A look at some recommendations and requirements related to connections in structural steel framing systems.

steelwise

THE RIGHT CONNECTION

EDITED BY LARRY S. MUIR, P.E.

AISC IS UPDATING the Frequently Asked Questions section of its website (www.aisc.org). As these updates are created, selected sections will be published as SteelWise articles. This month's installment covers connections.

5. Connections

The AISC *Specification for Structural Steel Buildings* covers requirements for the design of structural steel connections. Additional recommendations can be found in the AISC *Steel Construction Manual*. The FAQs in this section include a discussion of portions of these provisions and subsequent recommendations with regard to general issues in connection design, fabrication and erection. For bolting- and welding-specific issues, refer to the FAQs in Section 6 and Section 8, respectively.

5.1. Bolt Holes

5.1.1. Maximum hole sizes for bolts are specified in the *Specification* Table J3.3. What if an actual hole dimension is between two of the values?

AISC *Specification* Table J3.3 is based upon the RCSC *Specification* Table 3.1 and contains the maximum dimensions of standard, oversized, short-slotted and long-slotted holes. If an actual dimension exceeds the tabulated maximum, it must be treated as the next larger hole size. For example, a 13⁄16-in. by 1¼-in. slotted hole for a ¾-in.-diameter bolt must be treated as a long-slotted hole because it exceeds the maximum short-slotted hole size (13⁄16 in. by 1 in.). Note that the RCSC *Specification*, in the footnote of Table 3.1, allows a 1⁄32-in. tolerance on these maximum hole sizes as discussed in 2.4.2 and 2.5.5.

5.1.2. Alternatives are provided in the AISC *Specification* in Section J3.10 for the calculation of bearing strength at bolt holes with deformation considered or not considered. What is the philosophical difference between these options?

When deformation is a design consideration, the design strength is limited to the force at which the hole edge has deformed by a maximum of ¼ in. When deformation is not a design consideration, larger hole ovalization is permitted as the material attains its maximum bearing strength.

5.1.3. Does flame-cutting of bolt holes affect connection strength and performance?

Generally, no. Iwankiw and Schlafly (1982) investigated the performance of double-lap joints with holes made by punching, punching with burrs removed, sub-punching and reaming, drilling, flame-cutting and flame-cutting and reaming. The comparison of 18 samples using ½-in.-thick ASTM A36 steel plates with standard holes indicated that there is no significant variation in connection strength according to the method of hole formation under static load. Additional considerations may be warranted for much thicker plates, steel grades other than those tested and cyclically loaded structures.

Iwankiw, N.R. and Schlafly, T.J. (1982), "Effect of Hole-Making on the Strength of Double Lap Joints," *Engineering Journal*, Vol. 19, No. 3, (3rd Qtr.), AISC, Chicago, IL.

Last modified August 23, 2002.

5.1.4. AISC *Specification* Section J3.8 requires that bearing limit states be checked for slip-critical connections. Why is this check necessary? If the bolts go into bearing, hasn't the connection failed?

Although slip-critical connections are designed to resist slip, a target reliability is established similar to that used in main member design, as opposed to the higher reliabaility associated with most connection-related limit states. This is allowed since slip would not result in the loss of the connection between the elements. The language in *Specification* Section J3.8 is to preclude a rupture failure that could result in the loss of the connection between the elements if slip were to occur. In other words, it is intended to provide a consistent level of relaibility against a rupture failure.

5.2. Single-Plate Connections

5.2.1. In the design procedure for single-plate connections in the AISC *Steel Construction Manual*, the plate thickness is limited relative to the bolt diameter and the fillet weld size is required to be 5/8 of the plate thickness. What does this accomplish?

Larry Muir (muir@aisc.org) is AISC's director of technical assistance.

It ensures the plate will be the critical element of the connection and that the connection will possess sufficient rotation capacity to accommodate the simple beam end rotation as required by Section B3.6a of the AISC *Specification*. Bolt bearing will occur before bolt shear and plate yielding will occur before weld rupture; thereby, a ductile limit-state will control the strength of the connection.

5.2.2. Are through-plates always required for single-plate connections to HSS columns?

No. Sherman and Ales (1991) demonstrated that local yielding of the support was not a concern due to the self-limiting nature of simple-shear connection end rotation and that the compressive strength of the HSS column was unaffected by the associated local deformations. However, this same research indicated that punching shear may be of concern for relatively thin supporting material thicknesses. Punching shear can be prevented by selecting an HSS with a wall thickness t_w that meets the following criteria:

t_w is greater than or equal to $(F_{y\,pl})(t_{pl}) / F_{uw}$

where $F_{y\,pl}$ = the yield strength of the single plate

t_{pl} = the thickness of the single plate

F_{uw} = the tensile strength of the HSS wall

Note that this equation differs slightly from that given in Sherman and Ales (1991). Here, the expression is derived at the design strength level (omega factors included) whereas it was previously derived at the nominal strength level (no omega factors). If the actual maximum stress is known, it can be substituted for $F_{y\,pl}$ in the above equation for a less conservative result.

