A Tale of Tearouts: Web Supplement

This is a supplement to the May 2017 *Modern Steel Construction* article "A Tale of Tearouts" (available at www.modernsteel.com/archives). The information presented here was compiled while working on the article that appeared in the article, but was not printed to due space limitations.

A little history

Recent discussions and questions received at the AISC Steel Solutions Center related to bolt tearout sometimes tend to treat the limit state as a new and foreign addition to structural steel design. In fact, edge distance checks at bolted connections have existed for some time in the AISC *Specification for Structural Steel Buildings* (ANSI-AISC 360) available at www.aisc.org/specifications. A brief history is provided below.

The 1923 *Specification* stated: "The minimum distance from the center of any rivet hole to a sheared edge shall be 2¼ in. for 1¼-in. rivets, 2 in. for $1^1/_8$-in. rivets, 1¾ in. for 1-in. rivets, 1½ in. for $^7/_8$-in. rivets, 1¼ in. for ¾-in. rivets, $1^1/_8$ in. for $^5/_8$-in. rivets, and 1 in. for ½-in. rivets. The maximum distance from any edge shall be 12 times the thickness of the plate, but shall not exceed 6 in." The minimum ratio of edge distance to rivet diameter is $1^2/_3$. The *Specification* was in some ways less specific back then, as it often gave only allowable stresses and relied on engineers to figure out where to apply them, but it does not seem to be common practice to check a limit state related to tear-out of the bolt through the edge. None was necessary since the bearing strength topped out at only 30 ksi and would govern instead of edge tear-out for any reasonable assumed value for tear-out.

In 1936, the allowable bearing stress was increased from 30 ksi to 40 ksi. New edge distance requirements were also introduced. Section 18(f) stated:

> "The distance from the center of any rivet under computed stress, and that end or other boundary of the connected member toward which the pressure of the rivet is directed, shall be not less than the shearing area of the rivet shank (single or double shear respectively) divided by the plate thickness.
>
> This end distance may however be decreased in such proportion as the stress per rivet is less than that permitted under Section 10 (a); and the requirement may be disregarded in case the rivet in question is one of three or more in a line parallel to the direction of stress."

The Commentary to the 1936 *Specification* stated: "One interesting fact brought out by the test was, that the thinnest specimens failed by shearing a wedge-shaped piece out of the end of the bar. This action would be prevented, and the tensile value of the bar developed, by increasing the end distance beyond the rivet. Had the specimens contained several rivets in line, this should not have occurred, as the yielding of the end of the bar would no doubt have thrown more load back onto the interior rivets. Since there are structural connections in which this type of premature failure might control the design, and since such failure may sometimes be prevented by increase of thickness and in other cases by adding more rivets (two different means of reducing the shear behind a rivet), the Committee has added the provision contained in Section 18 (f)." Other than this tree fastener limit, the description of a model that assumed the throwing of "more load back onto the interior rivets" is exactly the model suggested in the User Note in the 2010 *Specification*.

The edge distance check did not apply to connections with three or more fasteners in the direction of the stress. A similar exclusion existed until 1978 though for much of this time the check was required only for connections with not more than two fasteners (as opposed to excluding the check for connections with three or more fasteners), and the required edge distance or allowable load was adjusted to account for the higher strength of high-strength bolts.

In 1978 the requirement became:

> Along a line of transmitted force, in the direction of the force, the distance from the center of a standard hole to the edge of the connected part shall be not less than
>
> $$2P/F_u t$$

There was no exception related to the number of bolts in a line. This equation can be rewritten as:

$$R_n = 0.5F_u L_e t$$

The 1978 Commentary explains the change: "Recent tests performed at Lehigh University have shown that when the capacity of a connection designed on the basis of the present higher allowable stresses is dependent upon bearing, rather than tension on the effective net area or shear in the fasteners, the critical bearing stress is significantly affected by reduction of the end distance, even with three fasteners in line."

The 1986 *Specification* introduced a different exception as a bearing check that applied to "a single bolt, or two or more bolts in line of force, each with an end distance less than 1½ d." The bearing check was essentially the same as the one in 1978 only shown in LRFD format.

