AISC LRFD Rules for Block Shear in Bolted Connections—A Review

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INTRODUCTION

The AISC LRFD specification (LRFD, 1999) for the design of steel structures contains provisions for what is customarily termed *block shear*. Notionally, this refers to the displacement of a block of material. It is usually associated with bolted details because a reduced area is present in that case, but in principle it can also be present in welded details. This paper deals only with bolted connections, however.

An example of what is used frequently to illustrate block shear is the gusset plate connection shown in Figure 1(a). It will be put forward later in this paper that the type of failure implied in this sketch does not in fact correspond to conditions at the time the ultimate load is reached. Another important case involving block shear is that of the web in a coped beam, as shown in Figure 1(b). The type of failure conventionally associated with this case will also be discussed.

The AISC LRFD rules use the relationship that shear yield and shear ultimate stress can be represented using the von Mises criterion, i.e., $\tau_y \approx 0.6\sigma_y$ and $\tau_u \approx 0.6\sigma_u$. The design rules are as follows:

$$\text{if} \quad \sigma_u A_{nt} \geq (0.6\,\sigma_u) A_{nv}$$
$$\text{then} \quad R_n = \sigma_u A_{nt} + (0.6\,\sigma_y) A_{gv} \tag{1}$$

$$\text{and if} \quad (0.6\,\sigma_u) A_{nt} > \sigma_u A_{nt}$$
$$\text{then} \quad R_n = (0.6\,\sigma_u) A_{nt} + \sigma_y A_{gt} \tag{2}$$

where
σ_y = tensile yield strength
σ_u = tensile ultimate strength
τ_y = shear yield strength
τ_u = ultimate shear strength
A_{nt} = net area subjected to tension
A_{nv} = net area subjected to shear
A_{gt} = gross area subjected to tension
A_{gv} = gross area subjected to shear
R_n = nominal block shear resistance

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The AISC LRFD rules are written in Section J4.3 of the Specification (where the nomenclature $F_u \equiv \sigma_u$ and $F_y \equiv \sigma_y$ is used). Of course the nominal strength given by Equations 1 or 2 must be multiplied by a resistance factor to obtain the design strength. The value used in the LRFD *Specification* for block shear is 0.75. The AISC LRFD rules also stipulate an upper limit, namely, the sum of tension rupture on the net tension area and shear rupture on the net shear area.

Equation 1 says that if the ultimate tensile resistance is greater than the ultimate shear resistance, then the block shear resistance of the connection is the sum of the tensile resistance (on the net section) and the shear yield resistance (on the gross shear area). Conversely, if the ultimate shear resistance is greater than the ultimate tensile resistance (Equation 2), then the block shear resistance of the connection is the sum of the ultimate shear resistance (net shear area) and the tension yield force (gross cross-section).

In themselves, the strength statements presented by Equations 1 and 2 are plausible. It is reasonable to think that the capacity in block shear could be the fracture tensile strength in combination with the shear yield strength (Equation 1). However, the possibility of attaining the shear ultimate strength in combination with the tensile yield strength seems unlikely. This requires that the ductility of the material in tension be sufficient to allow shear fracture to be reached. Examination of the test results in the case of the gusset plates presented herein shows that this is in fact the situation—there is not sufficient tensile ductility to permit shear fracture to occur.

The *Commentary* to the *Specification* says that the larger of Equations 1 and 2 is to be taken as the governing value and it provides a rationale for this rule. However, the comment seems to belong to the block shear rules of an earlier edition of the specification (LRFD, 1986). At that time, the same equations as given here as Equations 1 and 2 were presented (in the Commentary), and the user was advised to use the larger of the two results. In the 1999 (and also in 1993) LRFD rules, the qualifying statements that now precede Equations 1 and 2 means that the "larger" choice is no longer appropriate.

A review of the test results indicates that the failure modes seen in two important categories, gusset plate connections and the web of coped beams, are significantly different. The use of Equations 1 and 2 does not provide good predictions of the test results in either of these cases. Furthermore, examination of the test data upon which the rule is founded reveals that rupture or tearing of a *block* of material is not present at the time the ultimate load of a connection critical with respect to block shear is reached.

