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Keywords

plates, hot, capacity, rolled, bolted, connections, shear, block, steel

Disciplines

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BLOCK SHEAR CAPACITY OF BOLTED CONNECTIONS IN HOT-ROLLED STEEL PLATES

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ABSTRACT

This paper extends the research previously conducted at the University of Wollongong on block shear failure of bolted connections in cold-reduced steel sheets with low ductility to hot-rolled steel plates. It examines the applicability of the basic approach employed for cold-reduced sheet steel bolted connections, which makes use of the so-called active shear planes, to hot-rolled steel plate connections. The active shear planes lie midway between the gross and the net shear planes defined in the steel structures specifications. The paper shows that shear yielding leading to the block shear failure of a bolted connection in a hot-rolled steel gusset plate is typically accompanied by full strain hardening. The paper proposes a design equation that provides more accurate and consistent results compared to the American, Australian, Canadian and European code equations in determining the block shear capacities of bolted connections in hot-rolled steel gusset plates. A resistance factor of 0.85 is recommended in order to achieve a target reliability index of 4.0 or greater.

1. INTRODUCTION

Block shear failure is recognised as a strength limit state of bolted connections in the AISC Specification for Structural Steel Buildings (AISC 2010a), Eurocode 3 Part 1.8 (ECS 2005), Canadian and Australian steel structures standards (CSA 2009, SA 2012). However, since it was discovered by Birkemoe & Gilmor (1978) and first incorporated into the AISC specification (AISC 1978), the design provisions for determining the block shear capacity of a bolted connection have continued to change and even oscillate between certain equations, as described by Teh & Clements (2012) and summarised in Table 1. The reasons are at least two folds.

The first reason is that there was the uncertainty concerning the possible mechanisms even for conventional block shear failures. Some versions of the AISC specification (AISC 1978, 1989) assume the simultaneous shear and tensile rupture mechanism, while others provide for the shear yielding and tensile rupture mechanism and for the shear rupture and tensile yielding mechanism (AISC 1986, 1993). The latest version incorporates the simultaneous shear and tensile rupture mechanism and the shear yielding and tensile rupture mechanism (AISC 2010a).

The more important reason, however, is the inconsistent definitions of the shear failure planes used in the code equations for determining the block shear capacity. The gross shear area is used when the failure mechanism is shear yielding and tensile rupture, while the net shear area is used for the shear rupture and tensile yielding mechanism or simultaneous shear and tensile rupture mechanism. Such a procedure is awkward since shear yielding must precede shear rupture, and often leads to anomalies since a lower load is required to fail the connection by simultaneous shear and tensile ruptures.

Table 1. AISC's block shear design equations over the years

1978	$R_n = F_u A_{nt} + 0.6 F_u A_{nv}$
1986	$R_n = \max\left(F_u A_{nt} + 0.6 F_y A_{gv};\quad F_y A_{gt} + 0.6 F_u A_{nv}\right)$
1989	$R_n = F_u A_{nt} + 0.6 F_u A_{nv}$
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 $F_u A_{nt} \leq 0.6 F_u A_{nv}$: $R_n = F_y A_{gt} + 0.6 F_u A_{nv}$
1999	If $F_u A_{nt} \geq 0.6 F_u A_{nv}$: $R_n = \min\left(F_u A_{nt} + 0.6 F_u A_{nv};\quad F_u A_{nt} + 0.6 F_y A_{gv}\right)$ If $F_u A_{nt} \leq 0.6 F_u A_{nv}$: $R_n = \min\left(F_u A_{nt} + 0.6 F_u A_{nv};\quad F_y A_{gt} + 0.6 F_u A_{nv}\right)$
2005 2010a	$R_n = \min\left(F_u A_{nt} + 0.6 F_u A_{nv};\quad F_u A_{nt} + 0.6 F_y A_{gv}\right)$

The uncertainty and the inconsistency mentioned in the preceding paragraphs have been discussed by Teh & Clements (2012) and Clements & Teh (2012), who verified their theoretical expositions and proposed design equation against laboratory test results of bolted connections in cold-reduced steel sheets having low ductility. Teh & Clements (2012) found the use of the shear yield stress only in computing the shear resistance to block shear failure to be reasonably accurate.

