2005):

$$M_{n-op} = 2F_y t \left[H_b t + \sqrt{BHt(B + H)} \right] \quad (10-12)$$

Equation 10-12 is given in Table 10-2 for Limit State 11 and in AISC *Specification* Table K4.2 as Equation K4-7. A conservative resistance factor of $\phi = 0.95$ for LRFD (or $\Omega = 1.58$ for ASD) is used, and there is no basis for a Q_f factor. This failure mode can be prevented by the use of stiffeners or diaphragms to maintain the rectangular cross-sectional shape.

The nominal moment strengths of round HSS-to-HSS moment connections under branch out-of-plane bending are given in AISC *Specification* Table K4.1 and in Table 10-1, based on the two observed failure modes of chord plastification and chord punching shear. These connection capacities are predominantly empirical and are in accord with the CIDECT 1st Ed. Design Guide (Wardenier et al., 1991), with the limits of applicability based on those for axially loaded branches. The resistance factors in AISC *Specification* Table K4.1 and in Table 10-1, in combination with the constants in the nominal moment strength equations, give the same available strengths as in the CIDECT 1st Ed. Design Guide.

10.3 HSS-TO-HSS MOMENT CONNECTION AVAILABLE STRENGTH TABLES

Tables 10-1 and 10-2 provide available strengths, subject to certain limits of applicability (Tables 10-1A and 10-2A), for round and rectangular HSS-to-HSS unstiffened moment connections located remote from the end of the main (through) member.

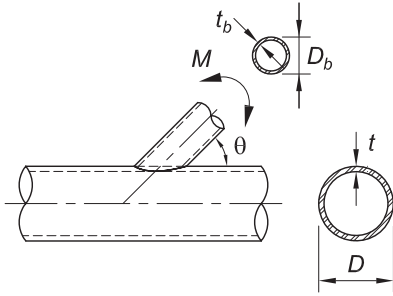
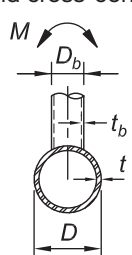
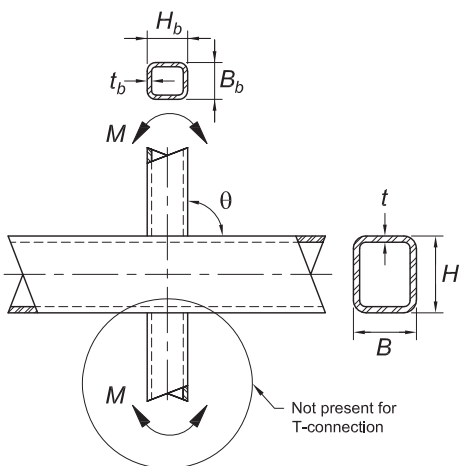
Table 10-1. Available Strengths of Round HSS-to-HSS Moment Connections	
Connection Type	Connection Available Flexural Strength ¹
Branch(es) under in-plane bending T-, Y-, and cross-connections 	Limit State 1: Chord plastification $M_{n-ip} = 5.39 F_y t^2 \gamma^{0.5} \beta \frac{D_b}{\sin \theta} Q_f$ (K4-1)
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
	Limit State 2: Chord shear yielding (punching shear), when $D_b < (D - 2t)$ $M_{n-ip} = 0.6 F_y t D_b^2 \left(\frac{1 + 3 \sin \theta}{4 \sin^2 \theta} \right)$ (K4-2)
	$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
Branch(es) under out-of-plane bending T-, Y-, and cross-connections 	Limit State 3: Chord plastification $M_{n-op} = \frac{F_y t^2 D_b}{\sin \theta} \left(\frac{3.0}{1 - 0.81 \beta} \right) Q_f$ (K4-3)
	$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
	Limit State 4: Chord shear yielding (punching shear), when $D_b < (D - 2t)$ $M_{n-op} = 0.6 F_y t D_b^2 \left(\frac{3 + \sin \theta}{4 \sin^2 \theta} \right)$ (K4-4)
	$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
For T-, Y-, and cross-connections, with branch(es) under combined axial load, in-plane bending, and out-of-plane bending, or any combination of these load effects: $\frac{P_r}{P_c} + \left(\frac{M_{r-ip}}{M_{c-ip}} \right)^2 + \frac{M_{r-op}}{M_{c-op}} \leq 1.0$ (K4-5) P_r = required axial strength in branch, using LRFD or ASD load combinations M_{r-ip} = required in-plane flexural strength in branch, using LRFD or ASD load combinations M_{r-op} = required out-of-plane flexural strength in branch, using LRFD or ASD load combinations P_c = available axial strength, from AISC Specification Table K3.1 and Table 9-1 M_{c-ip} = available strength for in-plane bending, from AISC Specification Table K4.1 and above M_{c-op} = available strength for out-of-plane bending, from AISC Specification Table K4.1 and above	
Functions	
$Q_f = 1$ for chord connecting surface in tension $Q_f = 1.0 - 0.3U(1 + U) \leq 1.0$ for chord connecting surface in compression (K1-3)	
$U = \left \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right \leq 1.0$ (K1-6) where P_{ro} and M_{ro} are determined on the side of the connection that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the chord: $P_{ro} = P_u$ for LRFD, P_a for ASD; $M_{ro} = M_u$ for LRFD, M_a for ASD.	
¹ Equation references are to the AISC Specification.	

Table 10-1A. Limits of Applicability for Limit States in Table 10-1

For Limit States 1 to 4 (per AISC Specification Table K4.1A):

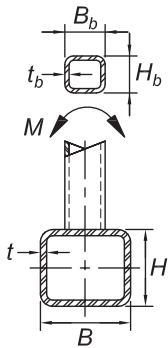
Chord wall slenderness:	$D/t \leq 50$ for T- and Y-connections
	$D/t \leq 40$ for cross-connections
Branch wall slenderness:	$D_b/t_b \leq 50$
	$D_b/t_b \leq 0.05E/F_{yb}$
Width ratio:	$0.2 \leq D_b/D \leq 1.0$
Material strength:	F_y and $F_{yb} \leq 52$ ksi
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \leq 0.8$ Note: ASTM A500/A500M Grade C is acceptable.

Table 10-2. Available Strengths of Rectangular HSS-to-HSS Moment Connections

Connection Type	Connection Available Flexural Strength ¹
Branch(es) under in-plane bending T-, Y-, and cross-connections 	Limit State 1: Chord plastification $M_{n-ip} = \frac{F_y t^2 H_b}{\sin \theta} \left(\frac{1}{2\eta} + \frac{2}{\sqrt{1-\beta}} + \frac{\eta}{1-\beta} \right) Q_f$ (2-6) $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit State 2: Chord shear yielding (punching shear) $M_{n-ip} = \frac{0.6 F_y t H_b}{\sin \theta} \left(\frac{H_b}{2 \sin \theta} + B_{ep} \right)$ (10-1) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 3: Local yielding of the branch/branches due to uneven load distribution $M_{n-ip} = F_{yb} \left[Z_b - \left(1 - \frac{B_e}{B_b} \right) \frac{B_b H_b t_b}{\sin \theta} \right]$ (10-3) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 4: Local yielding of the chord sidewalls $M_{n-ip} = 0.5 F_y t \left(\frac{H_b}{\sin \theta} + 5t \right)^2$ (K4-8), (10-4) $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit State 5: Buckling of the chord sidewalls $M_{n-ip} = 0.4 F_y t \left(\frac{H_b}{\sin \theta} + 5t \right)^2$ (K4-8), (10-5) $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

(Table 10-2 continues on the next page)

Table 10-2. Available Strengths of Rectangular HSS-to-HSS Moment Connections (continued)

Connection Type	Connection Available Flexural Strength ¹
Branch(es) under out-of-plane bending T- and cross-connections ($\theta \approx 90^\circ$) 	Limit State 6: Chord plastification $M_{n-op} = F_y t^2 \left[\frac{0.5 H_b (1 + \beta)}{1 - \beta} + \sqrt{\frac{2 B B_b (1 + \beta)}{1 - \beta}} \right] Q_f$ (10-6) $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit State 7: Chord shear yielding (punching shear) $M_{n-op} = 0.6 F_y t B_b \left[H_b + B_{ep} \left(1 - \frac{B_{ep}}{2 B_b} \right) \right]$ (10-7) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 8: Local yielding of the branch/branches due to uneven load distribution $M_{n-op} = F_{yb} \left[Z_b - 0.5 \left(1 - \frac{B_e}{B_b} \right)^2 B_b^2 t_b \right]$ (10-9) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State 9: Local yielding of the chord side walls $M_{n-op} = F_y t (H_b + 5t) (B - t)$ (K4-6), (10-10) $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit State 10: Buckling of the chord sidewall $M_{n-op} = 0.8 F_y t (H_b + 5t) (B - t)$ (K4-6), (10-11) $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)
	Limit State 11: Chord distortional failure $M_{n-op} = 2 F_y t \left[H_b t + \sqrt{B H t (B + H)} \right]$ (K4-7), (10-12) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

For T-, Y- (in-plane bending only), and cross-connections, with branch(es) under combined axial load, in-plane bending, and out-of-plane bending, or any combination of these load effects:

$$\frac{P_r}{P_c} + \left(\frac{M_{r-ip}}{M_{c-ip}} \right) + \frac{M_{r-op}}{M_{c-op}} \leq 1.0 \quad (K4-9)$$

P_r = required axial strength in branch, using LRFD or ASD load combinations

M_{r-ip} = required in-plane flexural strength in branch, using LRFD or ASD load combinations

M_{r-op} = required out-of-plane flexural strength in branch, using LRFD or ASD load combinations

P_c = available axial strength, from Table 9-2

M_{c-ip} = available strength for in-plane bending, from AISC Specification Table K4.2 and above

M_{c-op} = available strength for out-of-plane bending, from AISC Specification Table K4.2 and above

(Table 10-2 continues on the next page)

Table 10-2. Available Strengths of Rectangular HSS-to-HSS Moment Connections (continued)

Functions	
$Q_f = 1$ for chord connecting surface in tension	
$0.4 \leq Q_f = \left(1.3 - 0.4 \frac{U}{\beta}\right) \leq 1.0$ for chord connecting surface in compression	(K1-4)
$U = \left \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right \leq 1.0$	(K1-6)
where P_{ro} and M_{ro} are determined on the side of the connection that has the higher compression stress. P_{ro} and M_{ro} refer to required strengths in the chord: $P_{ro} = P_u$ for LRFD, P_a for ASD; $M_{ro} = M_u$ for LRFD, M_a for ASD.	
$B_e = \left(\frac{10}{B/t}\right) \left(\frac{F_y t}{F_{yb} t_b}\right) B_b \leq B_b$	(K1-1)
$B_{ep} = \left(\frac{10}{B/t}\right) B_b \leq B_b$	(K1-2)
¹ Equation references are to the AISC <i>Specification</i> or this Design Guide.	

