## Guidance on Shear Rupture, Ductility and Element Capacity in Welded Connections

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## ABSTRACT

Several considerations need to be made while in the process of designing welds and welded connections. For the most part, the AISC *Specification for Structural Steel Buildings*, in combination with corresponding parts of the AISC *Steel Construction Manual*, provides fairly good guidance on what is required to design *Specification*-compliant welds. However, there seems to be some confusion and controversy in regard to a few of these considerations. Specifically: (1) When is the load path from the weld to the connecting element(s) unclear? (2) When should the ductility factor be applied to a weld? (3) When should a weld be sized to develop the strength of a connecting plate? This paper is written in an effort to provide guidance in regard to these three considerations. Background into the development of the equations used to make these checks along with some discussion on the intent of application is provided and supported with some anecdotal examples. It is the objective of the authors to shed some light on these issues and hopefully clear any confusion and/or controversy, as well as to encourage more consistency throughout the steel construction industry with regard to these three considerations.

Keywords: welded connections, shear rupture, ductility, element capacity.

## **INTRODUCTION**

n the authors' opinion, three of the most misunderstood and misapplied limit state checks in welded connection design are (1) application of matching fillet weld strength to base material strength when the load path within the base metal under the load is not readily known (AISC, 2017a, p. 9-5), (2) when to apply the ductility factor (1.25) when sizing a weld, and (3) when a weld should develop the strength of the connecting material. The objective of this paper is to shed some light on the development of the equations used to make these three types of checks, provide discussion in regard to the various applications of these checks, and present example problems demonstrating those applications.

Data related to every possible condition that might be encountered in practice simply are not available. In some instances, the authors are recommending practices based on their own knowledge, experience and judgment. Many of the recommendations are conservative, though considered reasonable by the authors. Sources of conservatism are noted. Though the authors often approached the topics discussed

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with divergent views, this paper represents a consensus of the authors.

## PART 1: MATCHING WELD AND BASE MATERIAL STRENGTHS

## Derivation

Part 9 of the AISC *Steel Construction Manual*, hereafter referred to as the AISC *Manual*, provides a brief discussion of how to address base material rupture strength at welds entitled, "Connecting Element Rupture Strength at Welds." The equations given in the AISC *Manual* for one- and two-sided fillet welds (Equations 9-2 and 9-3) are repeated here for convenience (see Equations 1 and 2, respectively).

$$t_{min} = \frac{3.09D}{F_u} \tag{1}$$

$$t_{min} = \frac{6.19D}{F_u} \tag{2}$$

In Equations 1 and 2, D is the weld size in sixteenths of an inch and  $F_u$  is the specified minimum tensile strength of the base material adjacent to the weld.

The derivations of Equations 1 and 2 are fairly straightforward. As already stated, the intent is to match the weld strength to the base material strength. In other words, ensure that the shear rupture strength of the base material is at least equal to the rupture strength of the weld.

Figure 1 shows a one-sided weld condition. As shown in Figure 1, the shear rupture plane in the weld is assumed to be along the throat of the weld as shown with solid shading.

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To ensure that the base material is at least thick enough to develop the rupture strength of the weld, a minimum thickness will be required for the rupture area of the base material shown in Figure 1 with cross hatching. We simply write equations to describe the shear rupture strengths of the two areas and set the two to equal each other. For the weld strength, the area is written in terms of the weld leg size in sixteenths of an inch (commonly known as *D*). Also, as can be seen in the derivation, the specified minimum tensile strength of the weld material,  $F_{EXX}$ , is assumed to be 70 ksi.

Note that the length of the weld, l, as shown in Figure 1 is not important because the derivation will show that the limit state checks given in Equations 1 and 2 are unit length checks.

The nominal shear rupture strength of the weld, based on the nominal stress,  $F_{nw} = 0.60F_{exx}$ , from AISC *Specification* Table J2.5 (AISC, 2016c), is given by Equation 3.

$$R_{nw} = 0.60 F_{EXX} \cos 45^{\circ} \left(\frac{D}{16}\right) l$$
  
If  $F_{EXX} = 70$  ksi  
 $R_{nw} = (0.60)(70) \cos 45^{\circ} \left(\frac{D}{16}\right) l$   
 $R_{nw} = 1.856Dl$  (3)

The shear rupture strength of the base material, from AISC *Specification* Section J4, is given in Equation 4 in terms of a minimum plate thickness.

$$R_{np} = 0.60 F_u l t_{\min} \tag{4}$$

Setting Equation 3 and 4 equal to each other then rearranging to solve for the minimum plate thickness,  $t_{min}$ , gives Equation 1.

$$R_{nw} = R_{np}$$

$$1.856Dl = 0.60F_u lt_{min}$$

$$t_{min} = \frac{1.856Dl}{0.60F_u l}$$

$$t_{min} = \frac{3.09D}{F_u}$$

Suppose the base material shown in Figure 1 has connecting elements on both faces, as shown in Figure 2. In this case, the shear rupture area of the base material does not change. However, the weld rupture area doubles. In this case, the weld shear rupture strength is as given in Equation 5.

$$R_{nw} = (2)0.60 F_{EXX} \cos 45^{\circ} \left(\frac{D}{16}\right) l$$
  
If  $F_{EXX} = 70$  ksi  
 $R_{nw} = (2)(0.60)(70) \cos 45^{\circ} \left(\frac{D}{16}\right) l$   
 $R_{nw} = 3.712Dl$  (5)

Setting Equations 4 and 5 equal to each other and then rearranging to solve for the minimum plate thickness,  $t_{min}$ , gives Equation 2.

$$R_{nw} = R_{np}$$

$$3.712Dl = 0.60F_u lt_{min}$$

$$t_{min} = \frac{3.712Dl}{0.60F_u l}$$

$$t_{min} = \frac{6.19D}{F_u}$$



Fig. 1. Weld and base material shear rupture area in a one-sided weld condition.