This paper presents a modification to the block shear equation proposed by Teh & Clements (2012) to suit bolted connections in hot-rolled steel gusset plates, and verifies the resulting equation against laboratory test results obtained by various researchers around the world. All the steel materials used in the laboratory tests had much greater ductility compared to the cold-reduced sheet steels used by Teh & Clements (2012). For the purpose of the present work, the physical reasoning presented by Teh & Clements (2012) will be described in order to accentuate the feasible mechanism for a conventional block shear failure.

2. FEASIBLE MECHANISM FOR CONVENTIONAL BLOCK SHEAR FAILURE

Consider the connected end of a flat member shown in Figure 1 that is subjected to a concentric load and is restrained from out-of-plane failure modes. Leaving out the pure net section tension failure mode and the bearing failure mode from the present discussion, there are essentially only two possible failure modes for the connected end. If the connection shear length (which is denoted by e_n in Figure 1) is relatively short, it will fail by "shear out" of each bolt, as shown in Figure 2.

As the connection shear length e_n increases, or as the bolt spacing p_2 decreases, or both, any of which results in an increase of the aspect ratio (defined as e_n/p_2 for the connection depicted in Figure 1), a condition would be reached such that it is conceivable for the connected end to undergo block shear failure by simultaneous shear and tensile ruptures. The aspect ratio at which the hypothetical mechanism of simultaneous shear and tensile ruptures could occur is termed the threshold ratio in the present work.

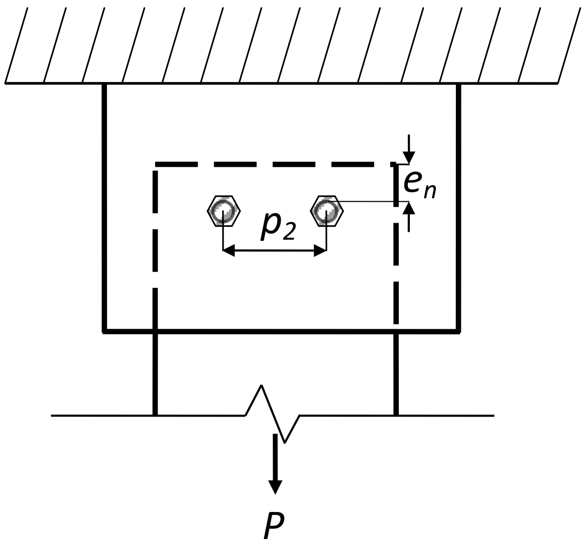


Figure 1. A two-bolt connection

Figure 2. Shear-out of individual bolts

However, once yielding around the perimeter of the block takes place and the block displaces as a whole, the tensile strains in the net section between bolt holes increase much more rapidly than the shear strains so that the block eventually fails by shear yielding and tensile rupture. Even at an aspect ratio that is slightly lower than the threshold ratio, a block shear failure by shear yielding and tensile rupture is still possible as shown in Figure 3, where the shear-out deformations were over-run by the shear yielding and tensile rupture mechanism. The change-over in the failure mode took place when yielding started in the tensile net section between the two bolt holes, where tensile rupture eventually took place.

Figure 3. Shear-out deformations gave way to block shear failure

As the aspect ratio increases beyond the threshold ratio, block shear failure can only be due to shear yielding and tensile rupture since the tensile strains are always more critical than the shear strains.

Obviously, at an aspect ratio that is sufficiently lower than the threshold ratio, the shear-out failure mode governs. There is therefore no aspect ratio at which a conventional block shear failure occurs by the shear rupture and tensile yielding mechanism.

In summary, a conventional block shear failure can only occur by the shear yielding and tensile rupture mechanism. However, shear yielding leading to a block shear failure may occur with significant strain hardening, depending on the ductility of the steel material. The greater the elongation at fracture exhibited by the tension coupon, the greater the scope for strain hardening along the shear yielding planes before fracture takes place in the tensile net section.

3. RELEVANT EQUATIONS FOR BLOCK SHEAR CAPACITY

Having established that a conventional block shear failure invariably fails by the shear yielding and tensile rupture mechanism, as borne out by laboratory test results, the present work is primarily concerned with the equations that are based on such a mechanism. There are four equations to consider.

The first equation is found in the AISC specification (AISC 2010a) and the Australian steel structures standard (SA 2012)

$$R_n = F_u A_{nt} + 0.6 F_y A_{gv} \tag{1}$$

in which F_u is the material tensile strength, F_y is the yield stress, A_{nt} is the net tensile area, and A_{gv} is the gross shear area. The implied block is depicted in Figure 4(a), which shows that the shear yielding planes assumed in Equation (1) lie at the outer perimeter of the block.