Table 10-2A. Limits of Applicability for Limit States in Table 10-2

For Limit States 1 to 11:	
Limit State 1:	$\beta = B_b/B < 1.0$; plus restrictions on chord wall slenderness, width ratio, and material strength and ductility (because of empirical Q_f).
Limit State 2:	$B_b \leq (B - 2t)$, but this limit state need not be checked for connections within the parameter limits.
Limit State 3:	Restrictions on chord connecting wall slenderness (B/t), branch wall slenderness, width ratio, branch aspect ratio, and material strength and ductility (because of empirical B_e). This limit state need not be checked when $\beta < 0.85$.
Limit State 4:	Applicable for T- and Y-connections, and $\beta \approx 1.0$.
Limit State 5:	Applicable for cross-connections, equilibrating branch moments, and $\beta \approx 1.0$.
Limit State 6:	$\beta = B_b/B < 1.0$; plus restrictions on chord wall slenderness, width ratio, and material strength and ductility (because of empirical Q_f).
Limit State 7:	$B_b \leq (B - 2t)$, but this limit state need not be checked for connections within the parameter limits.
Limit State 8:	Restrictions on chord connecting wall slenderness (B/t), branch wall slenderness, width ratio, branch aspect ratio, and material strength and ductility (because of empirical B_e). This limit state need not be checked when $\beta < 0.85$.
Limit State 9:	Applicable for T-connections, and $\beta \approx 1.0$.
Limit State 10:	Applicable for cross-connections, equilibrating branch moments, and $\beta \approx 1.0$.
Limit State 11:	Applicable for T-connections and for cross-connections with nonequilibrating branch moments. Also, restrictions on chord wall slenderness, chord aspect ratio, and material strength and ductility.
Parameter Limits (per AISC <i>Specification</i> Table K4.2A):	
Chord connecting wall slenderness:	$B/t \leq 35$
Chord side wall slenderness:	$H/t \leq 35$
Branch wall slenderness:	B_b/t_b and $H_b/t_b \leq 35$ $\leq 1.25 \sqrt{\frac{E}{F_{yb}}}$
Width ratio:	$B_b/B \geq 0.25$
Aspect ratio:	$0.5 \leq H_b/B_b \leq 2.0$ and $0.5 \leq H/B \leq 2.0$
Material strength:	F_y and $F_{yb} \leq 52$ ksi
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \leq 0.8$ Note: ASTM A500/A500M Grade C is acceptable.

10.4 HSS-TO-HSS MOMENT CONNECTION DESIGN EXAMPLES

EXAMPLE 10.1—Cross-Connection with Round HSS under Branch In-Plane Bending

Given:

Determine the adequacy of the welded round HSS cross-connection under the member loads shown in Figure 10-11. Loads consist of 25% dead load (P_D , M_D) and 75% live load (P_L , M_L). Assume the welds are strong enough to develop the yield strength of the connected branch wall at all locations around the branch. All members are ASTM A500/A500M Grade C.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500/A500M Grade C

$$F_y = 50 \text{ ksi}$$

$$F_u = 62 \text{ ksi}$$

From AISC *Manual* Table 1-13, the geometric properties are as follows:

Chord

HSS6.000×0.375

$$D = 6.00 \text{ in.}$$

$$t = 0.349 \text{ in.}$$

$$A = 6.20 \text{ in.}^2$$

Branches

HSS4.000×0.250

$$D_b = 4.00 \text{ in.}$$

$$t_b = 0.233 \text{ in.}$$

$$A_b = 2.76 \text{ in.}^2$$

From AISC *Specification* Table K4.1 and Table 10-1, there are two potential limit states for round HSS-to-HSS cross-connections with equilibrating bending moments applied to the branches. Chord shear yielding applies because:

$$\begin{aligned} D - 2t &= 6.00 \text{ in.} - 2(0.349 \text{ in.}) \\ &= 5.30 \text{ in.} > D_b = 4.00 \text{ in.} \end{aligned}$$

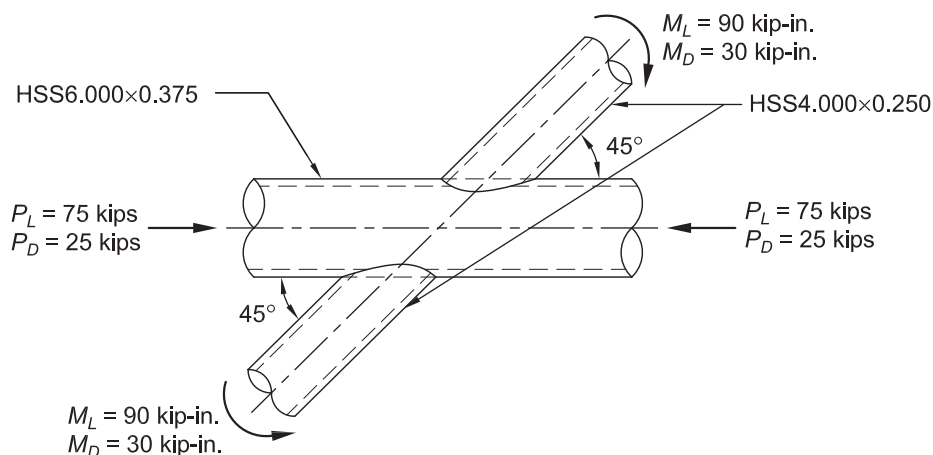


Fig. 10-11. Welded cross-connection with round HSS under branch in-plane bending for Example 10.1.

Limits of applicability

From AISC *Specification* Table K4.1A and Table 10-1A for a round cross-connection, the limits of applicability are:

Chord wall slenderness:

$$\begin{aligned} D/t &= 6.00 \text{ in.}/0.349 \text{ in.} \leq 40 \\ &= 17.2 < 40 \quad \mathbf{o.k.} \end{aligned}$$

Branch wall slenderness:

$$\begin{aligned} D_b/t_b &= 4.00 \text{ in.}/0.233 \text{ in.} \leq 50 \\ &= 17.2 < 50 \quad \mathbf{o.k.} \end{aligned}$$

$$\begin{aligned} 0.05 \left(\frac{E}{F_{yb}} \right) &= 0.05 \left(\frac{29,000 \text{ ksi}}{50 \text{ ksi}} \right) \\ &= 29.0 \\ D_b/t_b &= 17.2 < 29.0 \quad \mathbf{o.k.} \end{aligned}$$

Width ratio:

$$\begin{aligned} D_b/D &= 4.00 \text{ in.}/6.00 \text{ in.} \\ &= 0.667 \\ 0.2 &\leq 0.667 \leq 1.0 \quad \mathbf{o.k.} \end{aligned}$$

Material strength:

$$\begin{aligned} F_y &= 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.} \\ F_{yb} &= 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.} \end{aligned}$$

Ductility:

$$\begin{aligned} F_y/F_u &= 50 \text{ ksi}/62 \text{ ksi} \leq 0.8 \\ &= 0.806 > 0.8 \\ F_{yb}/F_{ub} &= 50 \text{ ksi}/62 \text{ ksi} \leq 0.8 \\ &= 0.806 > 0.8 \end{aligned}$$

However, the note in Table 10-1A indicates that ASTM A500/A500M Grade C is acceptable. **o.k.**

Required flexural strength (expressed as an in-plane bending moment in the branch)

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$\begin{aligned} M_{u-ip} &= 1.2M_D + 1.6M_L \\ &= 1.2(30 \text{ kip-in.}) + 1.6(90 \text{ kip-in.}) \\ &= 180 \text{ kip-in.} \end{aligned}$	$\begin{aligned} M_{a-ip} &= M_D + M_L \\ &= 30 \text{ kip-in.} + 90 \text{ kip-in.} \\ &= 120 \text{ kip-in.} \end{aligned}$

Chord plastification

From AISC *Specification* Table K4.1 and Table 10-1 (Limit State 1), the nominal flexural strength is:

$$M_{n-ip} = 5.39 F_y t^2 \gamma^{0.5} \beta \frac{D_b}{\sin \theta} Q_f \quad (\text{Spec. Eq. K4-1})$$

where

$$\begin{aligned} \beta &= D_b/D \\ &= 4.00 \text{ in.}/6.00 \text{ in.} \\ &= 0.667 \end{aligned}$$

$$\begin{aligned} \gamma &= D/2t \\ &= \frac{6.00 \text{ in.}}{2(0.349 \text{ in.})} \\ &= 8.60 \end{aligned}$$

For a chord connecting surface in compression:

$$Q_f = 1.0 - 0.5U(1+U) \leq 1.0 \quad (\text{Spec. Eq. K1-3})$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{Spec. Eq. K1-6})$$

From ASCE/SEI 7, Chapter 2, the required strength in the chord on the side of the connection with the lower normal compressive stress (which in this case is the same for both sides) is:

LRFD	ASD
$\begin{aligned} P_{ro} &= P_u \\ &= 1.2P_D + 1.6P_L \\ &= 1.2(25 \text{ kips}) + 1.6(75 \text{ kips}) \\ &= 150 \text{ kips} \\ M_{ro} &= 0 \\ F_c &= F_y \\ &= 50 \text{ ksi} \\ U &= \left \frac{150 \text{ kips}}{(50 \text{ ksi})(6.20 \text{ in.}^2)} + 0 \right \leq 1.0 \\ &= 0.484 < 1.0 \\ &= 0.484 \\ Q_f &= 1.0 - 0.3(0.484)(1 + 0.484) \leq 1.0 \\ &= 0.785 < 1.0 \\ &= 0.785 \end{aligned}$	$\begin{aligned} P_{ro} &= P_a \\ &= P_D + P_L \\ &= 25 \text{ kips} + 75 \text{ kips} \\ &= 100 \text{ kips} \\ M_{ro} &= 0 \\ F_c &= 0.6F_y \\ &= 0.6(50 \text{ ksi}) \\ &= 30.0 \text{ ksi} \\ U &= \left \frac{100 \text{ kips}}{(30.0 \text{ ksi})(6.20 \text{ in.}^2)} + 0 \right \leq 1.0 \\ &= 0.538 < 1.0 \\ &= 0.538 \\ Q_f &= 1.0 - 0.3(0.538)(1 + 0.538) \leq 1.0 \\ &= 0.752 < 1.0 \\ &= 0.752 \end{aligned}$

The available flexural strength for the chord plastification limit state is:

LRFD	ASD
$M_{n-ip} = 5.39(50 \text{ ksi})(0.349 \text{ in.})^2 (8.60)^{0.5} (0.667)$ $\times \left(\frac{4.00 \text{ in.}}{\sin 45^\circ} \right) (0.785)$ $= 285 \text{ kip-in.}$ $\phi = 0.90$ $\phi M_{n-ip} = 0.90(285 \text{ kip-in.})$ $= 257 \text{ kip-in.} > 180 \text{ kip-in.} \quad \mathbf{o.k.}$	$M_{n-ip} = 5.39(50 \text{ ksi})(0.349 \text{ in.})^2 (8.60)^{0.5} (0.667)$ $\times \left(\frac{4.00 \text{ in.}}{\sin 45^\circ} \right) (0.752)$ $= 273 \text{ kip-in.}$ $\Omega = 1.67$ $\frac{M_{n-ip}}{\Omega} = \frac{273 \text{ kip-in.}}{1.67}$ $= 163 \text{ kip-in.} > 120 \text{ kip-in.} \quad \mathbf{o.k.}$

Chord shear yielding (punching shear)

From AISC *Specification* Table K4.1 and Table 10-1 (Limit State 2), the nominal flexural strength is:

$$M_{n-ip} = 0.6F_y t D_b^2 \left(\frac{1 + 3 \sin \theta}{4 \sin^2 \theta} \right) \quad (\text{Spec. Eq. K4-2})$$

$$= 0.6(50 \text{ ksi})(0.349 \text{ in.})(4.00 \text{ in.})^2 \left(\frac{1 + 3 \sin 45^\circ}{4 \sin^2 45^\circ} \right)$$

$$= 261 \text{ kip-in.}$$

The available flexural strength for the chord shear yielding limit state is:

LRFD	ASD
$\phi = 0.95$ $\phi M_{n-ip} = 0.95(261 \text{ kip-in.})$ $= 248 \text{ kip-in.} > 180 \text{ kip-in.} \quad \mathbf{o.k.}$	$\Omega = 1.58$ $\frac{M_{n-ip}}{\Omega} = \frac{261 \text{ kip-in.}}{1.58}$ $= 165 \text{ kip-in.} > 120 \text{ kip-in.} \quad \mathbf{o.k.}$

EXAMPLE 10.2—Vierendeel Connection with Square HSS under Branch In-Plane Bending

Given:

A planar Vierendeel frame has an interior connection subjected to the loads shown in Figure 10-12. The loads consist of 50% dead load and 50% live load. Determine the connection adequacy under these in-plane actions. Assume the welds are strong enough to develop the yield strength of the connected branch wall at all locations around the branch. All members are ASTM A1085/A1085M.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A1085/A1085M

$F_y = 50 \text{ ksi}$

$F_u = 65 \text{ ksi}$

From the AISC *ASTM A1085/A1085M Shapes Database* (available at www.aisc.org/manualresources), the geometric properties are as follows:

Chord

HSS6×6×½

$H = 6.00$ in.
 $B = 6.00$ in.
 $t = 0.500$ in.
 $A = 10.4$ in.²
 $S = 16.8$ in.³

Branch
 HSS6×6×¼
 $H_b = 6.00$ in.
 $B_b = 6.00$ in.
 $t_b = 0.250$ in.
 $A_b = 5.59$ in.²
 $Z_b = 11.9$ in.³

Table 10-2 summarizes the five potential limit states for rectangular HSS-to-HSS T-, Y-, and cross-connections under branch in-plane bending. Limit State 1 (chord plastification) does not apply because this is a matched-width connection and $\beta = B_b/B = 1.0$ (see Table 10-2A). Limit State 2 (chord shear yielding or punching shear) does not apply because $B_b > (B - 2t)$ (see Table 10-2A). Limit State 5 (buckling of the chord sidewalls) does not apply because this is not a cross-connection (see Table 10-2A).

However, this Vierendeel connection also acts as an axially loaded T-connection, for which Table 9-2 lists the seven potential limit states for rectangular HSS-to-HSS T-, Y-, and cross-connections. Of these in Table 9-2, Limit State 1 (chord plastification) does not apply because this is a matched-width connection and $\beta = 1.0$ (see Table 9-2A), and Limit State 2 (chord shear yielding or punching shear) does not apply because $B_b > (B - 2t)$ (see Table 9-2A). Similarly, Limit State 6 (buckling of the chord sidewalls) does not apply because this is not a cross-connection (see Table 9-2A), and Limit State 7 (shear of the chord sidewalls in gap region) does not apply because this is not a cross-connection (see Table 9-2A). Considering the relevant limit states for branch axial loading and branch in-plane bending of this T-connection, the connection will ultimately be checked for its utilization under combined branch axial loading and bending.

Limits of applicability for axial and moment loading

From Tables 9-2A and 10-2A, the following represent the combined limits of applicability for all the relevant limit states.

Chord connecting wall and sidewall slenderness:

$$\begin{aligned}
 B/t &= H/t \leq 35 \\
 &= 6.00 \text{ in.}/0.500 \text{ in.} \leq 35 \\
 &= 12.0 < 35 \quad \text{o.k.}
 \end{aligned}$$

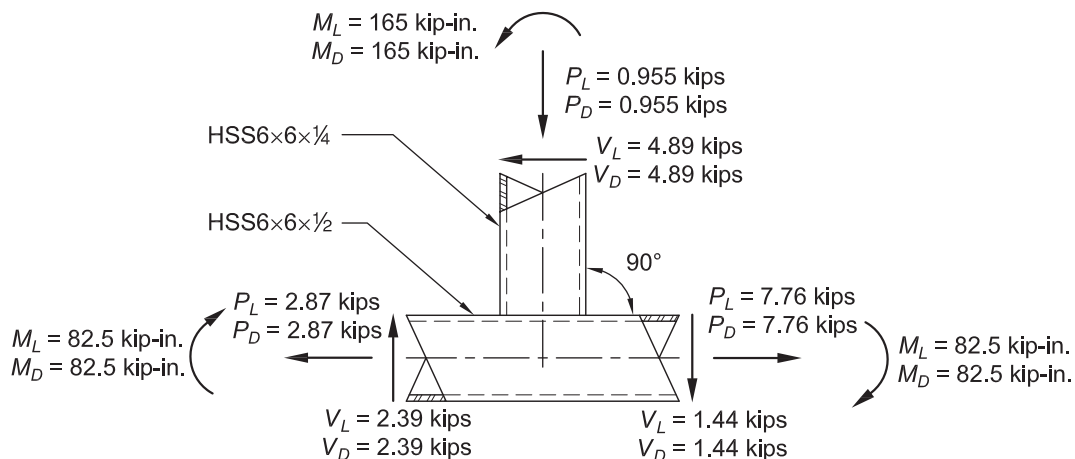


Fig. 10-12. Vierendeel connection with square HSS for Example 10.2.

Branch wall slenderness:

$$\begin{aligned}
 B_b/t_b &= H_b/t_b \leq 35 \\
 &= 6.00 \text{ in.}/0.250 \text{ in.} \leq 35 \\
 &= 24.0 < 35 \quad \mathbf{o.k.} \\
 1.25 \sqrt{\frac{E}{F_{yb}}} &= 1.25 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\
 &= 30.1 \\
 B_b/t_b &= 24.0 < 30.1 \quad \mathbf{o.k.}
 \end{aligned}$$

Width ratio:

$$\begin{aligned}
 B_b/B &= H_b/H \geq 0.25 \\
 &= 6.00 \text{ in.}/6.00 \text{ in.} \geq 0.25 \\
 &= 1.00 > 0.25 \quad \mathbf{o.k.}
 \end{aligned}$$

Aspect ratio for chord and branches:

$$\begin{aligned}
 H/B &= H_b/B_b \\
 &= 6.00 \text{ in.}/6.00 \text{ in.} \\
 &= 1.00 \\
 0.5 &\leq 1.00 \leq 2.0 \quad \mathbf{o.k.}
 \end{aligned}$$

Material strength:

$$\begin{aligned}
 F_y &= 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.} \\
 F_{yb} &= 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}
 \end{aligned}$$

Ductility:

$$\begin{aligned}
 F_y/F_u &= 50 \text{ ksi}/65 \text{ ksi} \leq 0.8 \\
 &= 0.769 < 0.8 \quad \mathbf{o.k.} \\
 F_{yb}/F_{ub} &= 50 \text{ ksi}/65 \text{ ksi} \leq 0.8 \\
 &= 0.769 < 0.8 \quad \mathbf{o.k.}
 \end{aligned}$$

Required axial strength (expressed as a force in the branch)

From ASCE/SEI 7, Chapter 2, the required strength of the connection due to axial load in the branch is:

LRFD	ASD
$ \begin{aligned} P_u &= 1.2P_D + 1.6P_L \\ &= 1.2(0.955 \text{ kips}) + 1.6(0.955 \text{ kips}) \\ &= 2.67 \text{ kips} \end{aligned} $	$ \begin{aligned} P_a &= P_D + P_L \\ &= 0.955 \text{ kips} + 0.955 \text{ kips} \\ &= 1.91 \text{ kips} \end{aligned} $