#### Discussion

In effect, Equations 1 and 2 ensure that the base material will not rupture in shear adjacent to the weld when the weld size used to calculate  $t_{min}$  is provided in the connection. But it is critical to understand how the weld size, D, is calculated in the derivation of Equations 1 and 2. If we examine Equation 3, we find that its derivation is the same as that for the fillet weld equations, Equations 8-2a and 8-2b, provided in AISC *Manual* Part 8. If we multiply Equation 3 by the LRFD  $\phi$  factor (0.75) or divide by the ASD  $\Omega$  factor (2.00), we get those two well-known equations. Equations 8-2a and 8-2b of the AISC *Manual* are repeated here for convenience. See Equations 6 and 7. Equation 5 is simply Equation 3 multiplied by 2, which is analogous to having two weld lines.

$$R_{nw} = 1.856Dl 
\phi R_{nw} = (0.75)(1.856)Dl 
\phi R_{nw} = 1.392Dl 
(6) 
R_{nw} = 1.856Dl 
 $\frac{R_{nw}}{\Omega} = \frac{1.856Dl}{2.00} 
\frac{R_{nw}}{\Omega} = 0.928Dl$ 
(7)$$

Equations 1 and 2 can be thought of as a shear rupture check for the base material. However, this is only accurate when the provided fillet weld size is exactly the size calculated from Equations 6 or 7—in other words, when the fillet weld is sized based on strength. If the provided fillet weld size is larger than that calculated using Equations 6 or 7, then Equations 1 and 2 will predict a required plate thickness larger than what is required by a factor of  $D_{prov}/D_{req}$ .

In simple terms, Equations 1 and 2 will result in a base material thickness that is able to develop the strength of the weld and is independent of the load required to be transferred by the weld.

#### Not Readily Known...

When is the shear rupture area not "readily known?" One way to look at this is that the demand on the base material is not readily known. When this is the case, Equations 1 and 2 provide a conservative approach to ensure that the base material adjacent to the weld, at every point along the length of the weld, is thick enough to develop the strength of the weld actually provided independent of the actual required load for which the weld was sized.

Another condition is when a weld group is loaded eccentrically and the weld size is determined using, for instance, the instantaneous center of rotation method. Figure 3 shows a W16×57 beam connected to a W14×90 column with a shop-welded/field-bolted, double-angle connection. The fibers of the web of the beam, adjacent to the weld, are subjected to a combination of shear and tensile stresses induced by the rotational demands inherent with the eccentric nature of the loading on the weld group. Therefore, the stresses in the beam web, adjacent to the weld, are not readily known, and the shear rupture check of the base metal provided by AISC *Specification* Section J4.2 cannot be applied directly.



Fig. 2. Weld and base material rupture area in a two-sided weld condition.

Again, in this case, Equations 1 and 2 can be used as a conservative approach to ensure that the base material adjacent to the weld, at every point along the length of the weld, is sufficient to develop the strength of the provided weld.

#### **Byproduct Use of Equations 1 and 2**

It is not unprecedented that Equations 1 and 2 have been used to check base material thickness in connections where shear rupture in the base material adjacent to the weld is not an applicable limit state. Figure 5 (discussed later) shows such a condition. Figure 4(a) shows an extended singleplate simple shear connection transferring load to the web of a wide flange column. There are various design example problems in AISC documents where Equations 1 or 2 are used to check the thickness of the column web; even though shear rupture in the column web is not an applicable limit state, the column is continuous past the extent of the plate connection. In the absence of an industry consensus approach to this problem, even though Equations 1 and 2 do not represent a viable limit state, they nevertheless give a conservative result.

There is a phenomenon that will occur as a result of this load transfer. Certainly, the column web will experience some shear stress but, more than likely, in combination with compression and tension stresses as the load, R, accumulates over the length (depth), l, of the plate. Figure 4(b) is a sketch of this possible phenomenon. In Figure 4(b), the load transferred from the plate to the column web may be some combination of the load hanging from the column web above the plate and pushing on the column web below the plate.



Fig. 3. Shop-welded/field-bolted, double-angle simple shear connection.



Fig. 4. Extended single-plate simple shear connection to column web.

How these stresses are actually distributed in the column web would be a function of the slenderness of the column web and the amount of stress present in the web as a result of loads applied from other sources.

Currently, the AISC *Specification* (AISC, 2016c) and the AISC *Manual* (AISC, 2017a) do not address this possible limit state. It is also important to recognize that, as far as the authors are aware, there are no case studies that have identified this as a problem. Regardless, there must be some phenomenon [similar to that shown in Figure 5(b)], occurring in the column web under this type of loading. Some designers, in an effort to address this in some manner, have used Equations 1 and 2 as a check on the web. Although, knowing the formulation of Equations 1 and 2, it is clear that Equations 1 and 2 do not address the phenomenon illustrated in Figure 4(b).

When a plate connection, like that shown in Figure 4, frames to only one side of the column web, Equation 1 has been used to check the column web thickness. When a plate connection frames to both sides of the column web, Equation 2 has been used. If a connection designer chooses to check the thickness of the base material for conditions like or similar to that shown in Figure 4, that is their preference; it certainly is conservative. However, they should consider that it is probably an opiate for the problem.

It should also be recognized that single-plate shear connections often employ a weld size equal to or greater than 5% of the plate thickness. Much of the weld size is intended to allow the plate to yield prior to weld fracture. Because steel does not generally fail in the through-thickness direction, the rupture strength of the web, if checked at all, should only be checked relative to the shear reaction. A check based on the weld size is too conservative.

## Recommendations

# Concentrically Loaded Longitudinal Welds or Weld Groups

- 1. When shear rupture of the base material adjacent to the weld is an applicable limit state, AISC *Specification* Section J4.2 should always be used.
- 2. If a longitudinal weld can be sized using Equations 6 and 7 (AISC *Manual* Equations 8-2a and 8-2b) and shear rupture of the base material adjacent to the weld is an applicable limit state, AISC *Specification* Section J4.2 should always be used.
- 3. Equations 1 and 2 (AISC *Manual* Equations 9-2 and 9-3) can always be used when shear rupture of the base material adjacent to the weld is an applicable limit state. Remember that it is directly a shear rupture check of the base material adjacent to the weld. However, it must be recognized that it is a conservative approach that may

result in thicker base material than what is actually required for the load being considered when the provided weld size is larger than that of the weld size required to transfer the load (e.g.,  $D_{prov}/D_{req}$ ).

