specwise

A TALE OF TEAR   OUTS

The strength of bolt groups or the *shear* delight of *bearing* change without *tearing* out your hair.

BY LARRY MUIR, PE

SECTION J3.10 of the 2016 *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) introduces a new limit state: tearout.

Actually, that's only half the story. More accurately, the section splits what had been presented in the 2010 *Specification* as a single check into two separate checks: bearing and tearout. There has been some confusion and controversy related to the proper application of this check that we'll attempt to clear up here. (*Note: For the sake of brevity, we have listed the bolt grades but not the full ASTM designation of F3125 Grades A325 and A490 throughout the text.*)

What is Tearout?

The limit state of bolt edge tearout was introduced in the 1999 *Specification* as part of the bolt bearing checks. Tearout is a limit state provided in Section J3.10 of the *Specification*. It is described in the Commentary as a bolt-by-bolt block shear rupture of the material upon which the bolt bears—a failure of the material in front of the bolt in the direction of the force. Though not a theoretically correct model, bolt tearout may be easier to understand if you think about a bolt tearing through the material (as shown in Figure 1). There are two shear planes. Assuming the planes shown, the strength is calculated as:

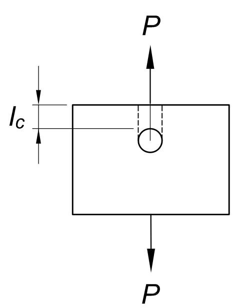


Figure 1

$$R_n = 2(0.6)l_c tF_u = 1.2l_c tF_u$$

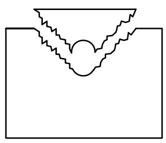


Figure 2

Though this is a simple and useful model, it does not reflect the actual behavior. If the bolt tears from the material, the phenomenon looks more similar to Figure 2. The model does, however, produce *Specification* Equation J3-6c, the nominal tearout strength when deformation at the bolt hole is a design consideration. The fact that the model is not precise is reflected by the fact that the *Specification* also presents a limit state for conditions when deformation at the bolt hole is not a design consideration with Equation J3-6d, $R_n = 1.5l_c tF_u$, which predicts a strength 25% higher than the Figure 1 model.

As described in the Commentary, when deformation at the bolt hole is a design consideration, the strength is limited such that hole elongation will not exceed ¼ in. when high tensile stress occurs on the net section. At this stress level, the bolt may not tear from the joint—but for simplicity, the limit state is still referred to as tearout.

Tearout can occur between a bolt and any edge, whether the edge occurs at the end of the material or at an adjacent bolt hole.

The Change

The change to the 2016 *Specification* is minor. Equation J3-6a in the 2010 *Specification* has been broken into two separate Equations, J3-6a and J3-6c, in the 2016 *Specification* (see Table 1). A similar change has been made to Equations J3-6b and J3-6d. This is intended to be an editorial change. The 2016 Commentary was also revised to provide further information and guidance.

Table 1. Comparison Between
2010 and 2016 *Specification* Tearout Provisions

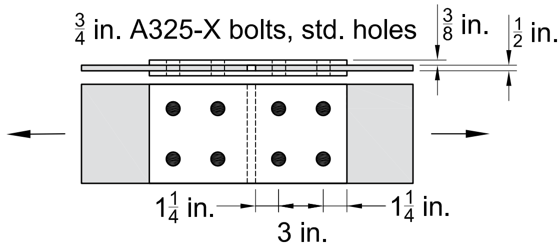
2010 *Specification*	**2016 *Specification***
$R_n = 1.2l_c tF_u \leq 2.4dtF_u$ (J3-6a)	$R_n = 2.4dtF_u$ (J3-6a) $R_n = 1.2l_c tF_u$ (J3-6a)

Lower Bound Method

As stated previously, there has been some confusion related to the proper application of this check. There are multiple approaches possible. However, the User Note in Section J3.6 of