Required flexural strength (expressed as a moment in the branch)

From ASCE/SEI 7, Chapter 2, the required strength of the connection due to moment in the branch is:

LRFD	ASD
$M_{u-ip} = 1.2M_D + 1.6M_L$ $= 1.2(165 \text{ kip-in.}) + 1.6(165 \text{ kip-in.})$ $= 462 \text{ kip-in.}$	$M_{a-ip} = M_D + M_L$ $= 165 \text{ kip-in.} + 165 \text{ kip-in.}$ $= 330 \text{ kip-in.}$

Analysis under branch axial loading

Local yielding of the branch due to uneven load distribution

From Table 9-2 (Limit State 3), the nominal strength is:

$$P_n = F_{yb}t_b(2H_b + 2B_e - 4t_b) \quad (2-15)$$

where B_e is the effective width of branch face transverse to the chord:

$$\begin{aligned}
 B_e &= \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b \\
 &= \left[\frac{10}{(6.00 \text{ in.}/0.500 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.500 \text{ in.})}{(50 \text{ ksi})(0.250 \text{ in.})} \right] (6.00 \text{ in.}) \leq 6.00 \text{ in.} \\
 &= 10.0 \text{ in.} > 6.00 \text{ in.} \\
 &= 6.00 \text{ in.}
 \end{aligned} \quad (\text{Spec. Eq. K1-1})$$

The nominal strength is:

$$\begin{aligned}
 P_n &= (50 \text{ ksi})(0.250 \text{ in.}) [2(6.00 \text{ in.}) + 2(6.00 \text{ in.}) - 4(0.250 \text{ in.})] \\
 &= 288 \text{ kips}
 \end{aligned}$$

The available axial strength for the branch local yielding limit state is:

LRFD	ASD
$\phi = 0.95$ $\phi P_n = 0.95(288 \text{ kips})$ $= 274 \text{ kips}$ Branch utilization: $\frac{P_u}{\phi P_n} = \frac{2.67 \text{ kips}}{274 \text{ kips}}$ $= 0.00975$	$\Omega = 1.58$ $\frac{P_n}{\Omega} = \frac{288 \text{ kips}}{1.58}$ $= 182 \text{ kips}$ Branch utilization: $\frac{P_a}{P_n/\Omega} = \frac{1.91 \text{ kips}}{182 \text{ kips}}$ $= 0.0105$

Local yielding of the chord sidewalls

From Table 9-2 (Limit State 4), the nominal strength is:

$$P_n \sin \theta = 2F_y t \left(7.5t + \frac{H_b}{\sin \theta} \right) \quad (2-17)$$

Therefore,

$$\begin{aligned} P_n &= \frac{2}{\sin 90^\circ} (50 \text{ ksi}) (0.500 \text{ in.}) \left[7.5(0.500 \text{ in.}) + \frac{6.00 \text{ in.}}{\sin 90^\circ} \right] \\ &= 488 \text{ kips} \end{aligned}$$

The available axial strength for the chord sidewall yielding limit state is:

LRFD	ASD
$\phi = 1.00$ $\phi P_n = 1.00(488 \text{ kips})$ $= 488 \text{ kips}$ Branch utilization: $\frac{P_u}{\phi P_n} = \frac{2.67 \text{ kips}}{488 \text{ kips}}$ $= 0.00547$	$\Omega = 1.50$ $\frac{P_n}{\Omega} = \frac{488 \text{ kips}}{1.50}$ $= 325 \text{ kips}$ Branch utilization: $\frac{P_a}{P_n/\Omega} = \frac{1.91 \text{ kips}}{325 \text{ kips}}$ $= 0.00588$

Local crippling of the chord sidewalls

From Table 9-2 (Limit State 5), the nominal strength is:

$$P_n \sin \theta = 1.6t^2 \left(1 + \frac{3H_b/\sin \theta}{H} \right) \sqrt{EF_y} Q_f \quad (\text{from Eq. 2-19})$$

For a chord connecting surface in compression:

$$0.4 \leq Q_f = \left(1.3 - 0.4 \frac{U}{\beta} \right) \leq 1.0 \quad (\text{Spec. Eq. K1-4})$$

On the left side of the connection due to the chord bending moment:

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{Spec. Eq. K1-6})$$

From ASCE/SEI 7, Chapter 2, the required strengths, P_{ro} and M_{ro} , are determined as follows:

LRFD	ASD
$P_{ro} = P_u$ $= 1.2P_D + 1.6P_L$ $= 1.2(2.87 \text{ kips}) + 1.6(2.87 \text{ kips})$ $= 8.04 \text{ kips (causing tension)}$ $M_{ro} = M_u$ $= 1.2M_D + 1.6M_L$ $= 1.2(82.5 \text{ kip-in.}) + 1.6(82.5 \text{ kip-in.})$ $= 231 \text{ kip-in. (causing compression)}$ $F_c = F_y$ $= 50 \text{ ksi}$ $U = \left \frac{8.04 \text{ kips}}{(50 \text{ ksi})(10.4 \text{ in.}^2)} - \frac{231 \text{ kip-in.}}{(50 \text{ ksi})(16.8 \text{ in.}^3)} \right \leq 1.0$ $= 0.260 < 1.0$ $= 0.260$ $Q_f = 1.3 - 0.4 \left(\frac{0.260}{1.00} \right) \leq 1.0$ $= 1.20 > 1.0$ $= 1.00$	$P_{ro} = P_a$ $= P_D + P_L$ $= 2.87 \text{ kips} + 2.87 \text{ kips}$ $= 5.74 \text{ kips (causing tension)}$ $M_{ro} = M_a$ $= M_D + M_L$ $= 82.5 \text{ kip-in.} + 82.5 \text{ kip-in.}$ $= 165 \text{ kip-in. (causing compression)}$ $F_c = 0.6F_y$ $= 0.6(50 \text{ ksi})$ $= 30.0 \text{ ksi}$ $U = \left \frac{5.74 \text{ kips}}{(30.0 \text{ ksi})(10.4 \text{ in.}^2)} - \frac{165 \text{ kip-in.}}{(30.0 \text{ ksi})(16.8 \text{ in.}^3)} \right \leq 1.0$ $= 0.309 < 1.0$ $= 0.309$ $Q_f = 1.3 - 0.4 \left(\frac{0.309}{1.00} \right) \leq 1.0$ $= 1.18 > 1.0$ $= 1.00$

Therefore, the nominal strength for the chord sidewall crippling limit state is:

$$P_n = \frac{1.6(0.500 \text{ in.})^2 \left[1 + \frac{3(6.00 \text{ in.})/\sin 90^\circ}{6.00 \text{ in.}} \right] \sqrt{(29,000 \text{ ksi})(50 \text{ ksi})(1.0)}}{\sin 90^\circ}$$

$$= 1,930 \text{ kips}$$

The available axial strength for the chord sidewall crippling limit state is:

LRFD	ASD
$\phi = 0.75$ $\phi P_n = 0.75(1,930 \text{ kips})$ $= 1,450 \text{ kips}$ Branch utilization: $\frac{P_u}{\phi P_n} = \frac{2.67 \text{ kips}}{1,450 \text{ kips}}$ $= 0.00184$	$\Omega = 2.00$ $\frac{P_n}{\Omega} = \frac{1,930 \text{ kips}}{2.00}$ $= 965 \text{ kips}$ Branch utilization: $\frac{P_a}{P_n/\Omega} = \frac{1.91 \text{ kips}}{965 \text{ kips}}$ $= 0.00198$

Thus, the connection utilization due to branch axial loading is very low and is governed by the limit state of branch local yielding, being 0.01 (1%) for both LRFD and ASD.

Analysis under branch moment loading

Local yielding of the branch due to uneven load distribution

From Table 10-2 (Limit State 3), where $B_e = 6.00$ in. from the analysis under branch axial loading, the nominal flexural strength is:

$$\begin{aligned}
 M_{n-ip} &= F_{yb} \left[Z_b - \left(1 - \frac{B_e}{B_b} \right) \frac{B_b H_b t_b}{\sin \theta} \right] \\
 &= (50 \text{ ksi}) \left[11.9 \text{ in.}^3 - \left(1 - \frac{6.00 \text{ in.}}{6.00 \text{ in.}} \right) (6.00 \text{ in.})(6.00 \text{ in.})(0.250 \text{ in.}) \right] \\
 &= 595 \text{ kip-in.}
 \end{aligned} \tag{10-3}$$

The available flexural strength for the branch local yielding limit state is:

LRFD	ASD
$\phi = 0.95$ $\phi M_{n-ip} = 0.95(595 \text{ kip-in.})$ $= 565 \text{ kip-in.}$ Branch utilization: $\frac{M_{u-ip}}{\phi M_{n-ip}} = \frac{462 \text{ kip-in.}}{565 \text{ kip-in.}}$ $= 0.818$	$\Omega = 1.58$ $\frac{M_{n-ip}}{\Omega} = \frac{595 \text{ kip-in.}}{1.58}$ $= 377 \text{ kip-in.}$ Branch utilization: $\frac{M_{u-ip}}{M_{n-ip}/\Omega} = \frac{330 \text{ kip-in.}}{377 \text{ kip-in.}}$ $= 0.875$

Local yielding of the chord sidewalls

From Table 10-2 (Limit State 4), the nominal flexural strength is:

$$\begin{aligned}
 M_{n-ip} &= 0.5 F_y t \left(\frac{H_b}{\sin \theta} + 5t \right)^2 \\
 &= 0.5(50 \text{ ksi})(0.500 \text{ in.}) \left[\frac{6.00 \text{ in.}}{\sin 90^\circ} + 5(0.500 \text{ in.}) \right]^2 \\
 &= 903 \text{ kip-in.}
 \end{aligned} \tag{10-4}$$

The available flexural strength for the chord sidewall yielding limit state is:

LRFD	ASD
$\phi = 1.00$ $\phi M_{n-ip} = 1.00(903 \text{ kip-in.})$ $= 903 \text{ kip-in.}$ Branch utilization: $\frac{M_{u-ip}}{\phi M_{n-ip}} = \frac{462 \text{ kip-in.}}{903 \text{ kip-in.}}$ $= 0.512$	$\Omega = 1.50$ $\frac{M_{n-ip}}{\Omega} = \frac{903 \text{ kip-in.}}{1.50}$ $= 602 \text{ kip-in.}$ Branch utilization: $\frac{M_{u-ip}}{M_{n-ip}/\Omega} = \frac{330 \text{ kip-in.}}{602 \text{ kip-in.}}$ $= 0.548$

Thus, the connection utilization due to branch moment loading is high and is governed by the limit state of branch local yielding, being 0.82 (82%) for LFRD and 0.88 (88%) for ASD.