The 1986 requirement appeared in 1989 and 1993 though in a somewhat different forms.

In 1999 the references to the number of bolts and the 1½ d limit were dropped, the current strength equations were adopted and it was stated that: "For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts."

The 1999 Commentary is mostly silent as to why this change was made other than to state that it was to "simplify and generalize such bearing strength calculations." A 1997 *Engineering Journal* paper, "A Summary of Changes and Derivation of LRFD Bolt Design Provisions," states: "Research by Kim and Yura (1996) and Lewis and Zwerneman (1996) indicated that while current AISC and RCSC *Specification* provisions for hole-elongation-controlled bearing strength calculation are correct, tearout-controlled cases are not adequately addressed." The paper also cites a desire to simplify the checks as an impetus for the change.

So, edge-distance-related checks for bolted and riveted connections have been a part of structural steel design from the beginning, though many engineers practicing today were saved from having to consider them for much of their careers by exceptions that were allowed if certain criteria were met.

Example (The original presentation)

Let's consider the connection shown in Figure 3:

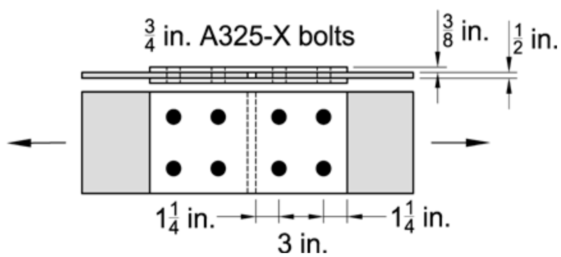

Figure 3

Prior to the 1999 *Specification*, the nominal strength of this connection would have been the lesser of the bolt shear strength and the bearing strength. In this case bearing on the ½-in. plate governs and not the combined ¾-in. thickness of the splice plates.

The Bolt shear strength is:

R_n = (4 bolts)(2 shear planes)(30.1 kips/bolt) = 241 kips

The bearing strength is:

r_n = 2.4($^3/_4$ in. diameter)($^1/_2$ in.)(58 ksi) = 52.2 kips/bolt

R_n = (4 bolts) (52.2 kips/bolt) = 209 kips

The available strength is the lesser, 209 kips.

The 1999 *Specification* added to this procedure consideration of tearout but provided little guidance as to how to apply the new check. One option was to calculate the bearing strength of the critical bolts, the one with the least strength, and apply it to all of the bolts. This is sometimes referred to as the poison bolt model.

The tearout strength is:

$$r_n = 1.2\left[1\,{}^1/_4\text{ in.} - \frac{{}^3/_4\text{ in.} + {}^1/_{16}\text{ in.}}{2}\right]\left({}^1/_2\text{ in.}\right)(58\text{ ksi}) = 29.4\text{ kips}$$

Assuming the poison bolt model the total strength is:

R_n = (4 bolts) (29.4 kips/bolt) = 118 kips

This is a lower bound solution, as the 29.4 kips can be applied to each bolt while satisfying statics and all of the limit states. It is therefore a conservative estimate of the strength, but it may result in uneconomical designs.

The 2010 *Specification* provided further guidance in a User Note, which reads: "The effective strength of an individual fastener is the lesser of the fastener shear strength per Section J3.6 or the bearing strength at the bolt hole per Section J3.10. The strength of the bolt group is the sum of the effective strengths of the individual fasteners." This is based on the Lower Bound Theorem. All of the limit states are checked against a force distribution that satisfies statics. The user note describes the force distribution: Each bolt resists the maximum force it can resist based on the limit states of bearing (which include a consideration of edge distance, sometimes referred to as tearout) and bolt shear. Sufficient ductility is assumed.

