

Effective Shear Plane Model for Tearout and Block Shear Failure of Bolted Connections

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ABSTRACT

In spite of many revisions to the block shear requirements of the AISC *Specification*, the model in the current *Specification* can result in calculated strengths and failure modes that are inconsistent with published test data. The inconsistencies are primarily related to the assumed interaction of tensile and shear resisting mechanisms, combined with the definition of net and gross shear planes that are unrealistic. Using recently published test results of single-bolt connections in mild and high-strength steel plates, the shear failure planes are observed to be neither the assumed net nor gross shear planes, which are the basis of the current design provision, but rather effective shear planes with a calculated area that is between the net and gross areas. Based on the tensile rupture and shear yielding mechanism, and assuming that the steel on the effective shear planes is fully strain hardened, a simpler and more accurate block shear design equation is proposed. The new equation is straightforward to implement as it requires a simple rearrangement of existing design variables to determine an effective shear failure area. Through verifications against 161 gusset plate specimens, tested by independent researchers around the world, the proposed equation is shown to be significantly more accurate than the current AISC, Canadian, European and Japanese block shear design provisions. A resistance factor of 0.85 is recommended for use with the proposed equation, based on the available statistics from tests and well-established LRFD reliability principles. An example is presented to illustrate the impact of the proposed design provision, which can result in significantly fewer bolts per connection and/or smaller gusset plates, leading to simpler and more economical designs.

Keywords: block shear, bolted connections, gusset plates, shear-out, shear planes, tearout.

INTRODUCTION

The block shear failure mode of bolted connections was first identified by Birkemoe and Gilmer (1978) and was incorporated in the 1978 AISC *Specification for Structural Steel Buildings* (AISC, 1978), which was still in the allowable stress design (ASD) format. Since then, the design provision to check block shear failures of bolted connections changed with every edition of the load and resistance factor design (LRFD) *Specification* until 2005, as summarized in Table 1 for concentrically loaded gusset plates. These changes were primarily motivated by ambiguities regarding the interaction of tension and shear behavior on assumed gross or net yield and rupture planes. In all of these specifications, there has been a presumption of yielding on gross areas and rupture on net areas, where the gross and net areas are A_{gt} and A_{nt} for tension and A_{gv} and A_{nv} for shear, and the corresponding stress limits for yielding and rupture are the tension yield stress, F_y , and ultimate stress, F_u . As will

be described later, these basic assumptions about characterizing behavior are much the reason for the perennial debate about block shear design provisions.

It may be noted that since the 2005 *Specification* (AISC, 2005), there is a nonuniform stress distribution factor, denoted U_{bs} , applied to the tensile strength component, $F_u A_{nt}$, of the block shear resistance. This reduction factor is equal to unity in most cases, including a concentrically loaded gusset plate, and is therefore not shown in the equations contained in this paper.

The absence of changes in the block shear design provision since 2005 masks a curiosity of the current provision (AISC, 2016), which suggests that the load required to fail a bolted connection by simultaneous tensile and shear ruptures can be lower than that required for the tensile rupture and shear yielding mechanism. The practical outcome of this incongruence is that the design provision can underestimate the actual block shear strength by almost 20% on average and up to 40% in certain cases.

There are several reasons for the repeated amendments and for the fact that the latest design provision is still substantially inaccurate, even though it represents an improvement over earlier provisions. This paper reviews the evolution of the block shear design provisions in the AISC *Specifications* since the first LRFD edition (AISC, 1986). Based on physical reasoning, the authors contend that the underlying premises of the *Specification* equations are incorrect. In particular, the available evidence suggests that the shear failure planes

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Table 1. AISC Specification Block Shear Design Provisions, 1978–2016

1978	$R_n = F_u A_{nt} + 0.6 F_u A_{nv}$
1986	$R_n = \max(F_u A_{nt} + 0.6 F_y A_{gv}; F_y A_{gt} + 0.6 F_u A_{nv})$
1993	If $F_u A_{nt} \geq 0.6 F_u A_{nv}$: $R_n = F_u A_{nt} + 0.6 F_y A_{gv}$ If $F_u A_{nt} < 0.6 F_u A_{nv}$: $R_n = F_y A_{gt} + 0.6 F_u A_{nv}$
1999	If $F_u A_{nt} \geq 0.6 F_u A_{nv}$: $R_n = \min(F_u A_{nt} + 0.6 F_u A_{nv}; F_u A_{nt} + 0.6 F_y A_{gv})$ If $F_u A_{nt} < 0.6 F_u A_{nv}$: $R_n = \min(F_u A_{nt} + 0.6 F_u A_{nv}; F_y A_{gt} + 0.6 F_u A_{nv})$
2005	
2010	$R_n = \min(F_u A_{nt} + 0.60 F_u A_{nv}; F_u A_{nt} + 0.60 F_y A_{gv})$
2016	

in bolted connections are neither the net nor the gross shear planes, as defined in the AISC *Specifications*. This evidence includes contact finite element analyses (Clements and Teh, 2013) and connection tests (Cai and Driver, 2010) that fail in shear tearout.

The determination of the effective shear planes resolves the ambiguity involving the net and gross shear planes used in the current block shear design provision (AISC, 2016). In addition, Teh and Yazici (2013) provide an explanation, substantiated by tests and analysis, that there is only one feasible mechanism for conventional block shear failures in bolted connections, which involves tensile rupture and shear yielding. In fact, the simultaneous tensile and shear rupture mechanism postulated by the first equation in the bottom row of Table 1 has never been observed in published laboratory tests. However, Teh and Uz (2015a) have demonstrated that the ductile shear yielding in a block shear failure is typically accompanied by significant strain hardening, such that the assumed yield stress can be significantly larger than $0.6F_y$, up to or even beyond $0.6F_u$. Based on these three observations, this paper proposes a design equation against the block shear failure mode of bolted connections that (1) is more accurate than existing models, including one proposed by Teh and Yazici (2013) and Teh and Uz (2015a); (2) is logical and straightforward to implement; and (3) is determined using parameters in current design provisions that are familiar to engineers.