The above minimum thicknesses would also be applicable to a welded plate tension connection (uniform stress distribution assumed). However, for cantilevered bracket connections, which do not have self-limiting rotations, yielding must also be checked.

Sherman, D.R. and Ales, J.M. (1991), "The Design of Shear Tabs with Tubular Columns," *Proceedings of the 1991 AISC National Steel Construction Conference*, AISC, Chicago, IL.

5.3. Other General Information

5.3.1. Are shop assembly requirements, such as sub-punching and reaming or reaming to a template, necessary in contract documents?

The use of modern punching and drilling equipment consistently produces and duplicates hole patterns with excellent dimensional accuracy. Some specifications fail to recognize this capability and still require that matching hole patterns be produced by drilling or reaming through a steel template.

In lieu of a template or assembly drilling or reaming, the fabricator should be allowed to demonstrate the capability to fabricate component structural members to the tolerance and accuracy specified so that further shop assembly to assure proper fit can be eliminated. In some cases, such as large trusses or plate girders, shop assembly may be advisable to reduce the occurrence of field fit-up problems. In any case, responsibility for final fit still rests with the fabricator.

5.3.2. How much of a joint must be in contact to be considered to be in full contact?

Projecting elements of bolted connection attachments, such as clip-angles or end-plates, often are not flat in the plane of the connection because of profile variations due to as-rolled mill tolerances or welding distortions. In double-angle connections, for example, the outstanding legs tend to bend back toward the centerline of the span. Any resulting gaps are usually drawn together when the bolts are installed, except in relatively thick material.

The additional tension in the bolts produced by pulling the plies together is not a concern. High-strength bolts must comply with AISC, RCSC and ASTM requirements, which ensures proper matching of the nut and bolt. One reason for this is to ensure that, if the bolt fails in the tightening operation, the failure will be a torque-tension fracture in the bolt shank—not a thread-stripping failure. When this happens, the bolt fractures completely and must be replaced. If the bolt is pretensioned to a higher value than the specified pretension, tests have shown that there are no negative effects on the bolt during service.

Neither bearing nor slip-critical connections require continuous contact between the plies. Therefore the RCSC *Specifcaiton* defines firm contact as "the condition that exists on a faying surface when the plies are solidly seated against each other, but not necessarily in continuous contact."

When firm contact exists between the connected elements, bolts in shear, or shear and tension, will not be subjected to additional bending stresses.

The slip resistance of slip-critical connections is not dependent on the contact area. It is only a function of the pretension and the slip coefficient of the faying surface. Whether the pretension results in a low clamping stress over a large area, or a higher clamping stress over a smaller area is immaterial.

5.3.3. Is lamellar tearing a signficant concern?

AISC Design Guide 21 states: "The incidence of lamellar tearing today is significantly reduced as compared to the past, due mostly to proper joint selection and better steel chemistry. Current steelmaking practices have helped to minimize lamellar tearing tendencies. With continuously cast steel, the degree of rolling after casting is diminished. The reduction in the amount of rolling has directly affected the degree to which these laminations are flattened, and has correspondingly reduced lamellar tearing tendencies."

Research (Melendrez and Dexter [2000]) demonstrates that W-shapes are not susceptible to lamellar tearing or other through-thickness failures when welded tee joints are made to the flanges at locations away from member ends. Special production practices can be specified for steel plates to enhance through-thickness ductility and assist in reducing the incidence of lamellar tearing. For further information, refer to ASTM A770. However, it must be recognized that the specification of premium-quality steel does not itself eliminate the potential for lamellar tearing—or the need for careful design, detailing and fabrication of highly restrained joints.

Melendrez, M.I. and Dexter, R.J. (2000), "Through-Thickness Properties of Column Flanges in Welded Moment Connections," *Journal of Structural Engineering*, Vol. 126, No. 1, pp. 24-31, ASCE, Reston, VA.

5.3.4. What is shear lag and when must it be considered?

Shear lag describes behavior at an end connection of a tension member where some but not all of the cross-sectional elements are connected; the area that is effective in resisting tension may be less than the full calculated net area. Procedures for treatment of shear lag and determination of the effective net area in bolted and welded connections are provided in the 2010 AISC *Specification* Section D3.3. Alternatively, shear lag concerns can be addressed by selecting a connection length that mobilizes the entire load-transmitting capability.