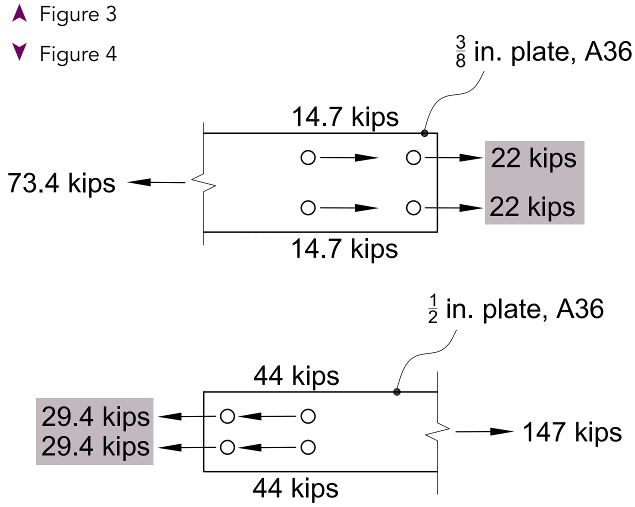
Larry Muir (muir@aisc.org) is AISC's director of technical assistance.

the 2016 *Specification* describes the preferred procedure. This same User Note appeared in Section J3.6 of the 2010 *Specification* as well. To illustrate, consider the connection with a top and bottom ⅜-in.-thick plate and a center ½-in.-thick plate as shown in Figure 3.



▲ Figure 3

▼ Figure 4



To save you from tearing your hair out, let's start with the outcome that tearout on the edges in both plates simultaneously controls—i.e., two bolts tear out of the loaded edge of the ½ -in. plate and the other two bolts tear out of the loaded edges of the ⅜-in. plates. The associated applicable nominal strengths are shown in the free body diagram in Figure 4. A free-body diagram showing the nominal strengths applicable, based on the User Note, is shown in Figure 4 (note that the governing forces are highlighted). To explain where this came from, there are 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 nominal single shear strength for a ¾-in.-diameter A325 bolt with the threads excluded from the shear plane is 30.1 kips/bolt.
2. The nominal bearing strength on the ½ in. plate is 52.2 kips/bolt.
3. The nominal bearing strength on each of the a in. plates can be found by prorating the strength of the ½ in. plate:

$$52.2 \text{ kips/bolt} \left(\frac{0.375 \text{ in.}}{0.5 \text{ in.}}\right) = 39.2 \text{ kips/bolt/plate}$$
$$\text{or } 78.4 \text{ kips/bolt}$$

4. The nominal tearout strength at the edge for the ½ in. plate is 29.4 kips/bolt.
5. The nominal tearout strength at the edge on each of the a in. plates can be found by prorating the strength of the ½ in. plate:

$$29.4 \text{ kips/bolt} \left(\frac{0.375 \text{ in.}}{0.5 \text{ in.}}\right) = 22.0 \text{ kips/bolt/plate}$$
$$\text{or } 44.0 \text{ kips/bolt}$$

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

The strength of the bolts at the inner bolt line is governed by the tearout strength at the edge for the ½-in. plate 2(29.4 kips) = 58.8 kips. Note this is less than the double shear value of 2(30.1 kips/bolt) = 60.2 kips and the bearing strength of 2(52.2 kips/bolt) = 104.4 kips.

The strength of the bolts at the outer bolt line is governed by the tearout strength at the edge for the ⅜-in. plate 2(22.0 kips) = 44.0 kips per shear plane. Note the tearout strength, 22 kips/bolt/plate, is less than the single shear value of 30.1 kips and the bearing strength of 39.2 kips/bolt/plate.

The total strength of the connection is 58.8 kips + 44.0 kips (2 shear planes) = 147 kips.

Poison Bolt Method

An alternate method, sometimes referred to as the poison bolt method, simply multiplies the least strength of any of the bolts by the total number of bolts. In this case the poison bolt method yields:

$$R_n = (4 \text{ bolts})(29.4 \text{ kips/bolt}) = 118 \text{ kips}$$

This is obviously significantly less work, but it results in about a 20% reduction in the predicted strength in this case. The underestimation of strength can be greater for some connections. This approach is not recommended.