#### **Eccentrically Loaded Welds or Weld Groups**

- 1. Welds of this nature are not wholly loaded along the longitudinal axis of the welds. For these welds or weld groups, the welds cannot be sized using Equations 6 or 7 (AISC *Manual* Equations 8-2a and 8-2b). Furthermore, the actual stresses in the base material adjacent to the weld are not readily known. As such, when rupture of the base material is an applicable limit state, Equations 1 and 2 (AISC *Manual* Equations 9-2 and 9-3) should be used. It should be recognized that this is a conservative approach, but the authors are not aware of a better alternative.
- 2. AISC *Specification* Section J4.2 applies but is not readily usable for eccentrically loaded welds.

## **Byproduct Use of Equations 1 and 2**

- 1. It is somewhat common to use Equations 1 and 2 (AISC *Manual* Equations 9-2 and 9-3) to check base material thickness when shear rupture of the base material adjacent to the weld is not applicable but no other known limit state check is available (like or similar to that shown in Figure 4).
- 2. Equations 1 and 2 can be used, as noted earlier, but these equations simply were not derived for such a purpose. The designer should recognize that such checks do not really address the issue.

## PART 2: THE DUCTILITY FACTOR; INTERFACE WELDS

## Background

The ductility factor for welds (some refer to this as the Richard factor), first showed up in AISC documents in the 1992 *Manual of Steel Construction*, Volume II: Connections (AISC, 1992). The ductility consideration arose during the development of the uniform force method (UFM), now commonly used for distributing forces in vertical brace connections framing to beam-column joints. One of the assumptions in the development of the UFM is that interface forces are distributed uniformly along the interfaces regardless of interface length, proximity of connected members, or other variables such as frame action (distortion).

Figure 5 shows a vertical brace connection used in a wind (or low seismic) application. Note the close proximity of the end of the brace relative to the beam-gusset interface.

Although the interface forces are assumed to be distributed uniformly, as shown in Figure 5(b), a stress (or force) concentration in the vicinity of the end of the brace may be present (as such, causing a nonuniform distribution of stress along the welded interface). It is for considerations such as this that the ductility factor was developed and implemented. It is worth noting that the work performed by Williams (1986) and Richard (1986) used in developing the ductility factor considered only braces that frame to beam-column joints.

The ductility factor was born from the work presented in Williams' (1986) dissertation [a summary of that work can be found in Richard (1986)]. Of the work presented in the Williams dissertation, 45 finite element (FE) specimens, similar to the configuration shown in Figure 6, were considered to be loaded to their "ultimate" load, and maximum stresses along the gusset-member interfaces were recorded. Figure 6 shows a copy of the plot provided in the Williams dissertation that plots the ratio of the maximum interface stress to the average interface stress for the 45 concentric connections considered. Table 1 presents the tabulated values illustrated in Figure 6.

The ductility factor equal to 1.40 used in the 1992 AISC *Manual* was determined by evaluating the maximum stress

ratio measured by the Williams' finite element analysis. As can be seen in Table 1, run (specimen) 26 has the largest reported value and is equal to 1.39. AISC simply rounded the number to 1.40.

Hewitt and Thornton (2004) subsequently performed statistical analysis on the data provided by Williams (see Tables 1 and 2) and recommended a reduced ratio equal to 1.25 based on a 90% confidence level (see Table 2). Note that, typically, this type of data and sample is evaluated on a 95% confidence level. As can be seen in Table 2, even at this confidence level, the upper bound is 1.26. Thus, even at a 95% confidence level, a ductility factor equal to 1.25 seems reasonable and is the value currently used for sizing welds.

How is the 1.25 factor used to accommodate ductility? Figure 7(a) shows the concentrated loads one might find to act on a welded interface. These interface loads are typically assumed to be uniformly distributed along the interface; the interface moment, M, is assumed to have a plastic stress distribution. The current method for determining whether or not the ductility factor should be used is to evaluate whether or not the peak stress/force,  $f_{peak}$ , is larger than 1.25 times the average stress/force,  $1.25f_{avg}$ , along the interface (the 1.25 coefficient is the value from the upper bound of the



Fig. 5. Corner brace gusset connection.

Table 1. Interface Stress Ratios Reported (Williams, 1986; Richard, 1986)												
Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio	Run	Stress Ratio			
1	1.22	10	1.22	19	1.16	28	1.20	37	1.16			
2	1.20	11	1.31	20	1.22	29	1.32	38	1.26			
3	1.19	12	1.19	21	1.19	30	1.18	39	1.20			
4	1.30	13	1.22	22	1.29	31	1.17	40	1.26			
5	1.29	14	1.24	23	1.32	32	1.18	41	1.30			
6	1.28	15	1.26	24	1.18	33	1.19	42	1.19			
7	1.19	16	1.32	25	1.33	34	1.27	43	1.22			
8	1.20	17	1.14	26	1.39	35	1.18	44	1.22			
9	1.29	18	1.26	27	1.37	36	1.12	45	1.30			

Table 2. Statistical Analysis of Interface Stress Ratios Given in Table 1												
	Standard	Confidence Level (90.0%)		Confidence Level (95.0%)		Confidence Interval (90.0%)		Confidence Interval (95.0%)				
Mean	Deviation	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper			
1.24	0.063	-0.0158	0.0158	-0.0189	0.0189	1.22	1.25	1.22	1.26			



Fig. 6. Reproduction of Williams' Figure 48 (maximum/average stress ratios).