Returning to our example, applying the user note results in the following free-body diagram (Figure 4):

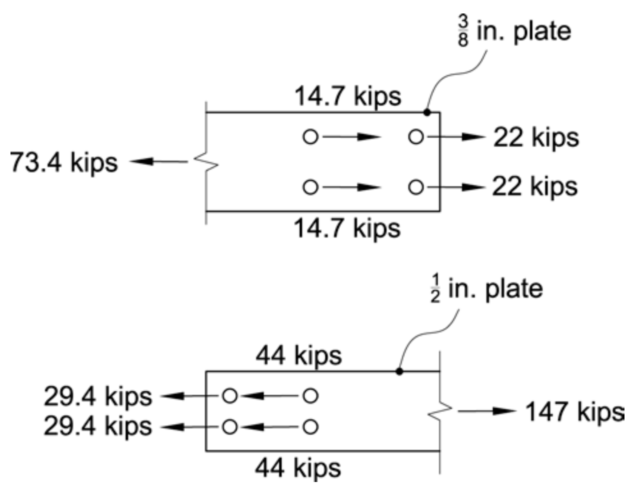

Figure 4

There are essentially five limit states to be checked for each bolt: (1) bolt shear, (2) bearing on the main material, (3) bearing on the connection material, (4) tearout on the main material and (5) tearout on the connection material. For this example, from the free-body diagram:

1. The single shear strength for a ¾-in. A325 bolt with the threads excluded from the shear plane is 30.1 kips/bolt.
2. The bearing strength on the ½-in. plate is 52.2 kips/bolt.
3. The bearing strength on each of the $^3/_8$-in. plates can be found by prorating the strength of the ½-in. plate: 52.2kips/bolt (0.375/0.5) = 39.2 kips/bolt/plate or 78.4 kips/bolt.
4. The tear-out strength at the edge for the ½-in. plate is 29.4 kips/bolt.
5. The tear-out strength at the edge on each of the $^3/_8$-in. plates can be found by prorating the strength of the ½-in. plate: 29.4(0.375/0.5) = 22.0 kips/bolt/plate or 44 kips/bolt.

As is typical the tearout strength between the bolts does not govern, though for unusual conditions it could.

The strength of the bolts at line 1 is governed by the tear-out strength at the edge for the ½-in. plate 2(29.4) = 58.8. Note this is less than the double shear value of 60.2 kips.

The strength of the bolts at line 2 is governed by the tear-out strength at the edge for the $^3/_8$-in. plate 2(22) = 44 kips per shear plane. Note the tearout strength, 22 kips/bolt/plate, is less than the single shear value of 30.1 kips.

The total strength of the connection is 58.8 + 44(2 shear planes) = 147 kips.

This is obviously a good deal more work than simply multiplying by the least strength by the number of bolts, but it does provide a better estimate of the strength of the connection.

The Commentary to the 2016 *Specification* suggests a simplification and states: "For typical connections, such as those shown in the AISC *Steel Construction Manual*, it is acceptable to calculate the shear, bearing and tearout limit states for each bolt in the same connected part and sum the lowest value of the bolt shear or the controlling bearing or tearout limit for each bolt to determine the group strength. The intent is that the separate bearing and tearout equations in this *Specification* be treated in the same way as the combined equations in the 2010 AISC *Specification*. This ignores the potential for interaction of these limit states in multiple connected parts, but that impact is small enough in common connection details within the range of the connections shown in Part 10 of the AISC *Manual,* to allow the benefit of this practical simplification in design. Nonstandard connections may be more sensitive to this interaction; if so, a more exact approach may be necessary."

Though the Commentary specifically cites shear connections shown in Part 10, the key is really that a "reasonable" connection is being considered. The example being considered makes sense. There is some parity between the bolts chosen and the plates and the edge distances are typical of those historically used and recommended in the *Specification*. So, let's apply the Commentary simplification to the example. The predicted strength based on the Commentary model is:

The strength based on bolt shear remains unchanged, 241 kips.

The bearing strength is:

r_n = 52.2 kips/bolt, on the ½-in. plate as shown previously

The tearout strength for the edge bolts is:

r_n = 29.4 kips/bolt, on the ½-in. plate as shown previously

The tearout strength for the interior bolts is:

$$r_n = 1.2\left[3 \text{ in.} - \left({}^3/_4 \text{ in.} + {}^1/_{16 \text{ in.}}\right)\right]\left({}^1/_2 \text{ in.}\right)(58 \text{ ksi}) = 76.2 \text{ kips}$$

Therefore, the strength of the connection is:

(2 bolts)(52.2 kips/bolt) + (2 bolts)(29.4 kips/bolt) = 163 kips.