Commentary Method

The Commentary to the 2016 *Specification* suggests a simplification for typical connections, such as those shown in the AISC *Steel Construction Manual*. The shear, bearing and tearout limit states for each bolt in the same connected part are determined and the lowest value summed to determine the strength of the group. This ignores the potential for interaction of these limit states among multiple connected parts, but the impact is small in common connection details. The key is that a "reasonable" connection is being considered, such as the example being considered here. 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*. The *Specification* does not prohibit the use of 1-in.-diameter A490-X bolts to connect ¼-in. material, but such an arrangement does not make a lot of sense, may not be economical and will present more of an issue relative to tearout and interaction between connected elements.

The Commentary simplification can be applied to the example. The strength based on bolt shear remains unchanged, 8(30.1 kips/bolt) = 241 kips. The bearing and edge bolt tearout strengths of the ½-in. plate was determined previously as 52.2 kips/bolt and 29.4 kips/bolt, respectively. The tearout strength between the bolts is 76.2 kips, and as is common in typical connections, it does not govern. By inspection, the limits states for the ⅜-in. plates do not govern. Therefore, the strength of the connection is:

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

The predicted strength, 163 kips, is higher than the 147 kips predicted by the User Note model but only by about 11%. We knew it would be higher, because it starts by assuming a failure mechanism instead of a force distribution; it is an upper-bound solution. As described in the 2016 Commentary, we have bounded the actual strength of the connection. A comparison of the various methods is presented in Table 2.

Table 2. Comparison of Methods

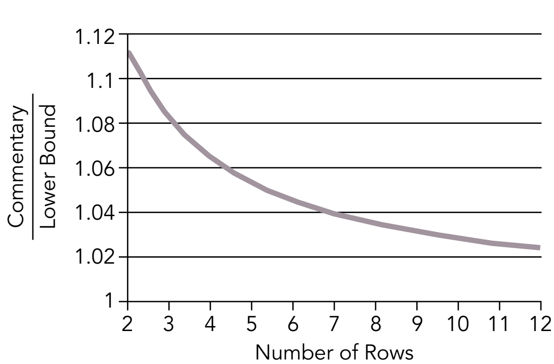
	Pre-1999	**Poison Bolt**	**Lower Bound***	**2016 Commentary**
Values (kips)	209	118	147	163
% of Lower Bound	142	80	100	111

*Per User Note in Section J3.6 in the 2010 and 2016 *Specification*.

Adding Bolt Rows

As is probably already clear, it is the way in which the tearout at the edge bolts is handled that is causing the difference between the various models. The effect of adding rows of bolts can be seen in Figure 5. The discrepancy between the methods drops off quickly as rows of bolts are added. This is consistent with assumptions, made as far back as at least 1936, that edge tearout is less of a concern for connections with multiple rows of bolts in the direction of the force.

Figure 5: Effect of number of rows.



Tearout Between Holes

Common connections typically provide for ¾-in. or ⅞-in. bolts spaced at 3 in. on center. Fortunately, for this common configuration, tearout is not a concern between the rows of bolts. For bolts larger than ⅞ in. in diameter, bolt shear, not tearout, will govern if the plate is made significantly thick. Table 3 presents the minimum thickness required to ensure that bolt shear (and not tearout) governs. The values assume either 3 in. spacing or the minimum allowed by the *Specification*: 2⅔ times the nominal diameter per Section J3.3. Single shear is also assumed.

Table 3. Minimum Thickness (inches) to Ensure Tearout Does not Govern Between Holes

Bolt Diameter (inches)	**A36**			**Grade 50**		
	F3125 Grade			F3125 Grade		
	A325-N	A325-X or A490-N	A490-X	A325-N	A325-X or A490-N	A490-X
1	⅜	7/16	9/16	5/16	⅜	½
1⅛	½	⅝	¾	7/16	9/16	⅝
1¼*	½	⅝	13/16	½	9/16	11/16
1⅜*	9/16	11/16	⅞	½	⅝	13/16
1½*	⅝	¾	15/16	9/16	11/16	⅞

*Spacing is 2⅔ times the nominal diameter.

A similar analysis can be performed to find the minimum thickness such that tearout does not govern given edge distances of 1¼ in. and 1½ in. The results are presented in Table 4.