Total utilization of the connection capacity, governed by branch local yielding:

$$\frac{P_r}{P_c} + \frac{M_{r-ip}}{M_{c-ip}} + \frac{M_{r-op}}{M_{c-op}} \leq 1.0 \quad (\text{Spec. Eq. K4-9})$$

LRFD	ASD
Utilization = $0.00975 + 0.818 + 0 \leq 1.0$ = $0.828 < 1.0$ o.k.	Utilization = $0.0105 + 0.875 + 0 \leq 1.0$ = $0.886 < 1.0$ o.k.

The summations differ between LRFD and ASD because of the particular live load to dead load ratio (1:1) in this example.

EXAMPLE 10.3—Cross-Connection with Rectangular HSS under Branch Out-Of-Plane Bending

Given:

Verify the adequacy of the welded rectangular HSS cross-connection shown in Figure 10-13, where the branches are subject to balanced out-of-plane bending moments. Loads consist of 25% dead load and 75% live load. Assume the welds are strong enough to develop the yield strength of the connected branch wall at all locations around the branch. All members are ASTM A500/A500M Grade C.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

Chord

HSS16×12× $\frac{1}{2}$

$B = 16.0$ in.

$H = 12.0$ in.

$t = 0.465$ in.

$A = 24.6$ in.²

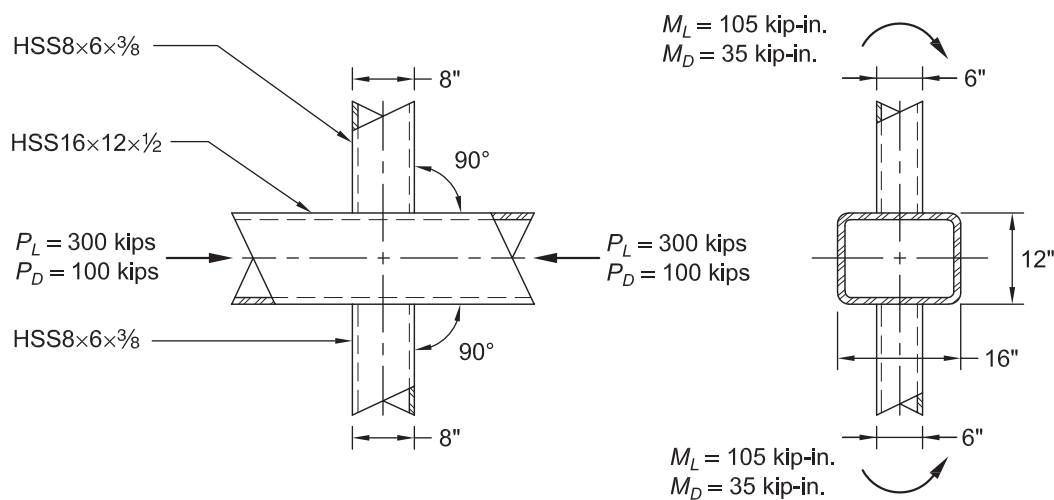


Fig. 10-13. Cross-connection with rectangular HSS under branch out-of-plane bending for Example 10.3.

Branches

HSS8×6× $\frac{3}{8}$

$H_b = 8.00$ in.

$B_b = 6.00$ in.

$t_b = 0.349$ in.

$A_b = 8.97$ in.²

Table 10-2 summarizes the six potential limit states for rectangular HSS-to-HSS 90° T- and cross-connections under branch out-of-plane bending. Limit State 7 (chord shear yielding or punching shear) will not apply if all parameter limits are satisfied (see Table 10-2A). Limit State 8 (local yielding of branches) does not apply because $\beta = 6.00$ in./16.0 in. = 0.375 < 0.85 (see Table 10-2A). Limit State 9 (local yielding of chord sidewalls) and Limit State 10 (buckling of the chord sidewall) do not apply because $\beta < 1.0$. Limit State 11 (chord distortional failure) does not apply because this cross-connection has equilibrating branch moments.

Limits of applicability

From Table 10-2A, the following represent the combined limits of applicability for potential Limit States 6 and 7.

Chord connecting wall slenderness:

$$\begin{aligned} B/t &= 16.0 \text{ in.}/0.465 \text{ in.} \leq 35 \\ &= 34.4 < 35 \quad \text{o.k.} \end{aligned}$$

Chord side wall slenderness:

$$\begin{aligned} H/t &= 12.0 \text{ in.}/0.465 \text{ in.} \leq 35 \\ &= 25.8 < 35 \quad \text{o.k.} \end{aligned}$$

Branch wall slenderness (for the most slender wall):

$$\begin{aligned} H_b/t_b &= 8.00 \text{ in.}/0.349 \text{ in.} \leq 35 \\ &= 22.9 < 35 \quad \text{o.k.} \end{aligned}$$

$$\begin{aligned} 1.25 \sqrt{\frac{E}{F_{yb}}} &= 1.25 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 30.1 \end{aligned}$$

$$H_b/t_b = 22.9 < 30.1 \quad \text{o.k.}$$

Width ratio:

$$\begin{aligned} \beta &= B_b/B \geq 0.25 \\ &= 6.00 \text{ in.}/16.0 \text{ in.} \geq 0.25 \\ &= 0.375 \text{ in.} > 0.25 \quad \text{o.k.} \end{aligned}$$

Aspect ratio for chord:

$$\begin{aligned} H/B &= 12.0 \text{ in.}/16.0 \text{ in.} \\ &= 0.750 \\ 0.5 &\leq 0.750 \leq 2.0 \quad \text{o.k.} \end{aligned}$$

Aspect ratio for branches:

$$\begin{aligned} H_b/B_b &= 8.00 \text{ in.}/6.00 \text{ in.} \\ &= 1.33 \\ 0.5 &\leq 1.33 \leq 2.0 \quad \text{o.k.} \end{aligned}$$

Material strength:

$$F_y = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

$$F_{yb} = 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.}$$

Ductility:

$$\begin{aligned} F_y/F_u &= 50 \text{ ksi}/62 \text{ ksi} \leq 0.8 \\ &= 0.806 > 0.8 \end{aligned}$$

$$\begin{aligned} F_{yb}/F_{ub} &= 50 \text{ ksi}/62 \text{ ksi} \leq 0.8 \\ &= 0.806 > 0.8 \end{aligned}$$

However, the note in Table 10-2A indicates ASTM A500/A500M Grade C is acceptable. **o.k.**

Because all limits of applicability in Table 10-2A are satisfied, only Limit State 6 (Chord plastification) applies.

Required flexural strength (expressed as a moment in the branch)

From ASCE/SEI 7, Chapter 2, the required strength of the connection due to moments in the branches is:

LRFD	ASD
$\begin{aligned} M_{u-op} &= 1.2M_D + 1.6M_L \\ &= 1.2(35 \text{ kip-in.}) + 1.6(105 \text{ kip-in.}) \\ &= 210 \text{ kip-in.} \end{aligned}$	$\begin{aligned} M_{a-op} &= M_D + M_L \\ &= 35 \text{ kip-in.} + 105 \text{ kip-in.} \\ &= 140 \text{ kip-in.} \end{aligned}$

Chord plastification

From Table 10-2 (Limit State 6), the nominal strength is:

$$M_{n-op} = F_y t^2 \left[\frac{0.5H_b(1+\beta)}{1-\beta} + \sqrt{\frac{2BB_b(1+\beta)}{1-\beta}} \right] Q_f \quad (10-6)$$

for chord connecting surface in compression

$$0.4 \leq Q_f = \left(1.3 - 0.4 \frac{U}{\beta} \right) \leq 1.0 \quad (\text{Spec. Eq. K1-4})$$

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \leq 1.0 \quad (\text{Spec. Eq. K1-6})$$

From ASCE/SEI 7, Chapter 2, the required strengths, P_{ro} and M_{ro} , are determined as follows:

LRFD	ASD
$P_{ro} = P_u$ $= 1.2P_D + 1.6P_L$ $= 1.2(100 \text{ kips}) + 1.6(300 \text{ kips})$ $= 600 \text{ kips}$ $M_{ro} = 0$ $F_c = F_y$ $= 50 \text{ ksi}$ $U = \left \frac{600 \text{ kips}}{(50 \text{ ksi})(24.6 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.488 < 1.0$ $= 0.488$ $Q_f = 1.3 - 0.4 \left(\frac{0.488}{0.375} \right)$ $= 0.779$ $0.4 \leq 0.779 \leq 1.0$	$P_{ro} = P_a$ $= P_D + P_L$ $= 100 \text{ kips} + 300 \text{ kips}$ $= 400 \text{ kips}$ $M_{ro} = 0$ $F_c = 0.6F_y$ $= 0.6(50 \text{ ksi})$ $= 30.0 \text{ ksi}$ $U = \left \frac{400 \text{ kips}}{(30.0 \text{ ksi})(24.6 \text{ in.}^2)} + 0 \right \leq 1.0$ $= 0.542 < 1.0$ $= 0.542$ $Q_f = 1.3 - 0.4 \left(\frac{0.542}{0.375} \right)$ $= 0.722$ $0.4 \leq 0.722 \leq 1.0$

The available flexural strength for the limit state of chord plastification is determined as follows:

LRFD	ASD
$M_{n-op} = (50 \text{ ksi})(0.465 \text{ in.})^2$ $\times \left[\frac{0.5(8.00 \text{ in.})(1+0.375)}{1-0.375} + \sqrt{\frac{2(16.0 \text{ in.})(6.00 \text{ in.})(1+0.375)}{1-0.375}} \right]$ $\times (0.779)$ $= 247 \text{ kip-in.}$ $\phi = 1.00$ $\phi M_{n-op} = 1.00(247 \text{ kip-in.})$ $= 247 \text{ kip-in.} > 210 \text{ kip-in.} \quad \text{o.k.}$	$M_{n-op} = (50 \text{ ksi})(0.465 \text{ in.})^2$ $\times \left[\frac{0.5(8.00 \text{ in.})(1+0.375)}{1-0.375} + \sqrt{\frac{2(16.0 \text{ in.})(6.00 \text{ in.})(1+0.375)}{1-0.375}} \right]$ $\times (0.722)$ $= 229 \text{ kip-in.}$ $\Omega = 1.50$ $\frac{M_{n-op}}{\Omega} = \frac{229 \text{ kip-in.}}{1.50}$ $= 153 \text{ kip-in.} > 140 \text{ kip-in.} \quad \text{o.k.}$

EXAMPLE 10.4—T-Connection with Rectangular HSS under Branch Out-of-Plane Bending

Given:

A welded rectangular HSS T-connection, shown in Figure 10-14, is subject to branch out-of-plane bending. This branch moment is resisted by torsion in the chord member. Loads consist of 50% dead load and 50% live load. Assume the welds are noncritical and determine the adequacy of the HSS-to-HSS connection. All members are ASTM A500/A500M Grade C.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500/A500M Grade C

$F_y = 50$ ksi

$F_u = 62$ ksi

From AISC *Manual* Table 1-12, the geometric properties are as follows:

Chord

HSS8×8× $\frac{1}{2}$

$H = 8.00$ in.

$B = 8.00$ in.

$t = 0.465$ in.

$A = 13.5$ in.²

Branch

HSS8×8× $\frac{3}{8}$

$H_b = 8.00$ in.

$B_b = 8.00$ in.

$t_b = 0.349$ in.

$A_b = 10.4$ in.²

$Z_b = 29.4$ in.³

Table 10-2 summarizes the six potential limit states for rectangular HSS-to-HSS 90° T- and cross-connections under branch out-of-plane bending. Limit State 6 (chord plastification) and Limit State 7 (chord shear yielding or punching shear) do not apply because of the matched-width ($\beta = 1.0$) connection (see Table 10-2A). Limit State 10 (buckling of the chord sidewall) does not apply because it is a T-connection (see Table 10-2A). Thus, Limit State 8 (local yielding of the branch), Limit State 9 (local yield of the chord side walls), and Limit State 11 (chord distortional failure) are viable.

Limits of applicability

From Table 10-2A, the following represent the combined limits of applicability for potential Limit States 8, 9, and 11.

Chord connecting wall slenderness:

$$B/t = 8.00 \text{ in.}/0.465 \text{ in.} \leq 35$$

$$= 17.2 < 35 \quad \text{o.k.}$$

Chord sidewall slenderness:

$$H/t = 8.00 \text{ in.}/0.465 \text{ in.} \leq 35$$

$$= 17.2 < 35 \quad \text{o.k.}$$

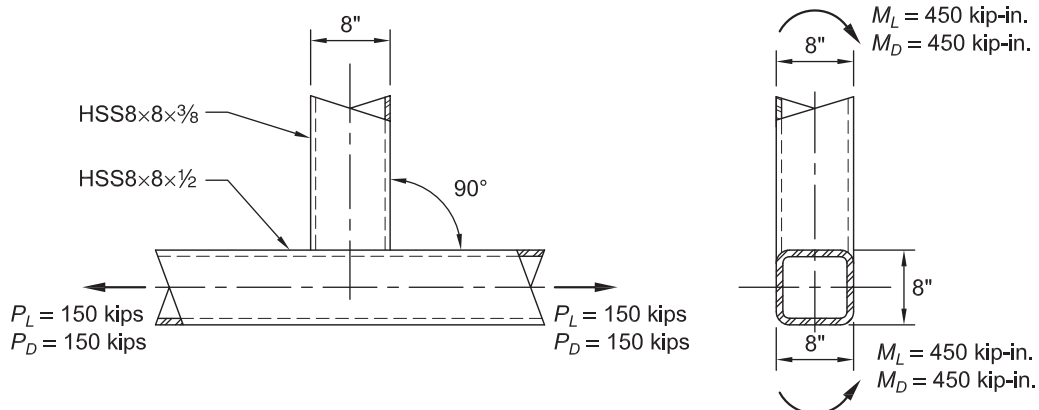


Fig. 10-14. T-connection with rectangular HSS under branch out-of-plane bending for Example 10.4.

Branch wall slenderness:

$$\begin{aligned} B_b/t_b &= H_b/t_b \\ &= 8.00 \text{ in.}/0.349 \text{ in.} \leq 35 \\ &= 22.9 < 35 \quad \mathbf{o.k.} \end{aligned}$$

$$\begin{aligned} 1.25 \sqrt{\frac{E}{F_{yb}}} &= 1.25 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 30.1 \end{aligned}$$

$$B_b/t_b = 22.9 < 30.1 \quad \mathbf{o.k.}$$

Width ratio:

$$\begin{aligned} B_b/B &= 8.00 \text{ in.}/8.00 \text{ in.} \geq 0.25 \\ &= 1.00 > 0.25 \quad \mathbf{o.k.} \end{aligned}$$

Aspect ratio for chord:

$$\begin{aligned} H/B &= 8.00 \text{ in.}/8.00 \text{ in.} \\ &= 1.00 \\ 0.5 &\leq 1.00 \leq 2.0 \quad \mathbf{o.k.} \end{aligned}$$

Aspect ratio for branches:

$$\begin{aligned} H_b/B_b &= 8.00 \text{ in.}/8.00 \text{ in.} \\ &= 1.00 \\ 0.5 &\leq 1.00 \leq 2.0 \quad \mathbf{o.k.} \end{aligned}$$

Material strength:

$$\begin{aligned} F_y &= 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.} \\ F_{yb} &= 50 \text{ ksi} < 52 \text{ ksi} \quad \mathbf{o.k.} \end{aligned}$$

Ductility:

$$\begin{aligned} F_y/F_u &= 50 \text{ ksi}/62 \text{ ksi} \leq 0.8 \\ &= 0.806 > 0.8 \\ F_{yb}/F_{ub} &= 50 \text{ ksi}/62 \text{ ksi} \leq 0.8 \\ &= 0.806 > 0.8 \end{aligned}$$

However, the note in Table 10-2A indicates ASTM A500/A500M Grade C is acceptable. **o.k.**

Required flexural strength (expressed as a moment in the branch)

From ASCE/SEI 7, Chapter 2, the required strength of the connection due to moments in the branches is:

LRFD	ASD
$\begin{aligned} M_{u-op} &= 1.2M_D + 1.6M_L \\ &= 1.2(450 \text{ kip-in.}) + 1.6(450 \text{ kip-in.}) \\ &= 1,260 \text{ kip-in.} \end{aligned}$	$\begin{aligned} M_{a-op} &= M_D + M_L \\ &= 450 \text{ kip-in.} + 450 \text{ kip-in.} \\ &= 900 \text{ kip-in.} \end{aligned}$

Local yielding of the branch due to uneven load distribution

From Table 10-2 (Limit State 8), the nominal flexural strength is:

$$M_{n-op} = F_{yb} \left[Z_b - 0.5 \left(1 - \frac{B_e}{B_b} \right)^2 B_b^2 t_b \right] \quad (10-9)$$

where B_e is the effective width of the branch face transverse to the chord:

$$\begin{aligned} B_e &= \left(\frac{10}{B/t} \right) \left(\frac{F_y t}{F_{yb} t_b} \right) B_b \leq B_b \\ &= \left[\frac{10}{(8.00 \text{ in.}/0.465 \text{ in.})} \right] \left[\frac{(50 \text{ ksi})(0.465 \text{ in.})}{(50 \text{ ksi})(0.349 \text{ in.})} \right] (8.00 \text{ in.}) \leq 8.00 \text{ in.} \\ &= 6.20 \text{ in.} < 8.00 \text{ in.} \\ &= 6.20 \text{ in.} \end{aligned} \quad (\text{Spec. Eq. K1-1})$$

The nominal flexural strength is:

$$\begin{aligned} M_{n-op} &= (50 \text{ ksi}) \left[29.4 \text{ in.}^3 - 0.5 \left(1 - \frac{6.20 \text{ in.}}{8.00 \text{ in.}} \right)^2 (8.00 \text{ in.})^2 (0.349 \text{ in.}) \right] \\ &= 1,440 \text{ kip-in.} \end{aligned}$$

The available flexural strength for the branch local yielding limit state is:

LRFD	ASD
$\phi = 0.95$ $\phi M_{n-op} = 0.95(1,440 \text{ kip-in.})$ $= 1,370 \text{ kip-in.} > 1,260 \text{ kip-in.} \quad \mathbf{o.k.}$	$\Omega = 1.58$ $\frac{M_{n-op}}{\Omega} = \frac{1,440 \text{ kip-in.}}{1.58}$ $= 911 \text{ kip-in.} > 900 \text{ kip-in.} \quad \mathbf{o.k.}$

Local yielding of the chord sidewalls

From Table 10-2 (Limit State 9), the nominal flexural strength is:

$$\begin{aligned} M_{n-op} &= F_y t (H_b + 5t)(B - t) \\ &= (50 \text{ ksi})(0.465 \text{ in.}) [8.00 \text{ in.} + 5(0.465 \text{ in.})] (8.00 \text{ in.} - 0.465 \text{ in.}) \\ &= 1,810 \text{ kip-in.} \end{aligned} \quad (\text{Spec. Eq. K4-6})$$

The available flexural strength for the chord sidewall yielding limit state is:

LRFD	ASD
$\phi = 1.00$ $\phi M_{n-op} = 1.00(1,810 \text{ kip-in.})$ $= 1,810 \text{ kip-in.} > 1,260 \text{ kip-in.} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $\frac{M_{n-op}}{\Omega} = \frac{1,810 \text{ kip-in.}}{1.50}$ $= 1,210 \text{ kip-in.} > 900 \text{ kip-in.} \quad \mathbf{o.k.}$