GUSSET PLATE TESTS

There are a large number of gusset plate tests reported in the literature for which block shear is the failure mode. Table 1 shows the results of 109 of these, taken from five different sources—Hardash and Bjorhovde (1985), Rabinovitch and Cheng (1993), Udagawa and Yamada (1998), Nast, Grondin, and Cheng (1999), and Swanson and Leon (2000). In addition, in their paper Hardash and Bjorhovde reported on a total of 14 more tests, from three other sources, which are not included here: the data pool is sufficiently large without these additional tests.

All the gusset plate tests show that the ultimate load is reached when the tensile ductility of the gusset plate material at the first (i.e., inner) transverse line of bolts is exhausted. (This mode of failure was also observed in the 14 additional tests cited in the Hardash and Bjorhovde study.) This was true even in cases where oversize holes were used and in cases where the connection was short (i.e., not much shear area available). The tests show that fracture at the net tension section is reached before shear fracture can take place on the other surfaces, i.e., tensile fracture (net section) plus some shear yielding takes place. The displacement of a block of material is seen only when the test is continued until the parts separate, and this occurs after the ultimate strength of the connection has been reached.

Use of Equations 1 and 2 gives conservative predictions of gusset plate nominal strength (resistance factor taken as unity) by 20 percent for the 109 tests listed in Table 1.

COPED BEAM TESTS

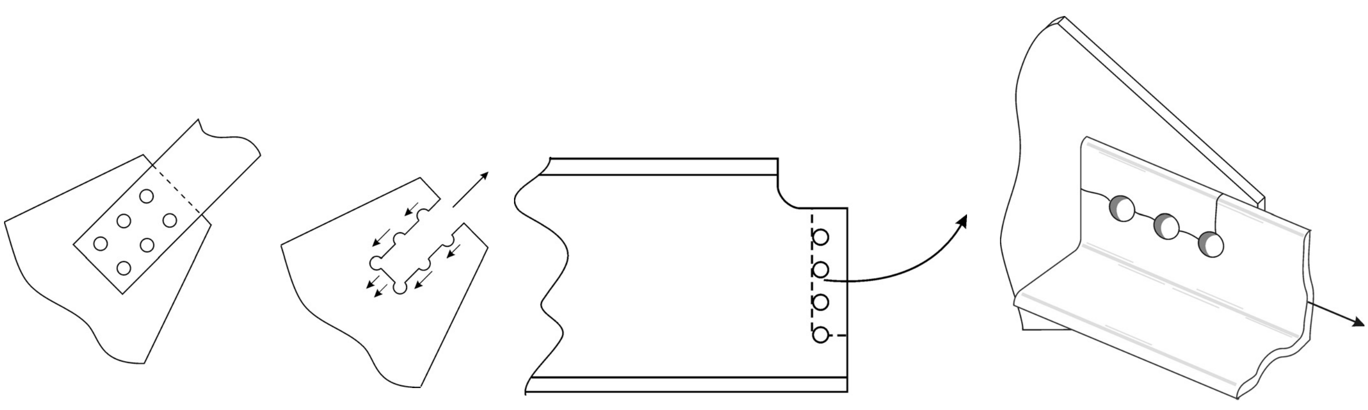
In contrast to the number of gusset plate tests available, there are not many tests of coped beams. Table 1 shows that there is a total of 21 tests. Seven of these involved connections with two lines of bolts and the other 14 had a single line of bolts. Two tests were for beams that had slotted holes.

The ratio of test ultimate load to the LRFD predicted nominal strength (i.e., $\phi = 1.0$) is non-conservative for three of the four series. In the test series that used two lines of bolts (Ricles and Yura, 1983), the ratio was significantly non-conservative (0.70). Although the test to LRFD predicted ratio in the Yura et al. series was only marginally less than unity, the standard deviation in this case is large. The test results presented by Aalberg and Larsen are the only results on coped beams that are predicted conservatively using AISC LRFD rules. It is clear that use of the AISC LRFD rules for block shear is not satisfactory for the important case of coped beams.

ANGLE TESTS

Figure 1 (c) shows a single angle connected to a gusset plate. Experience and test results show that block shear is potentially a failure mode for angles, particularly when the connection is short.

Epstein (1992) reported the results of a large number of tension tests for pairs of angles connected by bolts to a gusset plate passing between the angles. A total of 114 tests were conducted on 38 different configurations. The number of variables was large—size of outstanding leg as compared with connected leg, connection geometry (including bolt stagger and pitch, angle size, eccentricity of load, and so on)—and this led to different modes of failure. The failure modes included block shear, net section rupture, bolt shear, and various combinations of these. In only 15 individual



(a) gusset plate (b) coped beam (c) angle

Fig. 1. Examples of block shear.