This paper presents a comprehensive verification of block shear failure models of bolted connections in gusset plates composed of structural steel (Hardash and Bjorhovde, 1985; Rabinovitch and Cheng, 1993; Udagawa and Yamada, 1998; Aalberg and Larsen, 1999; Nast et al., 1999; Swanson and Leon, 2000; Puthli and Fleischer, 2001; Huns et al., 2002; Mullin, 2002; Moze and Beg, 2014) and aluminum alloy (Menzemer et al., 1999). The exercise includes both conventional and the less conventional “split” block shear failure mode. Comparisons are made against the design provisions found in the 2010 and 2016 AISC *Specifications* (AISC

2010, 2016) and the Canadian (CSA, 2014), European (ECS, 2005) and Japanese (AIJ, 2002) standards.

Design Provisions of AISC Specifications

As shown in Table 1, the first edition of the AISC LRFD *Specification* (AISC, 1986) specified that the larger of the following two resistances is to be used in determining the nominal block shear strength of bolted connections:

$$R_n = F_u A_{nt} + 0.6 F_y A_{gv} \quad (1a)$$

$$R_n = F_y A_{gt} + 0.6 F_u A_{nv} \quad (1b)$$

The net and gross shear and tension planes, as defined by the *Specification* are indicated in Figure 1. The accompanying Commentary argued that the provision was more conservative than the equation given in the earlier ASD *Specification* (AISC, 1978), which “implies that ultimate fracture strength on both planes occur simultaneously.” The 1978 equation, rewritten in the limit state format is as follows:

$$R_n = F_u A_{nt} + 0.6 F_u A_{nv} \quad (2)$$

In contrast to the 1986 Commentary’s claim of being more conservative, depending on the relative values of F_y and F_u , the 1978 equation can result in a lower resistance than the 1986 equation due to the its smaller shear area, as demonstrated by Teh and Yazici (2013).

Therefore, there are two fundamental problems with the 1986 provision (AISC, 1986). First, contrary to its intention of adopting a more conservative model, it often results in a less conservative design against the block shear failure mode compared to the original equation (AISC, 1978). Second, its prescription that “the controlling equation is one that produces the larger force” is contrary to well-established design conventions of choosing the lowest of multiple possible failure modes. The Commentary (AISC, 1986) attempts to explain the oddity of the design check by way of two extreme examples, shown in Figure 2. According

to the Commentary, Equation 1a gives a lower resistance than Equation 1b for the connection shown in Figure 2a. However, considering that the total force is resisted primarily by shear, the Commentary argues that shear fracture, not shear yielding, should control the block shear failure mode, and therefore, Equation 1b should be used for the connection in Figure 2a. A reverse argument is applied by the Commentary to the other connection. The Commentary further states that “when it is not obvious which failure plane fractures, it is easier just to use the larger of the two formulas.”

There are two points that have been overlooked in the 1986 Commentary. First, as with the comparison between Equations 1a and 2, Equation 1a does not, in general, give a lower resistance than Equation 1b for the connection shown in Figure 2a. Second, there is no evidence to support the contention that fracture will take place first in the primary resistance plane (i.e., tension or shear). In fact, the connection in Figure 2a will fracture first in the tension plane irrespective of the steel material ductility (Teh and Yazici, 2013). The connection in Figure 2b, on the other hand, will

fail in individual shear tearout of the bolts rather than block shear if the tensile resistance is sufficiently larger than the shear resistance.

The second LRFD *Specification* (AISC, 1993) recognizes the first point described in the preceding paragraph and qualifies the use of Equations 1a and 1b as shown in Table 1. The *Specification* Commentary modifies the 1986 prescription to “the controlling equation is one that produces the larger *rupture* force.” However, this modified prescription does not have a clear justification either, except that the 1993 Commentary repeats the earlier Commentary’s statement that “block shear is a rupture or tearing phenomenon not a yielding limit state.”

The 1993 Commentary has an additional argument for the form of Equations 1a and 1b that survives into the latest Commentary (AISC, 2016). It argues that the equations are consistent with the philosophy of tension member design, “where gross area is used for the limit state of yielding and net area is used for rupture.” However, the gross area is used for the tension member design in conjunction with the yield

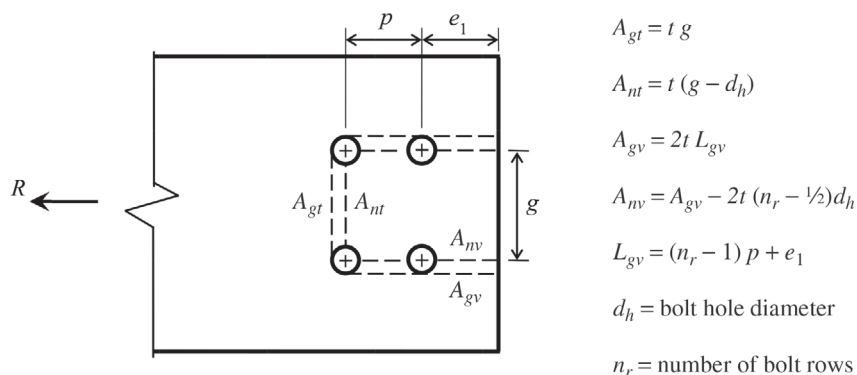


Fig. 1. Gross and net planes.

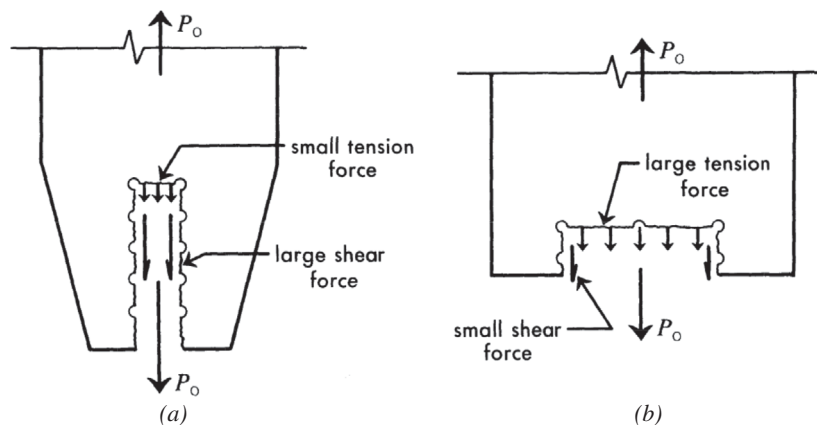


Fig. 2. Two extreme block shear examples cited in the AISC Commentary (AISC, 1986): (a) shear-resistance dominant; (b) tensile-resistance dominant.

stress to prevent excessive member elongation due to yielding along the member. This condition is not present in the design against a block shear failure, where yielding is local to the connection region only.