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90% confidence interval shown in Table 2). Refer to AISC Manual Part 13 and Hewitt and Thornton (2004). To calculate  $f_{peak}$  and  $f_{avg}$ , refer to Figure 7(b), where  $f_{peak}$  is the resultant stress/force acting on the left half of the interface (where *m* and *a* act in the same direction) as shown in Equation 8.

$$f_{peak} = \sqrt{v^2 + (a+m)^2}$$
 (8)

The minimum stress/force,  $f_{min}$ , along the interface is on the right half of the interface where *m* and *a* act in opposite directions and is given in Equation 9.

$$f_{min} = \sqrt{v^2 + (a - m)^2}$$
(9)

The average stress/force is the average of  $f_{peak}$  and  $f_{min}$  as shown in Equation 10.

$$f_{avg} = \frac{f_{peak} + f_{min}}{2} \tag{10}$$

Figure 8 provides an illustration of the resultant forces obtained for the distribution shown in Figure 7(b). In the

comparison of  $f_{peak}$  and  $1.25 f_{avg}$ , one can infer that where  $f_{peak}$  is smaller than  $1.25 f_{avg}$  that the assumed uniform distribution is a reasonable assumption.

## Discussion

For corner gussets, the ductility factor is always used on the welded interface. The reason for this is primarily due to the effect of frame distortion on interface demands. So, regardless of proximity or connection geometry, a ductility factor is applied due to the consideration of frame distortion. However, application of the ductility factor is not necessarily required for welded interfaces used in other types of connections and deserves consideration of the types of loads, or combination thereof, acting on the interface, connection geometry, and the type of connecting element used. The following is a discussion of welded interfaces in other types of connections and the variation of combined loads and connection geometry.



(b) distributed interface forces





Fig. 8. Resultant interface loads, fpeak, fmin, and favg-

### Shear Only

In this case, there is no moment or axial (normal) forces acting on the interface. As such, the ductility factor is **not** applicable.

## Axial Loads

For Equations 8 and 9, the terms *v* and *m* are zero, and these equations reduce to those shown in Equations 11 and 12.

$$f_{peak} = \sqrt{v^{2} + (a+m)^{2}}$$

$$f_{peak} = \sqrt{0^{2} + (0+m)^{2}}$$

$$f_{peak} = m$$
(11)
$$f_{min} = \sqrt{v^{2} + (a-m)^{2}}$$

$$f_{min} = \sqrt{0^{2} + (a-0)^{2}}$$

$$f_{min} = a$$
(12)

Taking the average of Equations 11 and 12 gives that shown in Equation 13.

$$f_{avg} = \frac{f_{peak} + f_{\min}}{2}$$

$$f_{avg} = \frac{a+a}{2}$$

$$f_{avg} = a$$
(13)

Thus, for an interface subjected to pure axial force,  $1.25f_{avg}$  will always be larger than  $f_{peak}$ , suggesting that the ductility factor should always be applied to axial-only cases. However, we need to consider how the ductility factor was originally developed (as discussed previously) and the type of connection that is actually being considered. The discussion given in reference to Figure 5 suggested that proximity was an issue, and it probably is for the connection shown in Figure 5. Referring to Figure 5(a), it can be seen that the Whitmore spread does not engage the entire beam-gusset interface, suggesting that a stress concentration is likely to exist on the interface in the vicinity of the end of the brace-to-gusset connection.

Consider the hanger connection shown in Figure 9(a). The authors have seen the ductility factor applied to such a connection. One argument is that the end of the hanging member is in very close proximity to the welded interface, and as such, a ductility factor should be applied. However, if one is to look at the load transfer from the hanging member to the gusset, it can be reasonably argued that the axial force in the hanger is transferred along the hanger-to-plate welds along a sufficient length and that the connecting material is of nearly the same width as that of the hanging member. Therefore, a uniform distribution is reasonable to assume, and the weld ductility factor need not be applied for this condition. Considering a Whitmore spread of even, say, 10°, as shown in Figure 10, illustrates this claim. Note that, typically, a Whitmore spread is assumed to be effective at as much as



Fig. 9. Various plate interface connections.

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30°. The spread of the load in a uniform manner along the line of action of the hanger is analogous to an application of Saint Venant's principle.

Suppose, however, that the interface length shown in Figure 10 has to be increased, as shown in Figures 11(a) and 11(b), in order to accommodate a heavier load. For this case, the Whitmore spread, even considered to be effective at 45°, does not suggest that a uniform distribution will occur. The entire welded interface is not engaged in a manner. For this condition, it is recommended to assume that only the weld within the Whitmore spread projected on the interface is effective when designing the weld. For this approach, the ductility factor would not be applied.



Fig. 10. Axial hanger connection with flat bar plate.

If the designer chooses to use the entire interface length, including the portion outside of the Whitmore length, this is a rational approach to analysis which accounts for the nonuniform stress distribution.

#### **Combined Shear and Axial**

Under this loading, the *m* term in Equations 8 and 9 is zero, giving equations for  $f_{peak}$  and  $f_{min}$  as shown in Equations 14 and 15.

$$f_{peak} = \sqrt{v^{2} + (a+m)^{2}}$$

$$f_{peak} = \sqrt{v^{2} + (a+0)^{2}}$$

$$f_{peak} = \sqrt{v^{2} + a^{2}}$$
(14)
$$f_{min} = \sqrt{v^{2} + (a-m)^{2}}$$

$$f_{min} = \sqrt{v^{2} + (a-0)^{2}}$$

$$f_{min} = \sqrt{v^{2} + a^{2}}$$
(15)

Taking the average of Equations 14 and 15 gives that shown in Equation 16.

$$f_{avg} = \frac{f_{peak} + f_{min}}{2}$$

$$f_{avg} = \frac{\sqrt{v^2 + a^2} + \sqrt{v^2 + a^2}}{2}$$

$$f_{avg} = \sqrt{v^2 + a^2}$$
(16)

Because  $f_{peak}$  and  $f_{avg}$  are the same,  $1.25f_{avg}$  will always be larger than  $f_{peak}$ , suggesting that a ductility factor should always be applied for this type of loading.

Consider the flat bar used for the brace connection shown



Fig. 11. Fanned hanger connections—axially loaded interface welds.

in Figure 9(b). The force distribution at the welded interface is as shown in Figure 12(a). Figure 12(b) shows that if one were to assume a  $30^{\circ}$  Whitmore spread, it is clear that the load transfer, from the start of the connection, spreads through the gusset such that the entire length of the welded interface is well engaged. Therefore, application of the weld ductility factor is not required here.