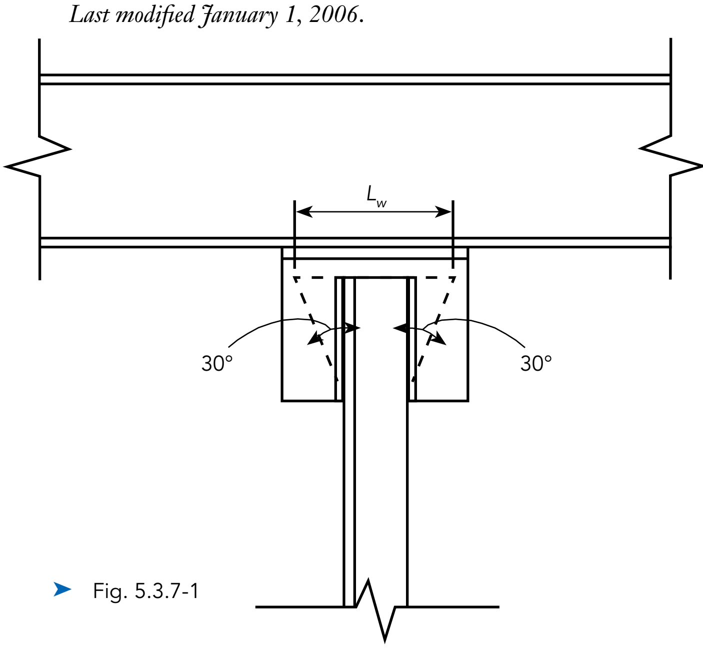
5.3.5. What column stiffening requirements apply at beam-to-column-flange moment connections?

Column stiffening requirements are covered in the AISC *Specification* Section J10 for concentrated flange forces and panel zone shear. Generally, the use of larger columns to eliminate column stiffening, particularly web doubler plates, is recommended. For seismic applications, see the AISC *Seismic Provisions*.

5.3.6. In many design examples in the *Manual of Steel Construction*, yielding and buckling in a gusset plate or similar fitting are checked on a Whitmore section. What is a Whitmore section?

A Whitmore section identifies a theoretically effective cross-sectional area at the end of a connection resisting tension or compression, such as that from a brace-to-gusset-plate connection or similar fitting. As illustrated in Figure 5.3.7-1 for a WT hanger connection, the effective length for the Whitmore section L_w is determined by using a spread-out angle of 30° along both sides of the connection, beginning at the start of the connection. It is applicable to both welded and bolted connections.

Last modified January 1, 2006.



➤ Fig. 5.3.7-1

5.3.7. How can adequate flexibility be maintained in double-angle simple shear connections subjected to combined shear and tension load?

As the tensile force component increases in a double-angle shear connection subjected to combined shear and tension, prying action and/or bending require that the fitting thickness be increased or the bolt gage be decreased, thereby decreasing the available flexibility. Thornton (1995) assesses the ductility of bolts in the outstanding legs of double-angle and similar simple-shear connections.

This study validates the long-standing AISC *Manual* recommendation that maximum angle thickness be limited to ⅝ in. for usual gages (4½ in. to 6½ in.) in double-angle simple-shear connections. For welded connections, a parallel examination can be made as illustrated in Thornton (1996). It should be noted that an alternative connection detail, such as a single-plate connection, may be more feasible for shear-tension applications.

It is important to realize that Section B1.6a in the AISC *Specification* requires only that a simple connection have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure. This may not dictate that bolts must be stronger than the angles. In some instances the beam may be deep relative to its length or lightly loaded in the vertical direction. In either case the required rotation will be small.

Thornton, W.A (1995), "Treatment of Simple Shear Connections Subject to Combined Shear and Axial Forces," *Modern Steel Construction*, September, pp. 9-10, AISC, Chicago, IL.

Thornton, W.A (1996), "A Rational Approach to the Design of Tee Shear Connections," *Engineering Journal*, Vol. 33, No. 1, (1st Qtr.), pp. 34-37, AISC, Chicago, IL.

5.3.8. What are some AISC resources for connection design?

Parts 7-14 of the AISC *Steel Construction Manual* provide a wealth of information related to connection design. Additional information can be found here:

- Design Guides 4 and 16 address the design of end-plate moment connections.
- Design Guide 8 addresses one type of partially-restrained connection.
- Design Guide 13 addresses the stiffening of wide-flange columns at moment connections.
- Design Guide 24 addresses HSS connections.

5.3.9. What are some AISC resources for steel detailers?

Detailing for Steel Construction (3rd Edition, 2009) is an excellent reference that discusses some common detailing practices and has many sample detail drawings. Among other things, the reference has a section on drafting, structural steel, detailing and fabricating of steel, some structural engineering fundamentals (stress and strain), bolted connections, welded connections, columns and framing for industrial buildings.

AISC also has a web-based Detailer Training Series. Originally developed by AISC and the National Institute of Steel Detailing, it is now being made available as a free web-based service thanks to funding from IMPACT (the Ironworker Management Progressive Action Cooperative Trust). See it online at **www.aisc.org/dts**. Note that it also is a great introduction to steel construction for anyone with an interest in steel construction, not just steel detailers. ■