The third LRFD *Specification* (AISC, 1999) recognizes the fact that Equation 2 may give a lower resistance than either Equation 1a or 1b. Accordingly, as noted in Table 1, the 1999 *Specification* requires the checking of Equation 2 in applying Equations 1a and 1b.

Two further changes to the block shear design provision have been incorporated in the 2005 *Specification* (AISC, 2005). First, the governing block shear strength is changed to simply the lower resistance computed from the prescribed equations. Second, the tensile yielding and shear rupture mechanism, Equation 1b, is no longer considered. The rationale for the removal of Equation 1b is explained by Teh and Yazici (2013), who point out that a conventional block shear failure cannot occur through tensile yielding and shear rupture.

Despite the first improvement mentioned in the preceding paragraph, the resulting block shear design provision, which remains the same in the 2010 and 2016 *Specifications* (AISC, 2010, 2016), often reduces to Equation 2 for modern structural steels, where the ratio of tensile strength F_u to yield stress F_y is not particularly high. As will be evident in the next section, the net shear area A_{nv} used in the equation is significantly smaller than the more realistic value given by the effective shear area. The practical outcome is that the design provision can be very conservative, relative to physical test data.

Effective Shear Planes

If one looks at Figures 1 and 2 closely, it will become apparent that the shear failure planes cannot coincide with the centerlines of the bolt holes in the direction of loading, where shear stresses would be minimal due to the bolts bearing “symmetrically” on the respective holes. This indication



Fig. 3. A downstream bolt failing in shear tearout (Cai and Driver, 2010).

has been confirmed by the contact finite element analysis results of Clements and Teh (2012), which show that maximum in-plane shear stresses take place between the net and the gross shear planes.

This assertion is clearly evident in the observed failure mode in laboratory tests of bolted connections failing in shear tearout. For example, consider the failure mode shown in Figure 3 for the downstream bolt of a serial bolted connection tested by Cai and Driver (2010). In a shear tearout failure, full strain hardening can be expected along the two shear failure planes, enabling the determination of their location and effective area based on simple calculation checks under the limiting stress of $0.6F_u$. Moreover, from the photo in Figure 3, the shear tearout plane can be seen to be roughly midway between the net and the gross shear planes. For the shear tearout failure, the net A_{nv} , gross A_{gv} , and effective A_{ev} shear planes are shown in Figure 4. Note that in contrast to the gross and effective shear planes, the literal interpretation of the net area is that it can have only one failure plane.

The Canadian steel design standard (CSA, 2014) determines the nominal shear tearout strength of bolts from the following equation:

$$R_n^* = 0.6 \left(\frac{F_u + F_y}{2} \right) (2A_{gv}) \quad (3)$$

which assumes partial strain hardening along the total area of two gross shear planes ($2A_{gv}$).

The shear tearout equation of the current AISC *Specification* (AISC, 2016) and the North American cold-formed steel structures *Specification* (AISII, 2012) are described as follows,

$$R_n^* = 1.2F_u A_{nv} \quad (4)$$

While it is obviously impossible to have the two shear failure planes coinciding with the centerlines of the bolt hole, two net shear planes are implied by the limiting shear stress, $1.2F_u$. Note that this shear tearout equation corresponds to the case in the AISC *Specification* where deformations are to be controlled. As an aside, the AISC shear tearout equation for the case where deformations are not controlled has a limiting stress of $1.5F_u$, which implies a failure stress of $0.75F_u$ on each net shear plane. The authors are not aware of test evidence to support the use of $0.75F_u$, and moreover, Teh and Uz (2015b) have pointed out that test evidence supports a limiting shear tearout stress of $0.6F_u$ on each effective shear area.

Following Teh and Uz (2015b), if each shear failure plane is taken to be midway between the gross and the net shear planes, then the nominal shear tearout strength is calculated as:

$$R_n^* = 0.6F_u (2A_{ev}) \quad (5a)$$

where

$$A_{ev} = \frac{A_{gv} + A_{nv}}{2} \quad (5b)$$

Equations 3, 4, and 5 have been verified by Teh and Uz (2015b) against independent laboratory test results. However, at the time, they were not aware of the single-bolt connection test results obtained by Moze and Beg (2010, 2014), which provide even stronger evidence that Equation 5 is significantly more accurate than Equations 3 and 4. Table 2 lists the geometric and material variables of the Moze and Beg tests, which included specimens of mild and high-strength steels with bolt holes ranging from 18 to 30 mm. The table summarizes ratios of ultimate test load P_t to the predicted shear tearout strength R_n^* (such a ratio is called the “professional factor”) given by Equations 3, 4 and 5. The variable d_h is the bolt hole diameter, e_1 is the distance between the center of the bolt hole and the downstream end, and t is the plate thickness. An empty cell indicates that the data in the above cell applies. The summary statistics are separated between tests of mild steel specimens, where the ratio F_u/F_y is 1.36, and tests of high-strength steel specimens, where the ratio is 1.04. This distinction is important to help differentiate between the assumed ultimate stresses versus shear failure planes used in the models. It should be noted that the reported F_y and F_u values are all measured, as opposed to nominal, values.

The results in Table 2 show that, despite the assumption of only partial strain hardening, Equation 3 specified in the Canadian steel design standard (CSA, 2014) unconservatively overestimates the shear tearout strengths of the mild steel specimens by about 10% on average. For the high-strength steel specimens, the largest overestimation

is over 35% ($1/0.73 = 1.37$). These overestimations are due to the optimistic assumption that the shear failure planes are the gross shear planes, rather than the absence of shear strain hardening in the test specimens.

Conversely, even though full strain hardening is considered by Equation 4, as specified in the AISC *Specification* (AISC, 2016) and the cold-formed steel specification (AISI, 2012), the use of the net shear planes leads to excessive conservatism in the predicted strengths.

Equation 5, which is based on the effective shear planes, calculated as the mean between the net and the gross shear planes, is consistently more accurate than both Equations 3 and 4. Thus, these data indicate that the shear failure planes lie midway between the net and the gross shear planes—a conclusion that is consistent with the design recommendation of Tolbert and Hackett (1974) for pin lugs.

Proposed Equation against Block Shear Failure

Having established strong evidence that the effective shear failure planes are located between the net and the gross shear planes, the block shear failure mechanism can be revised to the one shown Figure 5.