Table 1. Block Shear Examination for Selected Cases					
Source	**Type**	**No. Tests**	**Test / LRFD**	**Test / Proposed**	**Comments**
Hardash and Bjorhovde (1985)	Gusset Plates	28	1.22 (0.08)	1.16 (0.06)	Hardash and Bjorhovde also reported on three other test series totaling 14 tests. These are not included herein.
Rabinovitch and Cheng (1993)	Gusset Plates	4	1.22 (0.07)	1.01 (0.06)	
Nast, *et al.* (1999)	Gusset Plates	3	1.35 (0.02)	1.04 (0.01)	
Udagawa and Yamada (1998)	Gusset Plates	73	1.18 (0.05)	1.04 (0.09)	Out of 219 test specimens, 73 failed by block shear. Three steel grades were investigated.
Swanson and Leon (2000)	Gusset Plate	1	1.18	0.97	
Ricles and Yura (1983)	Coped Beams	7	0.70 (0.09)	1.00 (0.13)	Two lines of bolts, number of bolts per line not necessarily equal, one test used slotted holes.
Aalberg and Larsen (2000)	Coped Beams	8	1.13 (0.08)	1.04 (0.18)	One line of bolts.
Yura *et al.* (1982)	Coped Beams	5	0.99 (0.19)	1.19 (0.19)	One line of bolts, one test used slotted holes, bolts spread over most of the beam web depth. Note large standard deviation associated with mean value.
Birkemoe and Gilmor (1978)	Coped Beam	1	0.95	1.17	One line of three bolts. Connected depth relatively small portion of beam depth.
Orbison *et al.* (1999)	Single Angles and Tees	12	1.12 (0.07)	1.06 (0.05)	One line of bolts.
Gross *et al.* (1995)	Single Angles	13	0.96 (0.05)	0.90 (0.07)	One line of bolts.
Barthel *et al.* (1987)	Single Angles	13	1.01 (0.01)	0.87 (0.04)	One line of bolts.
Epstein (1992)	Double Angles	3	0.80	0.80	Two lines of bolts. The standard deviation is not reported because only the average of the triplicate tests was available.

Notes: 1. Values of the ratio Test/LRFD greater than unity are conservative. LRFD value is calculated using resistance factor equal to 1.0.
2. Number in parentheses is standard deviation.

tests (three tests each of five series) was it reported that block shear was the sole failure mode. Of these 15 tests, in only three was there no stagger between the two bolt lines. In all other cases the stagger present introduces another parameter in the strength equations. Consequently, it was the decision of the writers that only the non-staggered cases be included in the block shear examination.

Other test programs have also investigated block shear in single angle connections and structural tees with one line of bolts (Gross, Orbison, and Ziemian, 1995; Orbison, Wagner, and Fritz, 1999; Barthel, Peabody, and Cash, 1987). Tests on structural tees have been used to assess the effect of out-of-plane eccentricity inherent with bolted angles (Orbison et al., 1999). Block shear failure of angle sections can be affected by out-of-plane and in-plane eccentricity. Although Orbison et al. (1999) found that out-of-plane eccentricity was not a significant factor, Epstein (1992) concluded that the factor had to be considered in block shear calculations: in-plane eccentricity was found to be a significant factor. Tests at Bucknell University (Gross et al., 1995; Orbison et al., 1999) have shown that the block shear capacity decreases with an increase in eccentricity.

With the exception of the test results presented by Epstein (1992), the block shear capacity is predicted well by Equations 1 and 2. The equations overestimate Epstein's test results by 20 percent. Depending on the magnitude of in-plane eccentricity, the predictions can be either conservative (average test-to-predicted ratios of 1.12 and 1.01 observed by Orbison et al. (1999) and Barthel et al. (1987), respectively) or non-conservative (average test-to-predicted ratios of 0.96 and 0.80 observed by Gross et al. (1999) and Epstein (1992), respectively). It is not clear why the test results by Epstein are overestimated by as much as 20 percent. Epstein has suggested that the correction for shear lag effect described in Section B3 of the AISC LRFD *Specification* should be applied to block shear failure. The writers believe that the use of the shear lag correction factor is not appropriate for block shear calculations since the failure plane in a block shear failure goes through only the component of the cross section that is connected. When considering angle and tee sections, the effect of out-of-plane load eccentricity may not be negligible. The current AISC LRFD design rules for block shear failure were derived from tests where eccentricity either was not present (e.g., gusset plate tests) or where eccentricity was strictly in-plane (e.g., tests on coped beams). Further research is required to investigate the effect of out-of-plane eccentricity on block shear failure.