Suppose that the plate used for the connection shown in Figure 12 is shaped to increase the interface length in order to satisfy a larger load. This configuration, as shown in Figure 13(a), is a commonly used detail. Figure 13(a) shows such a connection. Figure 13(b) shows a 30° Whitmore spread. As can be seen, the spread does not engage the entire welded interface. For this case, as shown in Figure 13, it would be appropriate to use a ductility factor. A simpler alternative might be to assume only the weld within the Whitmore length is effective without applying the ductility factor.

#### **Combined Shear and Bending**

Under this loading, the *a* term in Equations 8 and 9 is zero, giving equations for  $f_{peak}$  and  $f_{min}$  as shown in Equations 17 and 18.

$$f_{peak} = \sqrt{v^{2} + (a+m)^{2}}$$

$$f_{peak} = \sqrt{v^{2} + (0+m)^{2}}$$

$$f_{peak} = \sqrt{v^{2} + m^{2}}$$
(17)

$$f_{min} = \sqrt{v^2 + (a - m)^2}$$
  

$$f_{min} = \sqrt{v^2 + (0 - m)^2}$$
  

$$f_{min} = \sqrt{v^2 + m^2}$$
(18)

Taking the average of Equations 17 and 18 gives Equation 19.

$$f_{avg} = \frac{f_{peak} + f_{min}}{2}$$

$$f_{avg} = \frac{\sqrt{v^2 + m^2} + \sqrt{v^2 + m^2}}{2}$$

$$f_{avg} = \sqrt{v^2 + m^2}$$
(19)

Because  $f_{peak}$  and  $f_{avg}$  are the same, 1.25  $f_{avg}$  will always be larger than  $f_{peak}$ , suggesting that a ductility factor should always be applied for this type of loading.

However, consider the bracket connection shown in Figure 14. The bracket plate-to-column flange interface is subjected to shear and bending, very similar to how a chevron gusset interface is subjected to load. If we consider only the comparison of  $f_{peak}$  and  $1.25f_{avg}$ , a ductility factor would almost always seem to be necessary. However, one should consider how the ductility factor was developed. It was developed to address proximity and distortional effects on interfaces where a uniform stress distribution is assumed. The bracket connection shown in Figure 14 certainly does



Fig. 12. Flat bar brace connection—shear and axial interface loads.



Fig. 13. Flat bar brace connection—shear and axial interface loads.



(a) shaped gusset – force distribution

(b) shaped gusset – Whitmore spread

Fig. 14. Bracket plate connection (combined shear and bending).

not present issues related to proximity like that shown in Figure 5. In regard to distortional effects, the length ("h" as shown in Figure 14) of the interface for this type of connection is typically relatively short; therefore, any curvature in the column would be negligible in regard to distortional effects, so applying a ductility factor is not required. With these considerations, the authors argue that a ductility factor is not required on the interface weld for such a connection. However, if the length of the interface was to increase substantially, such curvature of the column could rationally be assumed to affect the stress distribution along the weld. Engineering judgment would be required when evaluating the interface stresses, and an alternative rational approach to analysis would be required.

## Combined Shear, Axial, and Moment

Under this loading, all of the terms in Equations 8 and 9 are nonzero, giving equations for  $f_{peak}$  and  $f_{min}$  as shown in Equations 20 and 21.

$$f_{peak} = \sqrt{v^2 + (a+m)^2}$$
(20)

$$f_{min} = \sqrt{v^2 + (a - m)^2}$$
(21)

Taking the average of Equations 20 and 21 gives Equation 22.

$$f_{avg} = \frac{\sqrt{v^2 + (a+m)^2 + \sqrt{v^2 + (a-m)^2}}}{2}$$
(22)

Because  $f_{peak}$  and  $f_{avg}$  are different, one would have to evaluate whether or not a ductility factor would be applicable.

Consider the chevron (i.e., midspan) gusset connection shown in Figure 9(c). As discussed in Fortney and Thornton (2015, 2017), the welded interface will always transfer a combination of shear and axial loads along with bending [refer to Figure 7(a)]. The shear and axial loads are typically assumed to be uniformly distributed along the interface, and the moment is assumed to be distributed as a plastic moment distribution as shown in Figure 7(b).

When a moment acts on an interface weld in this type of connection, the interface moment has traditionally been converted to an equivalent normal force and added to the calculated normal force. Figure 15(a) shows a representative sketch of a combination of shear, V, normal force, A, and bending, M, acting on an interface of length, L. The moment, M, is converted to a force couple, acting at L/4 from the centroid of the weld, representing a plastic stress distribution as shown in Figure 15(b). From here, one could size the welds on the left and right halves of the interface based on the



Fig. 15. Converting interface moment into equivalent normal force.

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resultant forces acting on the two halves of the interface.

The resultant force acting on the left half of the interface [see Figure 15(b)] is given in Equation 23.

$$R_{left} = \sqrt{(0.5V)^2 + \left(0.5A + \frac{2M}{L}\right)^2}$$
(23)

The resultant force acting on the right half of the interface [see Figure 15(b)] is given in Equation 24.

$$R_{right} = \sqrt{(0.5V)^2 + \left(0.5A - \frac{2M}{L}\right)^2}$$
(24)

Each of the resultant forces determined from Equations 23 and 24 would have different directional strength increase "coefficients" due to the different vector directions. One would simply provide a weld along the entire interface length equal to the larger of the two required welds; the weld size required for the left side of the interface, as shown in Figure 15(b), will typically govern the weld size. One can think of  $R_{left}$  as  $R_{peak}$  and  $R_{right}$  as  $R_{min}$ . This typically is not done, however. For further discussion on evaluating  $R_{peak}$ and  $R_{min}$ , refer to AISC Design Guide 29, *Vertical Bracing Connections—Analysis and Design*, Appendix B (Muir and Thornton, 2014), where it is shown that the maximum possible variance between  $R_{peak}$  and  $R_{min}$  (referenced as  $R_{plus}$ and  $R_{minus}$  in Design Guide 29) is 3.37%.