The reasoning for this model is substantiated by Teh and Yazici (2013), who explain why there is only one feasible mechanism for the conventional block shear failure mode—namely, the tensile rupture and shear yielding mechanism. Teh and Uz (2015a) have further pointed out that shear yielding in a block shear failure is typically accompanied by full strain hardening ($0.6F_u$), even though shear fracture very rarely, if ever, is the triggering failure mechanism. This can be explained by the large ductility of steel in shear, where the steel in the shear yielding zone can strain harden up to

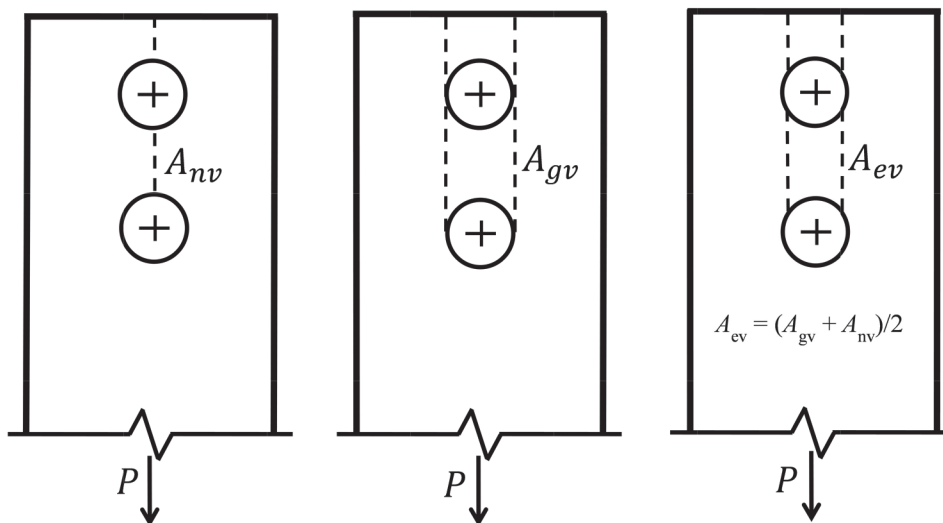


Fig. 4. Illustration of net, gross and effective shear planes for shear tearout.

Table 2. Comparison of Shear-Out Equations with Tests by Moze and Beg (2010, 2014)

Specimen	F_y , ksi (MPa)	F_u , ksi (MPa)	t , in. (mm)	d_h , in. (mm)	e_1 , in. (mm)	P_t/R_n^* of Equations		
						3	4	5
M101	45.4 (313)	61.6 (425)	0.472 (12)	1.02 (26)	1.26 (32)	0.89	1.30	0.97
M104	—	—	—	—	1.02 (26)	0.90	1.57	1.05
M105	—	—	—	—	1.26 (32)	0.97	1.41	1.05
M109	—	—	—	0.71 (18)	0.71 (18)	0.88	1.53	1.02
M110	—	—	—	—	0.87 (22)	0.89	1.31	0.97
M111	—	—	—	—	1.06 (27)	0.90	1.17	0.94
Moze and Beg (2014)					Mean	0.90	1.38	1.00
					COV	0.034	0.109	0.047
B109	122 (847)	128 (885)	0.39 (10)	1.18 (30)	1.18 (30)	0.73	1.43	0.95
B118	—	—	—	—	1.81 (46)	0.82	1.19	0.96
Moze and Beg (2010)					Mean	0.78	1.31	0.96
					COV	0.082	0.127	0.005

F_u and sustain large strains without the necking and rupture behavior that occurs in standard tensile coupons.

Based on this reasoning, the following block shear equation is proposed:

$$R_n = F_u A_{nt} + 0.6 F_u A_{ev} \quad (6)$$

This equation, in which the effective shear area A_{ev} is simply the mean between the gross and the net shear areas, as shown in Equation 5b, is slightly more accurate than a similar equation proposed by Teh and Yazici (2013) and Teh and Uz (2015a), which computes the shear areas from the shear plane length that ignores a quarter of the bolt hole diameter. In addition to being more accurate, the concept of an effective shear area is intuitive and straightforward to implement.

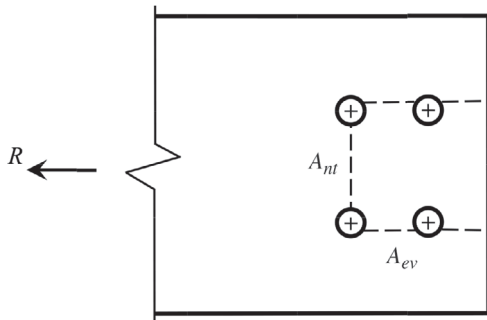


Fig. 5. Net tension and effective shear failure planes for proposed block shear model.

Verifications of Block Shear Equations

Strengths calculated using the proposed block shear Equation 6 and the current AISC provision (AISC, 2016) are compared to previously published test data in Table 3. The table covers 155 tests by 11 independent research teams, including tests of 20 aluminum specimens by Menzemer et al. (1999). Due to the large number of specimens involved, it is not practical to provide the details of individual specimens in the manner given by Table 2. In addition to the mean professional factors P_t/R_n , Table 3 provides the number of tests by each research group (N), their maximum number of bolt rows (n_{rmax}) and number of bolt lines (n_{lmax}), the range of their bolt hole diameters (d_h), the range of their ratios of measured tensile strength to yield stress (F_u/F_y), and the range of their ratios of ultimate test load to tensile strength component [$P_t/(F_u A_{nt})$]. The overall mean values and the coefficients of variation (COV) given at the bottom of the table refer to the professional factors of individual specimens, not the mean professional factors of the 11 test programs. All the specimens in the table failed in a conventional block shear mode along the failure planes illustrated in Figure 5.