The second equation to consider is found in the European steel structures code (ECS 2005)

$$R_n = F_u A_{nt} + F_y A_{nv} / \sqrt{3} \approx F_u A_{nt} + 0.577 F_y A_{nv} \tag{2}$$

in which A_{nv} is the net shear area indicated in Figure 4(b). This approach ignores the fact that the planes coinciding with the centrelines of the bolt holes in the direction of loading do not have maximum shear stresses due to the bolt bearing condition.

The third equation is given in the Canadian steel structures standard (CSA 2009) based on the research results of Driver et al. (2006)

$$R_n = F_u A_{nt} + 0.3 (F_y + F_u) A_{gv} \tag{3}$$

in which the mean between the yield stress and the tensile strength is used to simulate the contribution of strain hardening to the shear resistance. The definition of the shear yielding planes is the same as that used in the AISC specification.

The fourth equation results from a modification to the block shear equation for cold-reduced sheet steels with low ductility and little strain hardening proposed by Teh & Clements (2012)

$$R_n = F_u A_{nt} + 0.6 \left(\frac{a F_y + b F_u}{a + b} \right) A_{av} \tag{4a}$$

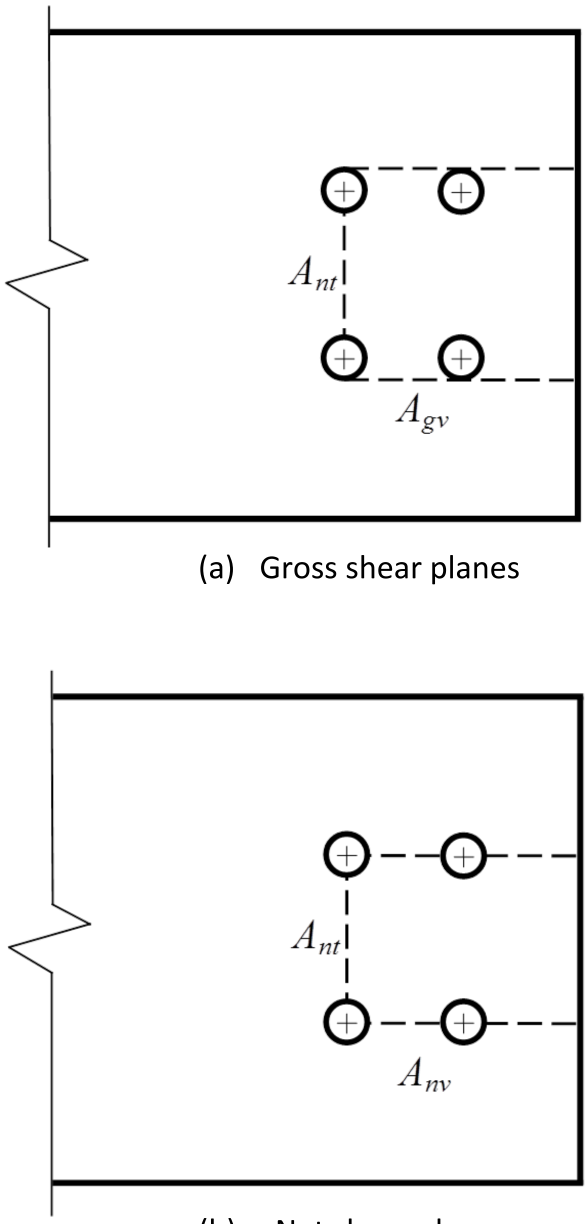
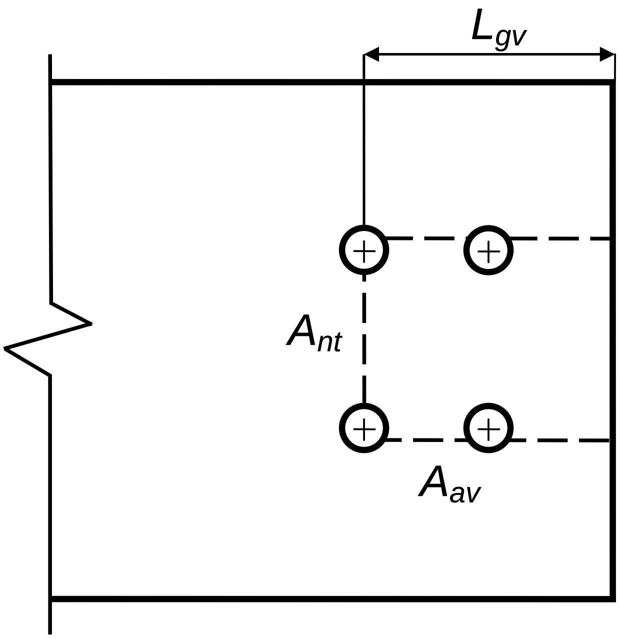