Typically, the force couple is converted into a total normal force and assumed to act in the same direction as the true normal force, A. These two forces would then be combined to give a total equivalent normal force, A (i.e.,  $N_{eq}$ ), as shown in Figures 15(c) and 15(d). The resultant force for the interface loads shown in Figure 15(d) is given in Equation 25.

$$R = \sqrt{V^2 + \left(A + \frac{4M}{L}\right)^2} \tag{25}$$

In Equation 25, the (A + 4M/L) term is the equivalent normal force. This method is used in several examples in the Design Examples Companion to the AISC *Steel Construction Manual* (AISC, 2017b) as well as AISC Design Guide 29.

It may not be immediately recognizable, but Equations 23 and 25 give the same resultant vector in both magnitude and direction. Thus, using the "equivalent normal force" method is another way of calculating the peak force (or stress if put in those terms). Where confusion seems to come into play with the "equivalent normal force" method is that there is no "average" stress with this method. So, how would one evaluate the need for using the ductility factor using this method? The answer is that this method is just another way of calculating the peak force (or stress). The average force/stress is still calculated using Equation 22.

It is important to mention that in most vertical brace connection example problems (braces framing to beam-column joints) presented in AISC documents, the equivalent normal force is used to calculate the peak force, and an average force is not calculated. However, a ductility factor is used on the resultant force calculated from the square root of the sum of the squares of interface shear and the equivalent normal force ( $f_{peak}$ ). Applying the 1.25 ductility factor to a resultant force determined using  $f_{peak}$  is not correct! This approach is taken simply as a conservative simplifying approach to avoid the trouble of calculating  $f_{avg}$ .

Should the ductility factor be applied to the welded interface for the connection shown in Figure 9(c)? Some would argue that it depends on how the force is spread from the start of the brace-to-gusset connection (i.e., the Whitmore spread) and through the gusset to the interface. Others argue that the ductility factor was developed for corner brace connections, and therefore, the ductility factor does not apply for this type of connection. The following discussion will demonstrate that the ductility factor is applicable for the connection shown in Figure 9(c).

### Whitmore Spread

If we assume that the welded interface shown in Figure 9(c) has a uniformly distributed load as a combined effect of both braces, then Saint Venant's principle should be evaluated. The evaluation can be done in terms of the Whitmore spread. Consider the midspan connection shown in Figure 16(a). Typically, the load effect of both braces would be assumed to be distributed uniformly along the entire welded interface. However, if we assume a Whitmore spread of 30° from the start of both connections, we see in Figure 16(a) that neither of the force spreads engage the entire interface. In this case, a ductility factor would be applied due to a proximity effect. However, if the force spread of each brace engages the entire interface length, as shown in Figure 16(b), then a ductility factor need not be applied due to a proximity effect.

## Interface Distortion

As discussed previously, the primary reason a ductility factor is applied to a weld at the welded interface of a corner gusset is because of frame distortion. That is, as the frame laterally displaces, the angle between the beam and column increases, thus applying a distortional tension on the gusset, or decreases, thus applying a distortional compression on the gusset. It is usually overlooked, but the gusset-to-beam interface in a midspan gusset is subjected to similar distortional forces.

Figure 17 shows a schematic of an exaggerated deflection of a braced frame beam with a midspan gusset. The beam tends to go through the textbook rotation of a simply supported beam loaded transversely. However, the load transfer among the beam, gusset and welds tends to "disturb" that typical beam rotation.



Fig. 16. Distribution overlap at interface.



Fig.17. Beam rotation relative to gusset along interface (deformation exaggerated for illustration).

Suppose the net transverse loads acting on the beam induce downward bending as shown in Figure 17. If a gusset is installed on the top flange, the edges of the gusset will tend to compress against the beam flange while the interface tends to open up along the interface length. Conversely, if a gusset is installed on the bottom flange, the end of the gusset will tend to move away from the flange while the interface will tend to close along the length of the interface. This curvature (greatly exaggerated in the figure for visual purposes) will create distortional forces along the welded interface. Therefore, regardless of Saint Venant's principle or the Whitmore spread, a ductility factor should always be applied for the type of connection shown in Figure 9(c).

## **Element Capacity Welds**

Some welds are designed to develop the tension, shear or flexural strength of the connecting element [e.g., single-plate shear connections, gusset-to-beam welds in corner gussets used in special concentrically braced frames (SCBF)]. When welds are sized to develop the strength of the connecting element, a ductility factor should not be applied.

## Recommendations

## Shear Only

A ductility factor is not applied to a weld under this type of loading.

#### Axial Only

For most typical connections, a ductility factor is not required. However, the spread of the load from the beginning of the load transfer point to the welded interface should be considered. If the Whitmore spread (or Saint Venant's principle) does not show that the entire interface length is engaged [e.g., Figure 11(b)], a ductility factor should be applied.

## **Combined Shear and Axial**

The use of a ductility factor needs to be evaluated on a caseby-case basis. For flat bar–type gussets, a ductility factor is typically not required. For other types of connections, the spread of the load from the beginning of the load transfer point to the welded interface should be considered. If the Whitmore spread (or Saint Venant's principle) does not show that the entire interface length is engaged [e.g., Figure 13(b)], a ductility factor should be applied.

## **Combined Shear and Bending**

Although a comparison of  $f_{peak}$  and  $1.25f_{avg}$  without considering proximity or distortional effects will almost always

suggest that a ductility factor should be applied, it would be rare for proximity or distortional effects to indicate the application of the ductility factor. For almost all cases, a ductility factor is not necessary. However, in rare cases—for example, in cases with relatively long interface lengths—one should use engineering judgment to determine if a ductility factor should be applied if distortional effects are present.

## Combined Shear, Axial and Bending

For most connections, an evaluation of Equations 20 and 22 should be performed to determine whether or not Equation 20 ( $f_{peak}$ ) is equal to or larger than 1.25 times Equation 22 ( $f_{avg}$ ). There are permutations of combinations of v, a and m that will show that the peak force/stress is larger than 1.25 times  $f_{avg}$ . However, if the welded interface can be reasonably assumed to be subjected to distortional forces (e.g., Figure 17), a ductility factor should be applied regardless of the evaluation of  $f_{peak}$  and 1.25 $f_{avg}$ .

