IF YOU'VE EVER ASKED YOURSELF "WHY?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you! Send your questions or comments to solutions@aisc.org.

Strength of a Tapped Hole

We have a unique connection configuration where we would prefer to fasten an ASTM A325 bolt into a threaded hole in a plate in lieu of using a nut. Is there a way to calculate the capacity of a threaded plate used as a nut?

It is difficult to use tapped holes in structural steel connections. The nuts typically used with high-strength bolts are matched so that they develop the strength of the bolt. This simplifies the design of the connection as it precludes thread stripping as a limit state.

Using a tapped plate in lieu of a properly matched nut means that thread stripping in the tapped plate may control the connection design. Or, to preclude thread stripping, the plate would have to be much thicker than the corresponding nut. Typical high-strength bolts also have a limited thread length. This is a practical upper limit on the usable thread length in the tapped hole.

The tapped hole could be designed using references that apply to the more general case of threaded parts. Publications from the Industrial Fasteners Institute (IFI) and other mechanical engineering references contain information on threaded part design, thread geometry and engagement lengths for fasteners. One such document is the IFI Technical Bulletin titled "Calculating Thread Strength." However, because this type of design is beyond the scope of the AISC *Specification*, you will need to rely on your engineering judgment in the selection of appropriate references with which to design this type of connection.

Erin Criste, LEED GA

Fracture Critical Members

Is the bottom flange of a plate girder fracture critical if it is connected to the web using all-bolted double angles?

Fracture Critical Members (FCM) are generally defined as tension members or tension components of members (including those subject to reversal of stress) whose failure would be expected to result in collapse of the bridge. Welding of such tension members must be in accordance with AWS D1.5 FCM requirements.

When incorporated in non-redundant structures, plate girders with welded flanges are considered fracture critical because a crack in the tension flange could propagate through the flange-to-web weld into the web and ultimately compromise the entire girder. Changing the flange-to-web connection from a welded connection to a double-angle bolted connection may be considered by some engineers as a strategy to remove the fracture critical designation because a crack in the flange would have no route to propagate into the web. However, this does not preclude the possibility of a fracture completely through the tension flange.

In that case, the structure's ability to remain standing with a completely fractured tension flange would need to be assessed by the Engineer of Record for the specific structure. Full-scale experiments have shown this can be the case for specific structures; see **www.aisc.org/contentNSBA.aspx?id=20728**.

Bill McEleney

Prying Action

When attaching a hanger connection to the bottom flange of a beam, is it appropriate to use the procedures in Part 9 of the 14th Edition AISC *Steel Construction Manual* to evaluate prying action of the continuous beam bottom flange?

Possibly, but there may be better resources you can use.

AISC *Steel Design Guide No. 4, Extended End-Plate Moment Connections Seismic and Wind Applications*, and *Steel Design Guide No. 16, Flush and Extended Multiple-Row Moment End-Plate Connections*, have procedures for the evaluation of prying action in member flanges that are specific to the four-bolt and eight-bolt end-plate fastener patterns and their associated yield lines. In the past, if your connection did not conform to those specific geometries, you would have to determine the yield-line pattern specific to your connection or use a simplifying conservative assumption of an "effective" flexural length of the flange. However, you may be interested in the article by Bo Dowswell in the 2nd Quarter 2011 *Engineering Journal* titled, "A Yield Line Component Method for Bolted Flange Connections." The article, which is available online at **www.aisc.org/ej**, proposes a general method for evaluating prying action that can be applied to continuous elements such as a beam bottom flange.

Heath Mitchell, S.E., P.E.

Stitch Plate Design

When adding stitch plates between back-to-back angles used for bracing, what criteria are used to design the stitch plate and its connection to the angles?

AISC *Specification* (ANSI/AISC 360-10), Section E6.2, third paragraph states, "Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds or bolts shall be adequate to provide for the transfer of the *required strength*." The question arises as to what is the "required strength" in this case? In a perfect world, until the member buckles, it is zero. After it buckles, there will be a shear flow between the back-to-back angles. But we can't calculate this shear flow until we know the transverse shear, and this is indeterminate in a first-order buckling analysis.

If the required strength of the built-up member is P, a reasonable solution to this situation is to size and space the bolts such that $P/2$ can be transferred from one angle to the other. The logic for this is that the two angles need to share the load, but in an imperfect world it is possible that most of the load flows initially into one due to allowable fabrication and erection tolerances. As the member settles in or "shakes down," the load will be shared by both members.