Table 4. Minimum Thickness (inches) to Ensure Tearout Does not Govern at Edge

Bolt Diameter (inches)	**l_e = 1.25 in.**			**l_e = 1.5 in.**		
	F3125 Grade			F3125 Grade		
	A325-N	A325-X or A490-N	A490-X	A325-N	A325-X or A490-N	A490-X
½	3/16	¼	¼	3/16	3/16	¼
⅝	5/16	⅜	7/16	¼	5/16	⅜
¾	7/16	9/16	11/16	⅜	7/16	½
⅞	⅝	13/16	1	½	⅝	¾
1	15/16	1 3/16	1 7/16	11/16	⅞	1 1/16

What About the *Manual*?

The most widespread change to AISC's *Steel Constrcution Manual* (**www.aisc.org/publications**) involved simply adding the term tearout in the text to reflect the breaking up of the tearout and bearing limit states.

The most substantial change was made to Table 10-1, which included a tabulation of beam web available strength per inch of thickness. Due to the format of the table, neither the lower bound nor the 2016 Commentary alternative approach is possible. It was therefore decided that the best option was to simply remove these values. This leaves the designer the task of evaluating tearout. *Manual* Table 7-5 can aid in this task. Tables 3 and 4 in this article can also be used to determine when tearout will and will not be an issue.

Rules of Thumb and Helpful Hints

Keep these tips in mind when considering tearout:

- ➤ Tearout will not govern between the bolts for many common connections
- ➤ Tearout will not govern the strength of shear connections to uncoped beams with ⅞-in.- or 1-in.-diameter bolts and the edge distance, l_e , is equal to 1.25 in. or 1.5 in.
- ➤ Edge tearout will generally not govern if the thickness of the plies is equal to the bolt diameter
- ➤ Bolt grades and diameters should be well-matched to the strength of the plies
- ➤ Edge distance must be considered, though often tearout can be deemed okay by inspection
- ➤ In the 2016 *Specification*, the hole clearance increased to ⅛ in. for bolts 1 in. and larger in diameter. This will affect the clear distance and therefore the tearout strength
- ➤ The five-limit-state approach described in a User Note is the recommended design approach and is the one reflected in many AISC Design Examples and more recent AISC Design Guides
- ➤ The poison bolt model is conservative and might be sufficient in some instances but may not result in the most economical design
- ➤ The mechanism-based approach, as described in the Commentary, though tending to overestimate the strength may be sufficient for many common conditions

Additional information, including a history of edge distance checks in the AISC *Specification*, more detailed calculations related to the example problem, references and other information about bolt tearout can be found at **www.aisc.org/tearout**. And you can view and download the current version of the *Specification* at **www.aisc.org/specifications**. ■

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:

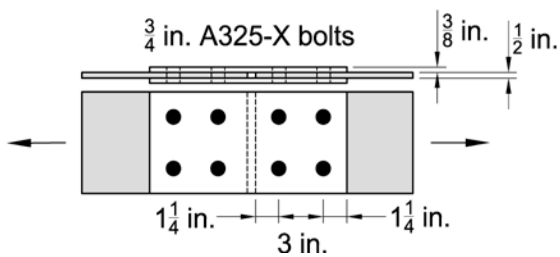


Figure 3

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):

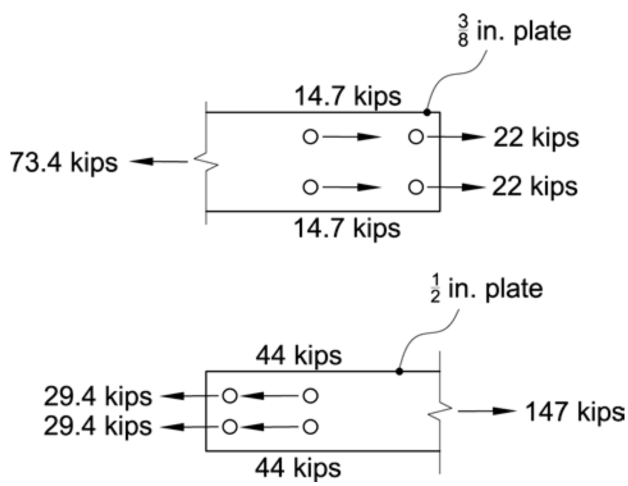


Figure 4

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.

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