It can be seen from Table 3 that strengths calculated by the proposed Equation 6 are consistently accurate across reported tests, where the overall mean professional factor of 1.01 has a 5% coefficient of variation. In contrast, the current AISC equation (AISC, 2016) has an overall mean professional factor of 1.18. Thus, the AISC provision is conservative by about 20%. Interestingly, the AISC results are almost always governed by Equation 2, simultaneous fracture on the net tension and shear areas, which is somewhat counterintuitive as being more conservative than the

	<i>N</i>	<i>n_{rmax}</i>	<i>n_{1max}</i>	<i>d_h</i> (mm)	<i>F_u/F_y</i>	<i>P_t/(F_uA_{nt})</i>	Mean <i>P_t/R_n</i>	
							AISC	Proposed
Hardash and Bjorhovde (1985)	28	5	2	14–17	1.30–1.41	2.2–6.7	1.20	1.03
Rabinovitch and Cheng (1993)	5	5	2	22	1.20	7.5–8.5	1.17	0.99
Udagawa and Yamada (1998)	72	4	4	18	1.08–1.70	1.7–6.0	1.18	0.99
Aalberg and Larsen (1999)	8	4	2	19	1.05–1.44	4.0–7.1	1.20	0.99
Menzemer et al. (1999)	20	7	2	17.5	1.12	3.7–14.0	1.16	1.00
Nast et al. (1999)	3	5	2	22	1.17	8.2–8.5	1.23	1.04
Swanson and Leon (2000)	1	4	2	24	1.33	4.1	1.30	1.05
Puthli and Fleischer (2001)	6	1	2	30	1.23	2.1–2.4	1.18	1.01
Huns et al. (2002)	5	3	4	21	1.34	2.6–8.0	1.26	1.08
Mullin (2002)	5	8	2	21	1.37	2.6–7.8	1.14	1.00
Moze and Beg (2014)	2	1	2	22	1.36	2.3–3.1	1.24	1.08
Overall mean							1.18	1.01
COV							0.051	0.048

Note: 1 in. = 25.4 mm

combined yielding-fracture condition. The performance of Equation 1a is included in the Appendix.

As noted previously, Table 3 only includes bolted connection specimens that failed in a conventional block shear mode along the failure planes illustrated in Figure 5. However, depending on the geometry, it is possible for a bolted gusset plate to fail in the “split” block shear mode along the planes indicated in Figure 6. In addition to testing six specimens that failed in the conventional (C) block shear mode, Puthli and Fleischer (2001) tested six specimens that failed in the split (S) mode. The split mode occurred in gusset plate specimens where the gauge length, *g*, was more than twice the edge distance, *e*₂. Results for the 12 specimens tested by Puthli and Fleischer (2001) are summarized in Table 4, including the six that failed in the conventional mode and are also included in Table 3. All the specimens had a plate thickness, *t*, of 17.5 mm, a bolt hole diameter, *d*_h, of 30 mm, an end distance, *e*₁, of 36 mm, and one row of two bolts. The measured yield stress, *F*_y, and tensile strength, *F*_u, were 524 MPa and 645 MPa, respectively.

For determining the split block shear strength, the “normal” block shear equations are still applicable provided the appropriate net tension area, *A*_{nt}, is used. Similar to the comparisons of Table 3, the results in Table 4 demonstrate that the strengths based on the proposed Equation 6 are significantly more accurate than those determined using the AISC (AISC, 2016), Canadian (CSA, 2014), European (ECS, 2005) and Japanese (AIJ, 2002) equations for both the conventional and the split block shear modes. Further details and discussion of the Canadian, European and Japanese standard equations are given in the Appendix.

Resistance Factor

In conjunction with the proposed new model for determining shear tearout and block shear, the comparisons with test data can be used to evaluate an appropriate resistance factor. The reliability analysis methodology and the statistical parameters are adopted from Driver et al. (2006), who evaluated the required resistance factor ϕ using the following equation proposed by Fisher et al. (1978):

$$\phi = (0.0062\beta^2 - 0.131\beta + 1.338)M_m F_m P_m e^{-P} \quad (7)$$

in which β is the target reliability index, *M*_m is the mean value of the material factor equal to 1.11 (Schmidt and Bartlett 2002), *F*_m is the mean value of the fabrication factor equal to 1.00 (Hardash and Bjorhovde 1985), and *P*_m is the mean value of the professional factor.

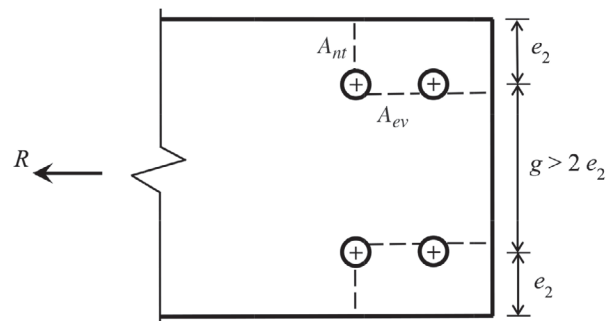


Fig. 6. Split block shear failure planes.

Table 4. Comparison of Model Equations for Tests That Failed in Both Conventional (C) and Split (S) Block Shear (Puthli and Fleischer, 2001)

Specimen	g, (mm)	e ₂ , (mm)	Mode	P _t /R _n				
				AISC	Eq. (6)	CSA	ECS	AIJ
12	54	36	C	1.16	0.98	0.90	1.28	1.07
13	—	40.5	—	1.16	0.98	0.90	1.28	1.07
14	—	45	—	1.19	1.01	0.93	1.32	1.10
17	63	36	—	1.19	1.03	0.96	1.29	1.11
18	—	40.5	—	1.21	1.05	0.97	1.32	1.13
19	—	45	—	1.19	1.03	0.96	1.30	1.11
20	72	27	S	1.23	1.04	0.96	1.36	1.14
21	—	31.5	—	1.21	1.05	0.97	1.31	1.13
22	81	27	—	1.18	1.00	0.92	1.31	1.09
23	—	31.5	—	1.19	1.03	0.96	1.29	1.11
24	90	27	—	1.20	1.01	0.93	1.33	1.11
25	—	31.5	—	1.19	1.03	0.96	1.30	1.12
Puthli and Fleischer (2001)			Mean	1.19	1.02	0.94	1.31	1.11
			COV	0.017	0.023	0.027	0.017	0.019

The exponential term p in Equation 7 is computed from

$$p = \alpha_R \beta \sqrt{V_m^2 + V_F^2 + V_P^2} \quad (8)$$

in which α_R is the separation variable equal to 0.55 (Ravindra and Galambos, 1978), V_M is the coefficient of variation of the material factor equal to 0.054 (Schmidt and Bartlett 2002), V_F is the coefficient of variation of the fabrication factor equal to 0.05 (Hardash and Bjorhovde, 1985), and V_P is the coefficient of variation of the professional factor.