Chord distortional failure

From Table 10-2 (Limit State 11), the nominal flexural strength is:

$$M_{n-op} = 2F_y t \left[H_b t + \sqrt{B H t (B + H)} \right] \quad (\text{Spec. Eq. K4-7})$$

$$= 2(50 \text{ ksi})(0.465 \text{ in.}) \left[(8.00 \text{ in.})(0.465 \text{ in.}) + \sqrt{(8.00 \text{ in.})(8.00 \text{ in.})(0.465 \text{ in.})(8.00 \text{ in.} + 8.00 \text{ in.})} \right]$$

$$= 1,190 \text{ kip-in.}$$

The available flexural strength for the chord distortional failure limit state is:

LRFD	ASD
$\phi = 0.95$ $\phi M_{n-op} = 0.95(1,190 \text{ kip-in.})$ $= 1,130 \text{ kip-in.} < 1,260 \text{ kip-in.} \quad \mathbf{n.g.}$	$\Omega = 1.58$ $\frac{M_{n-op}}{\Omega} = \frac{1,190 \text{ kip-in.}}{1.58}$ $= 753 \text{ kip-in.} < 900 \text{ kip-in.} \quad \mathbf{n.g.}$

Thus, the connection flexural strength is governed by the limit state of chord distortional failure, and the connection is inadequate. To increase the connection capacity for this limit state, the chord could be made deeper (by selecting an HSS with increased H) or thicker (by selecting an HSS with increased t). Alternatively, diaphragm plates could be welded into the chord on either side of the connection (as shown in Figure 10-15) to prevent this failure mode by maintaining the shape of the cross section.

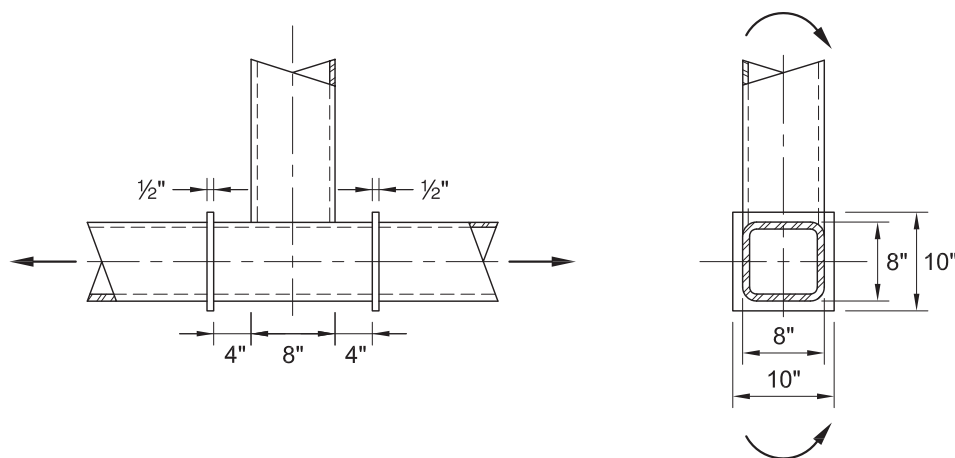


Fig. 10-15. Reinforced T-connection with rectangular HSS under branch out-of-plane bending.

Symbols

A	chord member cross-sectional area, refers to main member for HSS-to-HSS connection, in. ²	B_{eoi}	effective width of overlapping branch member transverse wall landing on the chord in a rectangular HSS overlapped K-connection, measured 90° to the plane of the connection, in.
A_b	branch member cross-sectional area, in. ²	B_{eov}	effective width of overlapping branch member transverse wall landing on the overlapped branch in a rectangular HSS overlapped K-connection, measured 90° to the plane of the connection, in.
A_{bi}	overlapping branch member cross-sectional area in an HSS overlapped K-connection, in. ²	B_{ep}	effective width for punching shear, measured 90° to the plane of the connection, in.
A_{bj}	overlapped branch member cross-sectional area in an HSS overlapped K-connection, in. ²	B_p	width of plate, measured 90° to the plane of the connection, in.
A_e	effective cross-sectional area of member, in. ²	B_{sp}	width of doubler plate stiffener, measured 90° to the plane of the connection, in.
A_{eb}	branch member effective cross-sectional area, in. ²	B_2	second order multiplier to account for P - Δ effects
A_{fg}	gross flange area, in. ²	C	coefficient for eccentrically loaded weld groups; compression force, kips
A_{fn}	net flange area, in. ²	C_a	required compressive strength using ASD load combinations, kips
A_g	gross cross-sectional area of member, in. ²	C_r	required compressive strength, kips
A_{gv}	gross area subject to shear, in. ²	C_u	required compressive strength using LRFD load combinations, kips
A_n	net cross-sectional area of member, in. ²	C_1	electrode strength coefficient
A_{nt}	net area subject to tension, in. ²	C_1, C_2	experimentally determined constants used to calculate the nominal strength of a round HSS chord connection for the limit state of wall plastification
A_{nv}	net area subject to shear, in. ²	D	outside diameter of round HSS main member, in.
A_{pb}	projected bearing area, in. ²	D	fillet weld size, sixteenths-of-an-inch
A_v	shear area, in. ²	D_b	outside diameter of round HSS branch member, in.
A_{we}	effective area of the weld, in. ²	D_{bi}	outside diameter of overlapping branch member in a round-to-rectangular HSS overlapped K-connection, in.
B	overall width of rectangular HSS main member, measured 90° to the plane of the connection, in.	D_{bj}	outside diameter of overlapped branch member in a round-to-rectangular HSS overlapped K-connection, in.
B_b	overall width of rectangular HSS branch member, measured 90° to the plane of the connection, in.	D_{eff}	effective size of weld, sixteenths-of-an-inch
B_{bi}	overall width of overlapping branch member transverse wall in a rectangular HSS overlapped K-connection, measured 90° to the plane of the connection, in.	D_f	distance from the weld face to the HSS face in a flare bevel groove weld, in.
B_{bj}	overall width of overlapped branch member transverse wall in a rectangular HSS overlapped K-connection, measured 90° to the plane of the connection, in.		
B_e	effective width of rectangular HSS branch member or plate, measured 90° to the plane of the connection, in.		

D_{min}	minimum fillet weld size, sixteenths-of-an-inch	F_{yw}	specified minimum yield stress of web, ksi
D_{req}	required fillet weld size, sixteenths-of-an-inch	H	overall height of rectangular HSS main member, measured in the plane of the connection, in.
E	modulus of elasticity of steel, 29,000 ksi	H_b	overall height of rectangular HSS branch member, measured in the plane of the connection, in.
E	effective throat thickness of weld, in.	H_{bi}	overall height of an overlapping HSS branch member in a rectangular HSS overlapped K-connection, measured in the plane of the connection, in.
F_c	available stress, F_y (LRFD) or $0.6F_y$ (ASD), ksi	H_{bj}	overall height of an overlapped HSS branch member, in a rectangular HSS overlapped K-connection measured in the plane of the connection, in.
F_{cr}	available critical buckling stress, ksi	I	moment of inertia, in. ⁴
F_{EXX}	filler metal classification strength, ksi	I_{ip}	effective moment of inertia of a weld group under in-plane bending, in. ⁴
F_e	elastic buckling stress, ksi	I_{op}	effective moment of inertia of a weld group under out-of-plane bending, in. ⁴
F_f	force in the flange, kips	K	effective length factor
F_{nBM}	nominal stress of the base metal, ksi	K	factor for use in prying calculations, in. ² /kip
F_{nt}	nominal tensile stress, ksi	K_a	relative length factor
F'_{nt}	nominal tensile stress modified to include the effects of shear stress, ksi	L	length of member or element or fastener, in.
F_{nv}	nominal shear stress, ksi	L_c	effective length, in.
F_{nw}	nominal stress of the weld metal, ksi	M	bending moment, kip-in.
F_u	specified minimum ultimate strength, refers to main member for branch-to-HSS connections, ksi	M_a	required flexural strength using ASD load combinations, kip-in.
F_u	specified minimum tensile strength of connecting element, ksi	M_{a-ip}	required in-plane flexural strength in branch, using ASD load combinations, kip-in.
F_{ub}	specified minimum tensile strength of HSS branch member material, ksi	M_{a-op}	required out-of-plane flexural strength in branch, using ASD load combinations, kip-in.
F_{up}	specified minimum ultimate strength of plate, ksi	M_c	available flexural strength, kip-in.
F_{u1}	tensile strength of element in contact with screw head or washer, ksi	M_{c-ip}	available flexural strength for in-plane bending, kip-in.
F_{u2}	tensile strength of element not in contact with screw head or washer, ksi	M_{c-op}	available flexural strength for out-of-plane bending, kip-in.
F_y	specified minimum yield stress, refers to main member for branch-to-HSS connection, ksi	M_D	specified dead load bending moment, kip-in.
F_{yb}	specified minimum yield stress of HSS branch member material, ksi	M_L	specified live load bending moment, kip-in.
F_{ybi}	specified minimum yield stress of overlapping branch member material, in an HSS overlapped K-connection, ksi	M_{ip}	in-plane bending moment, kip-in.
F_{ybj}	specified minimum yield stress of overlapped branch member material, in an HSS overlapped K-connection, ksi	M_n	nominal flexural strength, kip-in.
F_{yf}	specified minimum yield stress of flange, ksi	M_{n-ip}	nominal flexural strength under branch in-plane bending, kip-in.
F_{yp}	specified minimum yield stress of plate, ksi	M_{n-op}	nominal flexural strength under branch out-of-plane bending, kip-in.
F_{ys}	specified minimum yield stress of branch plate or stem plate, ksi		