By inspection the limits states for the $^{3}/_{8}$-in. plate do not govern.

The predicted strength of 163 kips is higher than the 147 kips predicted by the User Note model, but only by about 11%. We knew it would be unsafe, because it starts by assuming a failure mechanism instead of a force distribution. It is an upper bound solution. As described in the Commentary, we have bounded the actual strength of the connection. A comparison of the various methods is presented in Table1.

Table 1. Comparison of Methods

		Pre-1999 (kips)	Poison Bolt (kips)	Lower Bound (kips)	Commentary Simplification (kips)
Example 1	Values	209	118	147	163
	% of Lower Bound	142	80	100	111

Some Finer Points

Table 2 in the *Modern Steel Construction* article chooses the strength based on the five limit state lower bound model as the datum against which the other models are compared. This might give the impression that the five limit state lower bound model correctly predicts the strength and that the other models provide only estimates. This is not necessarily true. The actual strength of the connection, assuming sufficient ductility exists, will not be less than the strength predicted by the five limit state lower bound model, but it could be higher. As stated in the Commentary, "The group strength is a function of strain compatibility dependent on the relative stiffness of the bolts and connected parts." The difference between the lower bound prediction and the actual strength could be less than 11% for the connection considered.

The example above considers only the limit states of bolt shear, bearing and tearout. Assuming a 6-in. effective plate width, the nominal tensile strength of the plate will only be 108 kips, which will govern the strength of the connection rendering the discrepancy between the various methods academic—with the exception of the poison bolt model, which is so conservative it would still govern the predicted design strength. It is not uncommon for other limit states to govern or to predict strengths falling between the upper and lower bound models.

It should also be kept in mind that bearing is an odd limit state. If a connection is loaded to the bearing strength produced by the *Specification* when deformation at the bolt hole at service load is a design consideration, the result will not be a broken connection or even unrestrained deformations. It may simply be a connection with elongated holes. This adds some uncertainty to the issue of whether the lower bound model precisely predicts the actual strength of the connection rather than simply providing a conservative estimate.

It is also worth noting that the lower bound model assumes sufficient ductility exists. This primarily means that reasonable edge distances are provided. The spacing between bolts cannot drop below a certain threshold—setting aside the *Specification* requirements—due to the need to enter and tighten the bolts. Engineers are encouraged to use the edge distances provided in *Specification* Table J3.4, though the footnote permits smaller edge distances.

References

American Institute of Steel Construction, Inc. (AISC) (1923), *Standard Specification*, New York, NY.

American Institute of Steel Construction, Inc. (AISC) (1936), *Specification for the Design, Fabrication and Erection of Structural Steel Buildings*, New York, NY.

American Institute of Steel Construction, Inc. (AISC) (1978), *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings,* Chicago, IL.

American Institute of Steel Construction, Inc. (AISC) (1986), *Load and Resistance Factor Design Specification for Structural Steel Buildings,* Chicago, IL.

American Institute of Steel Construction, Inc. (AISC) (1999), *Load and Resistance Factor Design Specification for Structural Steel Buildings,* Chicago, IL.

American Institute of Steel Construction, Inc. (AISC) (2010), *Specification for Structural Steel Buildings,* Chicago, IL.

American Institute of Steel Construction, Inc. (AISC) (2016), *Specification for Structural Steel Buildings,* Chicago, IL.

Kim, H. J. and Yura, J. A., *The Effect of End Distance on the Bearing Strength of Bolted Connections*, PMFSEL Report No. 96-1, University of Texas, Austin, TX, 1996.

Lewis, B. E. and Zwerneman, F. J., *Edge Distance, Spacing, and Bearing in Bolted Connections*, Oklahoma State University, Stillwater, OK, 1996.