The mean professional factor P_m of the proposed Equation 6 for the 141 structural steel specimens included in Tables 3 and 4 is 1.01, and its coefficient of variation V_P is 0.051. The aluminum specimens tested by Menzemer et al. (1999) were not included in the reliability analysis, although it would have made little difference to the computed resistance factor. Using these values, along with a target reliability index β of 4.0, a resistance factor ϕ of 0.84 was computed per Equation 7. Accordingly, and based on the large number of test comparisons covering a very diverse range of connection geometry and steel grades, a resistance factor ϕ rounded up to 0.85 is recommended.

BLOCK SHEAR DESIGN EXAMPLE

Shown in Figure 7 is a connection between a pair of back-to-back C6×13 tension braces and an ASTM A572 Grade 50 5/8-in.-thick gusset plate (F_y of 50 ksi; F_u of 65 ksi). The gusset plate thickness is selected so as to have sufficient bolt

bearing strength to develop the full design shear strength of each 3/4-in. ASTM F3125 Grade A325 bolt of 45.1 kips (double shear, bearing). The bolted gusset plate, which has two lines of bolts with a hole diameter equal to 13/16 in., is to be designed against the block shear failure mode under a factored load of $R_u = 270$ kips. The pitch, p , and gauge, g , are all equal to 2 1/2 in., while the end distance e_1 is 1 1/2 in. These values satisfy the requirements prescribed in Sections J3.3 and J3.4 of the AISC Specification (AISC, 2016).

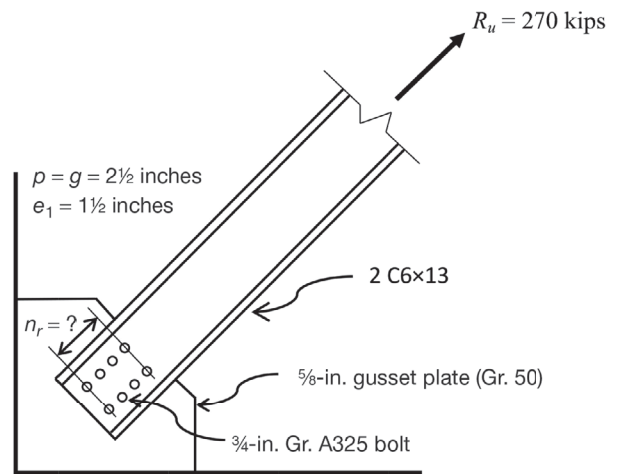


Fig. 7. Block shear design example.

Based on the bolt shear strength, six bolts are required to resist the factored load of 270 kips. The following calculations determine the number of bolt rows, n_r , required according to the AISC *Specification* (AISC, 2016) and the proposed block shear equation. For all block shear designs, the net tension area, A_{nt} , is constant at 1.02 in.². In all calculations, $1/16$ in. is added to the nominal bolt hole diameter in accordance with Section B4.3b of the *Specification*.

Try $n_r = 3 \rightarrow A_{gv} = 8.12 \text{ in.}^2; A_{nv} = 5.39 \text{ in.}^2$:

AISC:

$$\phi R_n = 0.75 \times \min(F_u A_{nt} + 0.60 F_u A_{nv}; F_u A_{nt} + 0.60 F_y A_{gv}) = 207 \text{ kips} \quad \mathbf{n.g.}$$

Proposed:

$$\phi R_n = 0.85 \times [F_u A_{nt} + 0.6 F_u (A_{nv} + A_{gv})/2] = 280 \text{ kips} \quad \mathbf{o.k.}$$

Try $n_r = 4 \rightarrow A_{gv} = 11.2 \text{ in.}^2; A_{nv} = 7.42 \text{ in.}^2$:

$$\text{AISC: } \phi R_n = 267 \text{ kips} < 270 \text{ kips}$$

In this example, the AISC block shear check of the gusset plate requires much more than the minimum number of six bolts based on the bolt shear capacity. It requires five rows of bolts (10 bolts), whereas the proposed block shear model requires only three rows of bolts (six bolts). These calculations assume that plate thickness, bolt pitch, and end distance are to be maintained.

An alternative to increasing the number of bolt rows in the current AISC block shear check is to increase the bolt pitch (there is a very limited scope for increasing the bolt gage since the clear width of each channel brace is only 5.31 in.). In order to resist the factored load of $R_u = 270$ kips, the bolt pitch must be increased to $3\frac{1}{2}$ in. in step increases of $\frac{1}{2}$ in. each.

Try $p = 3\frac{1}{2} \text{ in.} \rightarrow A_{gv} = 10.62 \text{ in.}^2; A_{nv} = 8.09 \text{ in.}^2$:

$$\text{AISC: } \phi R_n = 280 \text{ kips} \quad \mathbf{o.k.}$$

In either AISC solution, the resulting gusset plate dimension

is larger than that given by the proposed block shear solution.

While it can be argued that this example is not necessarily representative of a typical condition, it demonstrates how the proposed block shear check can result in more economical connections.

COPED BEAM SHEAR CONNECTIONS

In essence, all of the changes to the AISC block shear design provisions listed in Table 1 and discussed in the section “Design Provisions of AISC *Specifications*” relate to tension members. While the block shear failure mode was first discovered by Birkemoe and Gilmore (1978) for coped beam shear connections, the coped beam condition may involve another level of complexity that clouds the basic behavior of block shear. The nonuniform stress distribution factor, U_{bs} , contained in the AISC block shear design provision since 2005 (AISC, 2005), is believed to arise from the in-plane load eccentricity of the coped beam shear connection, which is outside the scope of this paper.

Nevertheless, a reviewer of this paper has suggested that a verification of Equation 6 be made against the recent coped beam test results of Fang et al. (2013). The experimental program of Fang et al. is well documented and interesting in that it included not only single- and double-line bolted connections, but also single- and double-sided angle cleat connections on the beam web. The single- and the double-line bolted connections are believed to result in uniform ($U_{bs} = 1.0$) and nonuniform ($U_{bs} = 0.5$) tensile stress distributions, respectively, as illustrated in Figure 8 adapted from the 2016 *Specification* (AISC, 2016). The single-sided cleat connections, on the other hand, result in out-of-plane load eccentricity.