Figure 4. Gross and net shear failure planes

in which A_{av} is the active shear area defined in Figure 5. The variable n_r denotes the number of bolt rows. The coefficients *a* and *b* are used to determine the extent of strain hardening in the shear yielding planes. The greater the value of *b* relative to *a*, the greater the extent of strain hardening. A zero value of *a* indicates full strain hardening, while a zero value of *b* indicates no strain hardening. The appropriate values are discussed in the next section.

The active shear area A_{av} was used by Teh & Clements (2012) based partially on the experimental evidence of Franchuk et al. (2003) shown in Fig. 3 of their paper. The location of the active shear planes, which lie midway between the gross and the net shear planes, has subsequently been confirmed by Clements & Teh (2012) through nonlinear contact finite element analysis. Maximum shear stresses do not take place in both the gross and the net shear planes, but between them irrespective of whether the block shear failure occurs by shear yielding or shear rupture.



$$L_{av} = L_{gv} - \left(\frac{n_r - 1}{2} + \frac{1}{4}\right) d_h$$

$$\approx L_{gv} - {}^{n_r d_h}\!/_{2}$$

$$A_{av} = 2 L_{av} t$$

Figure 5. Active shear planes

4. COMPARISONS OF ALTERNATIVE EQUATIONS

Equations (1) through (4) are verified against the laboratory test results of bolted connections in hot-rolled steel gusset plates failing by block shear obtained by various researchers around the world. The steel materials used in the tests had much higher strain hardening capability compared to the high strength sheet steels tested by Teh & Clements (2012), with the ratio of tensile strength F_u to yield stress F_y being as high as 1.75. The uniform elongations at fracture of such steels were 3 to 6 times those of the sheet steels tested by Teh & Clements (2012), the latter having low uniform elongations at fracture between 6 and 8 percent only.

Based on the conclusion of Driver et al. (2006) in comparing the same test results against Equation (3), which makes use of the mean between F_y and F_u in conjunction with the gross shear area A_{gv}, it was surmised that full or almost full strain hardening had been achieved along the shear yielding planes in most of the tested hot-rolled steel specimens. Therefore, Equation (4a) becomes, with a = 0 and b = 1:

$$R_n = F_u A_{nt} + 0.6 F_u A_{av} \tag{4b}$$

It should be noted that, unlike the AISC specifications (1978, 2010a), the use of the tensile strength F_u in Equation (4b) for computing the shear resistance does not represent the block shear failure by shear (and tensile) rupture.

Table 2 shows the *average* professional factors of the alternative equations for each set of laboratory test results. The variable N denotes the number of specimens failing by block shear in each set.

It can be seen from Table 2 that Equation (2) specified in the European steel structures code (ECS 2005) is excessively conservative even on the basis of the average professional factors. Equation (3) specified in the Canadian steel structures standard (CSA 2009), on the other hand, is over-optimistic for many of the tested specimens (see also Table 3).

Table 2. Average professional factors

	N	(1) AISC 2010a	(2) ECS 2005	(3) CSA 2009	(4b) Proposed
Udagawa & Yamada (1998)	73	1.04	1.38	0.94	1.03
Aalberg & Larsen (1999)	8	1.01	1.41	0.91	1.03
Rabinovitch & Cheng (1993)	5	1.01	1.36	0.93	1.01
Huns et al. (2002)	5	1.18	1.53	1.05	1.11
Mullin (2002)	5	1.14	1.44	1.00	1.01
Hardash & Bjorhovde (1985)	28	1.18	1.50	1.06	1.06

Based on the average professional factors, Equation (1) given in the AISC specification for the shear yielding and tensile rupture mechanism appears to be rather accurate. In fact, it would appear from Table 2 that Equation (1) was more accurate than Equation (4b) for the specimens tested by Aalberg & Larsen (1999).