The AISC *Seismic Provisions* (ANSI/AISC 341-10), Section 13.2e, has very clear requirements for stitches for SCBF. The idea that I presented above is what is required for SCBF, except that stitches in SCBF are required to transfer the expected tensile strength of each element of the built-up brace.

Bill Thornton, P.E., Ph.D.

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Eccentricity in Axially Loaded WT-Sections

Is it necessary to account for the eccentricity on a WT section used as a compression or tension member? That is, can AISC *Steel Construction Manual* Table 4-7 be used for a WT compression member connected at the flange only?

ANSI/AISC 360 Section J1.7 states, "Groups of welds or bolts at the ends of any member which transmit axial force into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of single-angle, double-angle and similar members." Based upon this, it is reasonable to neglect eccentricity for WTs with geometry and connections similar to those typically encountered with double angles. Otherwise, connection eccentricity should be considered and the member designed as a beam-column.

Table 4-7 in the 14th Edition *Manual* considers the axial capacity of the member; provision is not made for any flexural demand. The end connections and their effects on the design of the member are left to the judgment of the engineer.

Heath Mitchell, S.E., P.E.

Minimum Edge Distance

In the 2005 AISC *Specification*, Table J3.4 lists different edge distance values for bolt holes at sheared edges and rolled edges or thermally cut edges. I understand that the provisions of sections J3.10 and J4 must still be met, but what is the reasoning behind the different values in the table for different edge conditions?

In the 2005 *Specification*, there is a disconnect between the edge distance shown in Table J3.4 and the checks made in Sections J3 and J4. This conflict has been resolved in the 2010 *Specification*, which no longer distinguishes between sheared and rolled edges. The commentary to the 2010 *Specification* explains, "In previous editions of the *Specification*, separate minimum edge distances were given in Tables J3.4 and J3.4M for sheared edges and for rolled or thermally cut edges. Sections J3.10 and J4 are used to prevent exceeding bearing and tearout limits, are suitable for use with both thermally cut, sawed and sheared edges, and must be met for all bolt holes. Accordingly, the edge distances in Tables J3.4 and J3.4M are workmanship standards and are no longer dependent on edge condition or fabrication method."

Historically, the difference in edge distance requirements existed because it was felt that the deformation resulting from the shearing of edges would require a larger edge distance. However, the AISC *Specification* that accompanied the 3rd Edition AISC LRFD *Manual* incorporated a new edge distance check in the bearing requirements that did not distinguish between sheared and rolled edges. Because this requirement essentially superseded the edge distance requirements, as evidenced by the footnotes to Table J3.4, it really did not make sense to continue to distinguish between the edge conditions in the table.

As stated in the commentary, the values in Table J3.4 should be viewed as "workmanship standards," i.e. these are good practice, but other edge distances can be justified if the proper limit states are satisfied.

Larry S. Muir, P.E.

K-Factor for Gusset Buckling

The AISC Design Examples use K = 0.5 for compression buckling of some gusset plate connections. The commentary to Appendix 7 of AISC 360-10 lists the theoretical K-factor equal to 0.5 for these support conditions, but recommends using K = 0.65 for design. Is it correct to use K = 0.5 for compression buckling of gusset plates?

Gussets do not buckle in the same way that simple columns buckle. The AISC *Specification* commentary Table C-A-7.1 applies to columns, not gussets per se, and should be used only when no other information is available to evaluate gusset buckling.

The use of K = 0.5 comes from physical research performed by John Gross and published in the 3rd Quarter 1990 *Engineering Journal*. Gross recommended K = 0.5 as a conservative effective length factor for gusset buckling.

Another useful reference on this topic is a paper by Bo Dowswell in the 2nd Quarter 2006 *Engineering Journal* where the author provides a method to determine if a gusset is prone to buckle. All *Engineering Journal* articles are available as free downloads for AISC members at **www.aisc.org/ej**.

Bill Thornton, P.E., Ph.D.

Welding in the *k*-area

Is welding not permitted in the *k*-area of beams, or is it just not recommended?

Though there are some warnings about welding within the *k*-area, there are no prohibitions in the AISC *Specification*. It is relatively common, and in some instances necessary, to weld within the *k*-area.

Section Q5.2 provides additional inspection requirements related to *k*-area welding for connections that must meet the requirements of the AISC *Seismic Provisions*.

Larry S. Muir, P.E.

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