Table 5 shows the professional factors of Equation 6 for the coped beam shear connections of Fang et al. (2013). The geometric variables are defined in Figure 8, with the bolt pitch, p , being uniform at 75 mm for all specimens except for T1-1-3-a. The first seven specimens were single-line bolted,

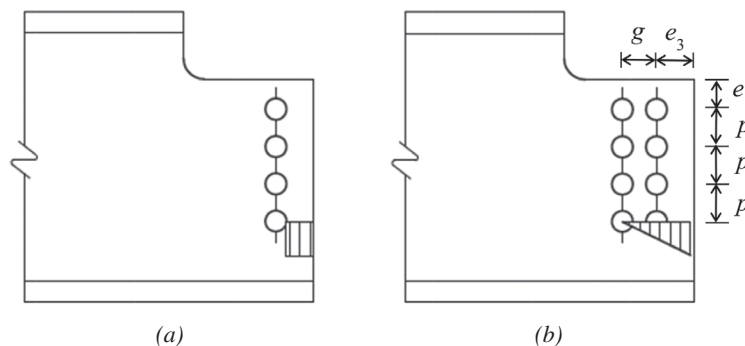


Fig. 8. AISC tensile stress distribution factor: (a) $U_{bs} = 1.0$; (b) $U_{bs} = 0.5$ (AISC, 2016).

Table 5. Verification against Coped Beam Test Results of Fang et al. (2013), $\rho = 75$ mm

Specimen	g , (mm)	e_3 , (mm)	e_1 , (mm)	t , (mm)	F_u , (MPa)	Bolt Lines	Single Sided?	P_t , (kN)	P_t / Eq. (6)
A1-1-3-a	N/A	28	28	6.6	459	Single	N	305	0.96
T1-1-3-a*	—	28	28	—	459	—	Y	332	1.05
A1-1-3-b	—	50	27	—	459	—	N	393	1.03
T1-1-3-b	—	50	28	—	459	—	Y	415	1.08
T2-1-3-a	—	27	28	6.8	464	—	Y	358	1.09
T2-1-3-b	—	51	28	—	464	—	Y	485	1.20
A1-1-3-a-S	—	28	29	—	459	—	Y	319	1.02
A2-2-2-a	75	28	27	—	464	Double	N	384	1.04
T1-2-2-a	—	28	28	—	464	—	Y	380	1.02
T2-2-2-a	—	28	27	—	459	—	Y	329	0.90
								Mean	1.04
								COV	0.077
* The upper pitch is equal to 74 mm.									

while the last three were double-line bolted. No nonuniform stress distribution factor was applied to any specimen. In this table, the ultimate test loads, P_t , are explicitly given in order to avoid potential confusion with the “Ultimate load P_{test} ” given in Table 4 of Fang et al. (2013), the latter referring to the ultimate load applied on the beam, which had two end connections (not carrying the same amount of load). In that paper, the ultimate block shear load, P_t , of each shear connection of the tested beam is called “ultimate connection reaction,” measured by a load cell. It can be seen that Equation 6 resulted in reasonable estimates of the ultimate block shear loads, including those of the double-line bolted, single-sided cleat connections.

The slightly conservative estimates by Equation 6—despite the equivalent use of $U_{bs} = 1.0$ in the presence of in-plane and out-of-plane load eccentricities, especially for Specimen T2-1-3-b—might have been due to the fact that the bolts were snug tightened. It may also be noted that Fang et al. (2013) have found that all the AISC, Canadian, European and Japanese block shear design provisions for coped beams were excessively conservative. The maximum professional factors of the international design specifications for the specimens in Table 5 were computed by Fang et al. (2013) to be 1.66, 1.62, 2.05 and 1.63, respectively.

The only significant overestimation by Equation 6 concerns specimen T2-2-2-a, for which the professional factor is 0.90. However, Fang et al. (2013) suggested that this specimen might have suffered from an “erratic test setup alignment” because it was similar to specimen T1-2-2-a (except for a slightly lower tensile strength F_u), but the latter had an ultimate test load that was 15% higher. In fact, the ultimate beam load of specimen T2-2-2-a obtained in the laboratory

test was about 15% lower than the finite element prediction of Fang et al. (2013).

It may be noted that, in addition to the “erratic” test result discussed in the preceding paragraph, test results involving coped beam shear connections have been known to be quite variable for nominally identical specimens. Such results include those obtained by Franchuk et al. (2003), which were among the few well-documented coped beam shear tests available in the literature.

Due to the limited verification against coped beam test results, the authors do not propose amending the current non-uniform stress distribution factor U_{bs} , which means that the proposed block shear design provision tends to be conservative for a double-line bolted coped beam shear connection.

CONCLUSIONS

In spite of changes to the block shear design provisions in the AISC *Specifications* over four decades, the current provision is shown to underestimate the block shear strength of bolted connections by about 20%, compared to published test data. The Canadian, European and Japanese block shear design provisions all have unique equations of their own, yet comparisons with these also reveal significant errors on either side of conservatism when verified against the independent test results, as cited in this paper, including those presented in the Appendix. All the *Specification* equations share two fundamental shortcomings. First, they use either the net area (too conservative) or the gross area (too optimistic) for the shear failure planes, neither of which is a reliable representation of the shear failure plane. Second, none of the provisions recognize that shear yielding in a block shear

failure is typically accompanied by full strain hardening, prior to rupture of the net tension failure plane.

Observed deformations and failure modes in shear tearout tests of bolted connections suggest that the effective shear failure plane lies midway between the net and the gross shear planes. This is further substantiated by comparisons between the measured failure loads and the strengths calculated assuming different failure plane areas under the maximum shear stress of $0.6F_u$. Accordingly, the effective shear area for evaluating shear tearout failure can be calculated as the mean between the gross and the net shear areas.

The proposed block shear model (Equation 6) is based on failure due to the tensile rupture and shear yielding mechanism, which is supported by test data and basic mechanics. The model assumes full strain hardening (plastic flow stress of $0.6F_u$) along the effective shear planes, up to the point of tensile fracture on the net tension plane. The calculated strengths agree well with data from 161 block shear tests of bolted gusset plates conducted by 11 independent research teams, including specimens composed of mild and high-strength steels, an aluminum alloy, and both conventional and split block shear failure modes. Based on the observed variability in calculated to measured strengths and well-established reliability indices for connections, a resistance factor of 0.85 is recommended for use with the proposed block shear strength equation. The proposed equation for calculating block shear can also be adapted to provide more consistent strengths for shear tearout failure of bolts.