However, examination of the individual professional factors reveals a very different outcome. Table 3 shows that, for every single specimen tested by Aalberg & Larsen (1999), Equation (4b) is consistently more accurate than Equation (1). The variable n_r denotes the number of bolt rows in a specimen.

Table 3. Individual professional factors for specimens of Aalberg & Larsen (1999)

	n_r	(1) AISC 2010a	(2) ECS 2005	(3) CSA 2009	(4b) Proposed
T7	2	1.21	1.59	1.05	1.12
T8	2	0.90	1.21	0.89	1.05
T9	3	1.18	1.65	1.01	1.07
T10	3	0.86	1.22	0.84	1.01
T11	4	1.13	1.64	0.96	1.02
T12	4	0.82	1.20	0.80	0.96
T15	3	1.12	1.56	0.95	1.01
T16	3	0.83	1.18	0.82	0.98
	Mean	1.01	1.41	0.91	1.03
	COV	0.169	0.154	0.100	0.049

Table 3 shows that, while the average professional factor of Equation (1) is close to unity (1.01), it underestimates the strength of specimen T7 by 17% (1/1.21 = 0.83) but overestimates that of specimen T12 by 22% (1/0.82 = 1.22). It can also be seen from the table that Equation (4b) is much more accurate and consistent than all the code equations, which can overestimate or underestimate the block shear capacity considerably.

Table 4 shows the results of using the complete block shear equation prescribed in the current AISC specification (AISC 2010a)

$$R_n = \min\left(F_u A_{nt} + 0.6 F_u A_{nv};\quad F_u A_{nt} + 0.6 F_y A_{gv}\right) \tag{5}$$

Table 4. Average professional factors of AISC (2010a)

	N	$R_n = \min\left(F_u A_{nt} + 0.6 F_u A_{nv};\quad F_u A_{nt} + 0.6 F_y A_{gv}\right)$
Udagawa & Yamada (1998)	73	1.18 (0.99 - 1.38)
Aalberg & Larsen (1999)	8	1.20 (1.14 - 1.25)
Rabinovitch & Cheng (1993)	5	1.17 (1.10 – 1.25)
Huns et al. (2002)	5	1.26 (1.19 – 1.32)
Mullin (2002)	5	1.14 (1.09 – 1.19)
Hardash & Bjorhovde (1985)	28	1.20 (1.07 – 1.38)

By comparing the results of Equation (5) shown in Table 4 against those of Equation (1) shown in Table 2, it becomes evident that the first term of Equation (5), which is based on the simultaneous shear and tensile rupture mechanism, virtually determines the block shear capacity of all specimens since it leads to the lower computed capacities. In any case, it can be seen that the current AISC block shear equation is over-conservative even on the basis of average professional factors.

5. RELIABILITY ANALYSIS AND RESISTANCE FACTOR

The reliability analysis methodology and the statistical parameters used in the present work have been adopted from Driver et al. (2006), who determined the required resistance factor ϕ using the equation proposed by Fisher et al. (1978)

$$\phi = \left(0.0062\beta^2 - 0.131\beta + 1.338\right) M_m F_m P_m e^{-p} \tag{6}$$

in which β is the target reliability index, M_m is the mean value of the material factor equal to 1.11 (Schmidt & Bartlett 2002), F_m is the mean value of the fabrication factor equal to 1.00 (Hardash & Bjorhovde 1985), and P_m is the mean value of the relevant professional factor computed for all the 124 specimens listed in Table 2.

The exponential term p in Equation (6) is computed from

$$p = \alpha_R \beta \sqrt{V_m^2 + V_F^2 + V_P^2} \tag{7}$$

in which α_R is the separation variable equal to 0.55 (Ravindra & Galambos 1978), V_M is the coefficient of variation of the material factor equal to 0.054 (Schmidt & Bartlett 2002), V_F is the coefficient of variation of the fabrication factor equal to 0.05 (Hardash & Bjorhovde 1985), V_P is the coefficient of variation of the relevant professional factor computed for all the 124 specimens listed in Table 2.

Table 5 shows the resulting reliability indices β of the code and proposed equations if the resistance factor ϕ of 0.75 prescribed in the AISC specification (AISC 2010a) is used. The “acceptable” values for β range from 4 to 5 (AISC 2010b).