IMPROVED DESIGN EQUATIONS

Gusset Plates

A good predictor of the ultimate strength of a gusset plate connection is obtained by adding the ultimate tensile strength (net tensile area) and the shear yield strength (gross shear area). This brings the predicted capacity much more closely into line with the test values. Furthermore, the predicted and observed failure modes are consistent. For the 109 gusset plate tests reported in Table 1, the ratio Test/LRFD is 1.07, standard deviation 0.08.

For an even better estimate of strength, the proposal made in Hardash and Bjorhovde (1985) can be used. The model proposed by these researchers uses net section tensile strength plus a shear strength component that reflects connection length. In the limit, short connections, the contribution from shear is nearly the same as that suggested here, i.e., shear yield acting on the gross shear area. It is the opinion of the writers that the existing AISC LRFD rule, Equations 1 and 2, is not a satisfactory model. It gives answers that are too conservative and it uses a failure mode (Equation 2) that is not demonstrated in the tests.

Coped Beams

The mode of failure in coped beam webs is different than that of gusset plates. Because the shear resistance is present only on one surface, there must be some rotation of the block of material that is providing the total resistance. Although tensile failure is observed through the net section on the horizontal plane in the tests, as expected, the distribution of tensile stress is not uniform. Rather, higher tensile stresses are present toward the end of the web. The prediction of capacity given by Equations 1 and 2 is significantly non-conservative when there are two lines of bolts present. If only one line is present, then the prediction is non-conservative for at least some cases.

As already noted, there are relatively few test results for block shear failure in coped beams. However, for these tests a satisfactory model is obtained using a capacity equal to one-half the tensile fracture load (net section on the horizontal plane) plus the shear yield load (gross section on the vertical plane). The test-to-predicted ratios and standard deviation obtained using this model are presented in Table 1. This model was first suggested by Yura and Ricles (1983). In addition, care should be taken to use generous end distances, particularly when slotted or oversize holes are present or when the bolts are distributed more-or-less from the top of the web to the bottom. If the latter detail is used, the bolt arrangement can carry appreciable moment and bolt forces may produce splitting between the bolts and the end of the beam web.

Angles

As was the case for the coped beam connection, the shear resistance for an angle is present only on one surface of the potential block of sheared material. There are a number of complicating factors present in the angle connection as compared with the coped beam, however. It is found that the failure model suggested for gusset plates, i.e., tensile fracture of the tension surface followed by shear failure along the shear surface gives reasonably good results. This is also consistent with what is observed in the majority of tests. For the 41 test results on angles and structural tees presented in Table 1, the test-to-predicted ratio using the same rule as has been suggested here for gusset plates results in a value of 0.93 for this ratio, with a standard deviation of 0.05.

As seen in Table 1, the existing LRFD rules (Equations 1 and 2) give better agreement with the tests than the model recommended here. However, the writers consider that these equations do not provide a rational explanation for how the block shear phenomenon actually takes place.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This examination has identified that use of the LRFD Specification rules for block shear leads to conservative designs for gusset plates, non-conservative designs for coped beams, and satisfactory results for angles. Importantly, in most cases these rules do not reflect the mode of failure observed in the tests.

The writers recommend that the following equations for calculation of block shear resistance be used:

Gusset plates, angles: $R_n = A_{nt}F_u + 0.6F_y A_{gv}$ (3)

Coped beam webs: $R_n = 0.5A_{nt}F_u + 0.6\ F_y A_{gv}$ (4)

A resistance factor must be applied to Equations 3 and 4. The value $\phi = 0.75$ is used in the current LRFD formulation. Although it is likely a conservative choice, further work must be done in order to establish a more appropriate value.

This review has identified the need for further studies of block shear. Work currently underway at the University of Alberta includes numerical modeling of the block shear resistance of gusset plate and angle connections and physical testing and numerical modeling of block shear in coped beams. The physical testing of coped beams will be done under controlled beam rotations, a feature not present in most of the tests reported in the literature.

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