M_{op}	out-of-plane bending moment, kip-in.	Q	portion of bolt strength available, accounting for prying action, kips
M_p	available flexural strength, kip-in.	Q_f	chord stress interaction parameter
M_{px}	available flexural strength about the x -axis, kip-in.	R	force, kips
M_{pz}	available flexural strength about the z -axis, kip-in.	R	HSS corner radius, in.
M_r	required flexural strength, kip-in.	R	root opening due to skew, in.
M_{ro}	required flexural strength in the chord, kip-in.	R	round HSS radius, in.
M_{rx}	required flexural strength about the x -axis, kip-in.	R_a	required strength using ASD load combinations, kips
M_{rz}	required flexural strength about the z -axis, kip-in.	R_{min}	minimum outside bend radius, ft
M_{r-ip}	required in-plane flexural strength in branch, using LRFD or ASD load combinations as applicable, kip-in.	R_n	nominal strength, kips
M_{r-op}	required out-of-plane flexural strength in branch, using LRFD or ASD load combinations as applicable, kip-in.	R_r	connection required strength, kips
M_t	torsional strength, kip-in.	R_u	required strength using LRFD load combinations, kips
M_u	required flexural strength using LRFD load combinations, kip-in.	R_y	ratio of the expected yield stress to the specified minimum yield stress, F_y
M_{u-ip}	required in-plane flexural strength in branch, using LRFD load combinations, kip-in.	S	chord elastic section modulus, in. ³
M_{u-op}	required out-of-plane flexural strength in branch, using LRFD load combinations, kip-in.	S	PJP groove weld depth, in.
N	maximum dispersed load width, in.	S_{ip}	effective elastic section modulus of welds for in-plane bending, in. ³
O_v	overlap, $(l_{ov}/l_p) \times 100\%$	S_{op}	effective elastic section modulus of welds for out-of-plane bending, in. ³
P	load, kips	S_x	elastic section modulus taken about the x -axis, in. ³
P_a	required axial strength using ASD load combinations, kips	T	distance between web toes of fillets at top and bottom of web, in.
P_c	available axial strength	T_a	required tensile strength of connection using ASD load combinations, kips
P_D	specified axial dead load, kips	T_c	available tensile strength of connection, kips
P_L	specified axial live load, kips	$T_{c, adj}$	available tensile strength of bolts including the effects of prying, kips
P_n	nominal axial strength, kips	T_p	available tensile strength, kips
P_{nt}	nominal tensile strength of screwed joint, per fastener, kips	T_r	required tensile strength of connection, kips
P_{nv}	nominal shear strength of a screwed joint, per fastener, kips	T_u	required tensile strength of connection using LRFD load combinations, kips
P_r	required axial strength, kips	U	chord utilization ratio
P_{ro}	required axial strength in the chord, kips	U	shear lag factor
P_u	required axial strength using LRFD load combinations, kips	U_{bs}	reduction coefficient, used in calculating block shear rupture strength
P_y	available compression strength, kips	V_D	specified shear dead load, kips

V_L	specified shear live load, kips	d'	effective diameter of fastener, in.
V_a	required shear strength using ASD load combinations, kips	d_b	nominal bolt diameter, in.
V_f	shear in the welds at the connection faces, kips	d_h	diameter of bolt hole, in.
V_{fa}	required shear strength in the welds at the connection faces using ASD load combinations, kips	d_p	diameter of part (head or nut) in contact with the inner surface of the HSS, in.
V_{fu}	required shear strength in the welds at the connection faces using LRFD load combinations, kips	d_s	diameter of welded stud, in.
V_h	horizontal component of applied load, kips	d_{sh}	diameter of screw head, in.
V_p	available shear strength, kips	d_w	diameter of washer, in.
V_r	required shear strength, kips	d'_w	effective pull-over diameter, in.
V_u	required shear strength using LRFD load combinations, kips	e	eccentricity in a truss connection, positive being away from the branches, in.
V_v	vertical component of applied load, kips	e	eccentricity of the required force with respect to the centroid of the bolt group or weld group, in.
W	width across the flats, in.	e	eccentricity of the required load with respect to the center of the connected plates, in.
Y_t	coefficient for use in calculating tensile rupture of the tension flange	e	Euler's constant, approximately 2.71828
Z_b	branch plastic section modulus about the axis of bending, in. ³	e_w	eccentricity of the required load with respect to the face of the HSS column, in.
Z_x	plastic section modulus taken about the x -axis, in. ³	e_x	eccentricity of the required force with respect to the centroid of the weld group, in.
a	distance from bolt centerline to edge of end plate or column face, in.	f_{rv}	required shear stress using LRFD or ASD load combinations, ksi
a'	$a_e + d/2 \leq 1.25b + d/2$, for use in prying calculations, in.	f_3	coefficient used for calculating round HSS end-plate available strength in tension
a_e	effective distance from bolt centerline to edge of the fitting, in.	g	gap between toes of branch members in a gapped K-connection, neglecting the welds, in.
b	distance from the bolt centerline to the face of the connected member or element, in.	g	workable gage, in.
b	width of element, in.	h	center-to-center spacing of bolts along the longitudinal axis of the HSS, in.
b'	distance from face of bolt to face of connected member or element, $b - d/2$, in.	h_p	height of plate, in.
b_b	center-to-center spacing of bolts across the width of the HSS, in.	k	outside corner radius of the HSS, which is permitted to be taken as $1.5t$ if unknown, in.
b_{eff}	effective width, in.	k	distance from outer face of flange to the web toe of fillet, in.
b_f	flange width, in.	k_{des}	distance from outer face of flange to the web toe of fillet used for design, in.
b_p	plate width, in.	k_{ds}	directional strength increase factor for fillet welds
c_{eff}	effective width of the attached element accounting for uneven stress distribution, in.	k_1	distance from web centerline to flange toe of fillet used for detailing, in.
d	nominal diameter of fastener, in.	k_1, k_3	coefficients used for calculating round HSS end-plate available strength in tension
d	full nominal depth of the section, in.		

l	length of weld, in.		
l	length of connection, in.	r_1, r_2, r_3	radii used for calculating round HSS end-plate available strength in tension, in.
l_b	bearing length of the load, measured parallel to the axis of the HSS member, in.	s	bolt spacing, in.
l_b	length of bolt group, in.	s	cap plate extension beyond HSS, shelf dimension, in.
l_c	clear distance in the direction of the force between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.	t	design wall thickness of HSS main member, in.
l_e	total effective length of groove and fillet welds to a branch attached to an HSS, for weld strength calculations, in.	t	thickness of element, in.
l_e	edge distance, in.	t_b	design wall thickness of HSS branch member, in.
l_{eh}	horizontal edge distance, in.	t_{bi}	design wall thickness of overlapping branch member, in an HSS overlapped K-connection, in.
l_{end}	distance from the near side of the connecting branch or plate to end of chord, in.	t_{bj}	design wall thickness of overlapped branch member, in an HSS overlapped K-connection, in.
l_{ev}	vertical edge distance, in.	t_c	lesser of depth of penetration and thickness, t_2 , in.
$l_{e,i}$	weld total effective length for the overlapping branch member, in.	t_{des}	design wall thickness of HSS member, in.
$l_{e,j}$	weld total effective length for the overlapped branch member, in.	t_f	thickness of the loaded flange, in.
l_{max}	maximum plate length, in.	t_{min}	minimum thickness of HSS material to develop full strength of the weld, in.
l_{min}	minimum plate length, in.	t_{np}	minimum required thickness to eliminate prying action, in.
l_{nv}	net length subjected to shear, in.	t_p	thickness of plate, in.
l_{ov}	overlap length measured along the connecting face of the chord beneath the two branches, in.	t_s	thickness of stem plate, in.
l_p	projected length of the overlapping branch on the chord, in.	t_{sp}	thickness of doubler plate stiffener, in.
l_w	total weld length, in.	t_w	thickness of web, in.
l_w	width of the Whitmore section, in.	t_w	smallest effective weld throat thickness around the perimeter of branch or plate, in.
m	number of bolts along the longitudinal axis of the HSS	t_w	thickness of washer, in.
n	number of bolts	t_{weld}	weld throat, in.
p	tributary end plate length per bolt, in.	t_1	thickness of element in contact with fastener head or washer, in.
p_{max}	maximum tributary end plate length per bolt, in.	t_2	thickness of element not in contact with fastener head or washer, in.
q_r	required prying force per bolt, kips	w	weld leg size, in.
r	radius of gyration, in.	w_{adj}	adjusted weld size, in.
r_n	nominal strength per bolt, kips	w_b	width of bolt group, in.
$r_{n,edge}$	nominal strength per edge bolt per inch, kips/in.	w_{eq}	equivalent weld size, in.
$r_{n,int}$	nominal strength per interior bolt per inch, kips/in.	w_p	width of plate, in.
		w_s	required cut-out plate projection, in.
		$\bar{x}$	connection eccentricity, in.

Ω	safety factor	ξ	gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord, g/B for rectangular HSS
Ω_{BM}	safety factor for base metal	η	load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width
Ω_b	safety factor for flexure	θ, θ_b	acute angle between the branch and chord, degrees
Ω_c	safety factor for compression	θ	angle between the line of action of the required force and the weld longitudinal axis, degrees
Ω_t	safety factor for tension	θ_{op}	angle between two planes in a multi-planar connection, degrees
Ω_w	safety factor for weld metal	λ_r	limiting width-to-thickness ratio (non-slender/slender)
α	skew angle, degrees	μ	factor to be applied to uni-planar connection strength to obtain multi-planar connection strength
α	1.0 (LRFD), 1.6 (ASD)	ρ	b'/a'
α'	function used in prying calculations	τ	branch-to-chord thickness ratio
α^*	function used in prying calculations	ϕ	resistance factor
β	width ratio; the ratio of branch diameter to chord diameter for round HSS; the ratio of overall branch width to chord width for rectangular HSS	ϕ_{BM}	resistance factor for base metal
β^*	increased width ratio for a stiffened branch plate	ϕ_b	resistance factor for flexure
β_{eff}	effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width	ϕ_c	resistance factor for compression
β_{eop}	effective outside punching parameter, $5\beta/\gamma \leq \beta$	ϕ_t	resistance factor for tension
β_{max}	maximum width ratio	ϕ_w	resistance factor for weld metal
γ	chord slenderness ratio; the ratio of one-half the diameter to the wall thickness, $D/2t$ for round HSS; the ratio of one-half the width to wall thickness, $B/2t$ for rectangular HSS	ψ	local dihedral angle, degrees
δ	connection deformation, $1 - d_h/p$		

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