#### Generally

It's important to note that one can simply always use a ductility factor. It will always be conservative; it just may not be necessary. However, with regard to welds designed to develop the strength of the connecting element, a weld ductility factor should not be used.

## PART 3: ELEMENT CAPACITY AND (%)*t*<sub>P</sub> WELDS

Generally, welds need only be designed to resist the loads transferred between the parts based on the structural analysis. Generally, welds need not be sized based on the available or expected strength of the joined parts. When welds are sized based on the strength of the joined parts, this is often referred to as "developing," as in "developing the plate" or "developing the strength of the beam."

One option is to provide a complete-joint-penetration (CJP) groove weld. As indicated in AISC *Specification* Table J2.5, at CJP groove welds "the strength of the joint is controlled by the base metal" not the strength of the weld. Partial-joint-penetration (PJP) groove welds with or without reinforcing fillet welds can also be used to develop steel elements. This discussion will concentrate primarily on the design of fillet welds used to develop steel elements, though CJP and PJP groove welds will be briefly addressed as well.

#### **Typical Conditions**

For a majority of conditions encountered in practice, a weld can be considered to develop the strength of the joined parts if the available strength of the weld equals or exceeds the least available strength of the parts joined.

#### Shear

The required weld size to develop a part subjected to shear can be determined by setting the available strength of the weld equal to the available shear yield strength of the part from AISC *Specification* Section J4.2. This is illustrated below using LRFD and Equation 6:

$$\phi 0.60 F_y A_g = 1.392 DL$$

Assuming a double-sided fillet weld gives Equation 26.

$$1.00(0.60)F_y t_p L_p = 1.392D(2)L_p$$
$$D = 0.216F_y t_p$$
(26)

A single-sided fillet weld could be used to develop the shear strength of an element. However, this is not a common practice and is typically uneconomical. Providing a doublesided fillet weld would be a much better detail.

#### Tension

The required weld size to develop a part subjected to tension applied transverse to the longitudinal axis of the weld can be determined by setting the available strength of the weld equal to the available tensile yield strength of the part, in accordance with AISC *Specification* Section J4.1. This is illustrated below using LRFD and Equation 6.

$$\phi F_v A_g = (1.5)1.392DL$$

Note that the 1.5 factor on the right side of the preceding equation is the directional strength factor as determined using AISC *Specification* Section J2.4.

Assuming a double-sided fillet weld gives Equation 27.

$$0.90F_y t_p L_p = (1.5)1.392D(2)L_p$$
$$D = 0.216F_y t_p$$
(27)

Single-sided fillet and PJP groove welds generally should not be subjected to tension applied transverse to the longitudinal axis of the weld because rotation can occur about the axis of the weld, placing increased and uncertain demand on the weld root. Where restraint prevents such rotation, the concern is less critical, and single-sided welds may be an option.

## Compression

The available strength of welds relative to compression load applied transverse to the longitudinal axis of the weld is generally assumed to be equal to that relative to tension load applied transverse to the longitudinal axis of the weld. There has been little testing of such conditions. There are reasons to believe that the strength of fillet welds subjected to compression will be greater than that for welds subjected to tension. Whereas applied tension will tend to open the root of the fillet, which is a stress riser, applied compression will tend to close the root of the weld, which is not a stress riser. There may also be bearing between the parts over some portion of the joint, which is generally neglected, and it should be neglected unless the parts are fit to bear. The authors recommend that the tension and compression cases be treated identically during design while recognizing that this is conservative.

## Bending

The intended meaning of "developing" the element can be less clear when related to bending. Various criteria can and are commonly used in design: elastic strength (first yield), plastic strength, and plastic strength with continued rotation. Both the elastic strength and the plastic strength conditions will be considered here. The condition of plastic strength with continued rotation will be addressed in a subsequent section.

From mechanics, the elastic strength of an element is determined from its elastic section modulus, *S*. The required weld size to develop the elastic strength of a part can be determined by setting the available strength of the weld equal to the available flexure strength of the part. This is illustrated in the following, assuming a double-sided fillet weld; using LRFD and a modified version of Equation 6 gives Equation 28.

$$\phi F_{y}S = (2)(1.5)1.392D \frac{l^{2}}{4}$$

$$0.90F_{y}S = (2)(1.5)1.392D \frac{l^{2}}{4}$$

$$D = 0.862 \frac{F_{y}S}{l^{2}}$$
(28)

In Equation 28, *l* is the length of the weld.

Equation 29 can be derived for a rectangular plate bent about its strong axis.

$$D = 0.144F_{\nu}t_{p} \tag{29}$$

Beyond first yield, the element will begin to lose stiffness, and further increases in applied load will tend to be attracted to stiffer, nonyielded portions of the structure. For this reason, sizing the weld to develop the elastic strength may often be sufficient. In some instances, it may be desirable to develop the plastic strength of the element.

Following a procedure similar to that illustrated for the elastic strength, the weld size required to develop the plastic strength of a rectangular plate bent about its strong axis gives Equation 30.

$$D = 0.216F_y t_p \tag{30}$$

#### Combined Shear, Axial and/or Bending

A similar procedure as those shown earlier will also result in a weld size of  $D = 0.216F_y t_p$  for combinations of applied shear, axial and/or bending.

For convenience, in practice it is useful to recognize that the weld size required to develop a plate for many of the loads considered thus far is  $D = 0.216F_y t_p$ . For a plate with a yield strength of 50 ksi, this can be expressed as  $w = 0.675t_p$ .

## Special Conditions

The procedures illustrated earlier can be used for many of the conditions most commonly encountered in practice that require development of the joined elements. There are, however, some instances where those procedures will result in weld sizes that are either larger than necessary or potentially ill-suited for the demands.