Although the proposed equation resulted in reasonable estimates for the studied coped beam shear connections, including those with a double line of bolts without the application of any nonuniform stress distribution factor, more research is required to determine the accurate nonuniform stress distribution factor if it is necessary at all. By retaining the existing nonuniform stress distribution factor $U_{bs} = 0.5$ for a double-line bolted coped beam connection, the proposed block shear provision tends to err on the safe side for such a connection.

Relative to the block shear design provisions in the current AISC *Specification* (2016), the use of the proposed provision will facilitate structural designs that are more reliable and economical. This is demonstrated in a gusset plate example, where the proposed block shear equation leads to a significant reduction in the number of required bolt rows or in the bolt pitch and, therefore, smaller gusset plate dimensions than the current provision.

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APPENDIX—INTERNATIONAL SPECIFICATIONS

The block shear equation in the Canadian standard (CSA, 2014) is essentially the same as that originally proposed by Huns et al. (2002), which assumes partial strain hardening along the gross shear planes,

Table A-1. Effects of Assumptions and Approximations in Block Shear Equations

Specimens	F_y , (MPa)	F_u , (MPa)	t , (mm)	n_r	P_i/R_n				
					Eq. (1a)	Eq. (6)	CSA	ECS	AIJ
T7	373	537	8.4	2	1.21	1.06	1.05	1.59	1.38
T9	—	—	—	3	1.18	1.03	1.01	1.65	1.36
T11	—	—	—	4	1.13	0.99	0.96	1.64	1.32
T15	—	—	—	3	1.12	0.98	0.95	1.56	1.29
T8	786	822	7.7	2	0.90	1.00	0.89	1.21	1.04
T10	—	—	—	3	0.86	0.97	0.84	1.22	1.00
T12	—	—	—	4	0.82	0.94	0.80	1.20	0.96
T16	—	—	—	3	0.83	0.94	0.82	1.18	0.97
Aalberg and Larsen (1999)				Mean	1.01	0.99	0.91	1.41	1.16
				COV	0.169	0.043	0.100	0.154	0.163

Note: $d_h = 19$ mm; $e_1 = 38$ mm; $p = 48$ mm; $g = 48$ mm

$$R_n = F_u A_{nt} + \left(\frac{F_u + F_y}{2\sqrt{3}} \right) A_{gv} \quad (A-1)$$

except that the von Mises shear coefficient ($1/\sqrt{3} = 0.577$) is replaced by 0.6 in the standard

$$R_n = F_u A_{nt} + 0.6 \left(\frac{F_u + F_y}{2} \right) A_{gv} \quad (A-2)$$

Huns et al. (2002) found that Equation 1 was reasonably accurate for many of the specimens included in Table 3. However, this apparent accuracy has been due to the fact that the optimistic use of the gross shear area, A_{gv} , is offset by the pessimistic assumption of only partial strain hardening in specimens having high ratios of tensile strength to yield stress, F_u/F_y . The specimens of Puthli and Fleischer (2001) listed in Table 4 had a F_u/F_y ratio of 1.23, which is not particularly high, and the Canadian standard’s equation (CSA, 2014) leads to overestimations as large as 10%, depending on the geometry. These overestimations are not due to the inability of the specimens to experience shear strain hardening, but rather are due to the assumption of gross shear planes.

Table A-1 lists the professional factors of alternative equations for the block shear specimens tested by Aalberg and Larsen (1999), where the first four specimens had material with a high F_u/F_y of 1.44 and the last four had a low F_u/F_y of 1.05. Data from the first four specimens might substantiate the Canadian standard’s Equation A-2, which assumes partial strain hardening along the gross shear planes; however, the unconservative errors for the last four specimens, which have a low F_u/F_y ratio, highlight the overestimation of strengths that are based on the gross shear planes. This

point is further supported by the results of Equation 1a, which assumes no shear strain hardening at all, where the results are similar to those of the Canadian equation for the specimens with a low F_u/F_y ratio. Lack of shear strain hardening is, therefore, not a factor. What is common to Equations 1a and A-2 is the use of gross shear planes, which are larger than the effective shear planes used by Equation 6. For specimens with relatively low ratios of F_u/F_y , the pessimistic assumption of nil, or only partial, strain hardening does not offset the excess of the gross shear areas over the effective shear areas.

In recent literature, the von Mises shear coefficient of $1/\sqrt{3}$ is sometimes identified as a more “correct” value than the commonly used shear coefficient of 0.6 for evaluating the shear fracture limit state, $0.6F_u$. For example, Moze and Beg (2014) proposed replacing the shear coefficient in the current AISC block shear provision (AISC, 2016) with the von Mises shear coefficient. However, given the discrepancies noted in Table 3, the practical difference of about 4% between $1/\sqrt{3}$ ($= 0.577$) and 0.6 is insignificant. Moreover, while the von Mises shear coefficient of $1/\sqrt{3}$ has a theoretical basis for yielding behavior (i.e., Huber–von Mises–Hencky distortion energy theory, Timoshenko, 1953), there is no such theoretical basis for the ultimate shear coefficient.

While the Eurocode block shear design equation (ECS, 2005) uses the von Mises coefficient in conjunction with shear yielding along the net shear planes,

$$R_n = F_u A_{nt} + \frac{F_y A_{nv}}{\sqrt{3}} \quad (A-3)$$

as shown in Tables 4 and A-1, the Eurocode’s Equation A-3 is often excessively conservative.

Among the models in current standards, the Japanese one (AIJ, 2002) is the most accurate for the results shown in Table 4, where it is only about 10% conservative as compared to the unconservative Canadian model (CSA, 2014) and the 20 to 30% conservatism in the AISC (AISC, 2016) and European (ECS, 2005) models. However, the AIJ equation, which assumes yielding along the gross shear planes with a reduced shear coefficient of 0.5 (AIJ, 2002)

$$R_n = F_u A_{nt} + 0.5F_y A_{gv} \quad (A-4)$$

is quite conservative in most cases. In addition to the first four specimens listed in Table A-1, the conservatism of

Equation A-4 is 30% or more for many of the specimens tested by Udagawa and Yamada (1998), Huns et al. (2002), Mullin (2002), and Moze and Beg (2014).

In line with the results shown in Tables 3 and 4, the proposed Equation 6 is shown to be reasonably accurate for all block shear specimens tested by Aalberg and Larsen (1999) and listed in Table A-1. It is the only equation that is consistently accurate across all gusset plate specimens tested by independent research groups around the world.