#### Single-Plate Shear Connections

AISC Manual Part 10 contains recommended design procedures for single-plate shear connections. For both the conventional and extended configurations, the AISC Manual recommends that "... the weld between the single plate and the support should be sized as  $(5/8)t_p$ , which will develop the strength of either a 36-ksi or 50-ksi plate..." The weld is sized such that the plate will yield prior to the weld fracturing, allowing the plate to act as a fuse that accommodates the beam end rotation in a ductile manner (Muir and Hewitt, 2009). It should be noted that this is only a recommendation. There is no provision in the AISC Specification requiring that the weld be stronger than the plate. Instead the  $(5/8)t_p$  recommendation is used as a means of satisfying AISC Specification Sections B3.4a and J1.2. AISC Specification Section B3.4a requires that "A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure." AISC Specification Section J1.2 requires that "Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam."

Rather than requiring engineers to determine the simple beam end rotation for every beam receiving a single-plate shear connection, the AISC *Manual* procedure is intended to accommodate rotations of about 0.03 rad, a rotation that exceeds the end rotation required of serviceable beams. In other words, the recommended ( $\frac{5}{8}$ ) $t_p$  weld size reflects a conservative simplification. It is important to note that this recommendation is only for single-plate simple shear connections at beam ends considered to have simple beam end boundary conditions. In other words, the recommendation, which results in a smaller weld than the more general procedure described earlier [i.e., ( $\frac{5}{8}$ ) $t_p$ ] applies only where the rotation is self-limiting and similar to a single-plate shear connection in configuration and expected behavior.

The design procedures for single-plate shear connections provided in the AISC *Manual* assume that plate yielding in some form accommodates simple beam end rotation. The conventional configuration relies on bolt plowing, or local yielding due to bearing at the plate (or potentially the beam web). The extended configuration primarily relies on flexural yielding of the plate, with bolt plowing considered in some cases. These are not the only mechanisms that can accommodate simple beam end rotation. In reality, a combination of mechanisms will be mobilized to accommodate the rotation. It may not be necessary to adhere to the (5%) $t_p$  weld size recommendation when other mechanisms are available or the simple beam end rotation from the analysis is small.

#### Large Inelastic Rotations—Seismic

In some instances, primarily related to seismic design, the weld must not only develop the flexural strength of the joined parts, but must also maintain its strength through large inelastic rotations of one of the parts joined.

One such condition involves the welds of gusset plates attaching vertical braces used in a SCBF. Per the AISC *Seismic Provisions* (AISC, 2016b), SCBF are "expected to provide significant inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension." When the buckling occurs out-of-plane, large inelastic rotations occur about approximately the longitudinal axis of the weld group, which could lead to premature rupture of the weld. AISC *Seismic Provisions* Section F2.6c.4 is intended to address this concern and states, "For out-of-plane brace buckling, welds that attach a gusset plate directly to a beam flange or column flange shall have available shear strength equal to  $0.6R_yF_yt_p/\alpha_s$  times the joint length." Even with the inclusion of  $R_y$ , the required weld size is still ( $\frac{3}{4}$ ) $t_p$ .

The thickness of the gusset plate is rarely governed by the demands at the welded interfaces. Developing the weakaxis flexural strength based on the full thickness of the gusset is not often necessary. A smaller, more economical weld can sometimes be obtained by sizing the weld to develop the maximum weak-axis moment occurring in combination with the shear, compression, and strong-axis moment that result on the gusset plate edge from the brace compression force. Carter et al. (2016) developed such a method utilizing a generalized interaction equation recommended by Dowswell (2015).

Other situations where welds are required to develop the strength of the joined part while that part undergoes large inelastic rotations are the moment connections in intermediate moment frames (IMF) and special moment frames (SMF). The AISC *Seismic Provisions* require physical testing of the beam-to-column connections to confirm the strength and ductility of such connections. Either prequalified connections provided in AISC 358, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2016a), or connections qualified per AISC Seismic Provisions Section K2 can be used. In either case, the suitability of the welds, though sometimes sized by calculation, is ultimately established empirically.

## **Exceptions to the "Rules"**

As stated in the Introduction to this paper, in some instances the authors are recommending practices based on their own knowledge, experience and judgment. Many of the recommendations are conservative. Over the decades, engineering judgment and common sense have dictated the use of certain practices or assumptions. Common engineering practice tends to, as it should, err on the side of conservatism. Apportioning of loads like "stress cops" is much maligned, though ultimately, all practical design is based on flawed assumptions. Limit states are sometimes segregated into ductile and nonductile categories. However, even though arguably the least ductile element commonly encountered in structural steel design, welds and welded joints often demonstrate ductility disproportionately greater than our common assumptions would suggest. A couple of illustrative examples will be discussed.

It is commonly assumed that the entire section must be engaged in order to develop the strength of the element. However, direct-welded, beam-to-column moment connections provide a counterexample to this common assumption. These connections are typically designed based on the assumption that the web connection carries the entire shear force and the moment is resolved into a couple with a lever arm equal to the distance between the flange centroids. There have been many tests of direct-welded, beamto-column moment connections loaded to failure under monotonic and cyclic loading, and the specimens generally had a final failure mode of tension flange rupture with the applied moment consistently exceeding the plastic moment capacity of the beam calculated with the yield strength from tensile coupon tests (Dowswell and Muir, 2012).

The authors do not provide these examples to support more general changes to standard practice but, instead, provide them to guard against the tendency to heap ever-moreconservative and onerous requirements on conditions that are likely to perform far better than we typically assume.

## Ductility Factor and Welds Sized to Develop Connected Part

As discussed previously, the ductility factor is used to enhance the ductility of the weld relative to proximity and distortional effects. When developing the joined parts, such enhancement of the weld is unnecessary because it is the yielding of the joined part(s) that provides the ductility and associated redistribution of the stress. The ductility factor should not be applied when the weld develops the joined part(s).

## CONCLUSIONS

Discussion of three common connection design applications has been provided. Little background into the evolution of issues related to connecting element rupture strength at welds, the weld ductility factor, and element capacity welds is readily available in archival journals. The authors have attempted to provide insight into the backgrounds of these limit state evaluations.

The authors make recommendations on analysis and design approaches when dealing with the connection design issues discussed. These recommendations are based on the collective experience of the authors. Readers should not interpret these recommendations as the only approaches that can be used. Any rational method of analysis or rational approach can be implemented. It is the intention of the authors that the discussion and background information provided offers additional insight that can be used when considering the three issues presented in this paper.

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