Table 5. Reliability analysis results of code and proposed equations

	(5) AISC 2010a	(2) ECS 2005	(3) CSA 2009	(4b) Proposed
P_m	1.19	1.41	0.96	1.04
V_P	0.051	0.093	0.074	0.058
β; ϕ = 0.75	6.1	6.7	4.2	5.0
ϕ; β = 4.0	0.99	1.10	0.78	0.86

It can be seen from Table 5 that the current AISC provision for the limit state of block shear failure is very conservative, resulting in a reliability index β of 6.1. The resistance factor ϕ of 0.75 has remained the same since the first LRFD edition (AISC 1986), even though the block shear equations have changed as shown in Table 1. The application of the resistance factor ϕ of 0.75 to the first LRFD block shear equation (AISC 1986) resulted in a reliability index β of 4.5 for the same set of test results.

Table 5 also shows the required resistance factor ϕ for each equation if the target reliability index $\boldsymbol{\beta}$ is set to be 4.0 (AISC 2010b). Consistent with the finding of Driver et al. (2006), it was found that the required resistance factor ϕ for Equation (5) is close to unity. This outcome does not demonstrate the superiority of Equation (5), but merely indicates the conservatism of the existing resistance factor ϕ of 0.75 for bolted connections in gusset plates.

For the same level of procedural complexity, the design equation should be the one which is the most *consistently* accurate among available alternatives, as reflected in the individual professional factors. A good rule of thumb may be that the average professional factor should fall between 0.95 and 1.05, with a standard deviation not greater than 0.10. However, Equation (3), which satisfies this rule, overestimates the capacity of 33 specimens by more than 10% (and that of 5 specimens by 20% or more). Equation (4b), on the other hand, only overestimates the capacity of one specimen by more than 10% (20% for an outlier specimen for all equations).

While in theory the application of capacity factors ϕ equal to 0.78 and 0.86 to Equations (3) and (4), respectively, would result in the same reliability index $\boldsymbol{\beta}$ equal to 4.0, it is evident from Table 3 and the facts in the preceding paragraph that Equation (3) leads to significantly more varied safety margins across connections of different configurations, notwithstanding the aim of the LRFD approach to achieve consistent reliability in structural design. The same is true for the other code equations. The implication is that, while the code equations may or may not be unsafe, they are (in conjunction with the corresponding capacity factors) too conservative for many if not most connections. Reliance on the use of an “appropriate” capacity factor as determined from a reliability analysis to compensate for a grossly inaccurate equation is not ideal for achieving efficient designs.

6. SUMMARY AND CONCLUSIONS

Among the various mechanisms for conventional block shear failures postulated in the literature and anticipated in the design codes, there is only one feasible mechanism, that which involves shear yielding and tensile rupture. The physical reasoning described by the authors explains why extensive published laboratory tests of hot-rolled steel bolted connections have never found a block shear failure caused by any other mechanisms anticipated in the steel design codes around the world.

The shear yielding planes in a block shear failure, termed the active shear planes, lie midway between the gross and the net shear planes assumed in the AISC specification for shear yielding and for shear rupture, respectively. In reality, the shear failure planes are unique and do not depend on the prevalent mechanism of block shear failure.

The use of the net shear area in the European steel structures code in conjunction with the shear yield stress leads to excessive conservatism in determining the block shear capacity of hot-rolled steel bolted connections. This conservatism is somewhat tempered in the AISC specification by the use of the shear ultimate stress instead of the shear yield stress (when the net shear area is used to compute the shear rupture resistance component), but the resulting equation is still quite conservative with more than 25% underestimation in some cases. The significant conservatism of the current European and American code equations is also reflected in the reliability analysis results presented by the authors and Driver et al. (2006).

For bolted connections in hot-rolled steel plates, full or almost full strain hardening takes place along the shear yielding planes. This means that the shear ultimate stress rather than the shear yield stress should be used in computing the shear yielding resistance component in conjunction with the active shear area.

The use of the gross shear area in conjunction with the shear yield stress in the AISC specification for the shear yielding and tensile rupture mechanism led to overestimation and underestimation by 20% or more of the block shear capacity.

The use of the gross shear area in conjunction with the mean between the shear yield stress and the shear ultimate stress in the Canadian steel structures standard led to overestimation by up to 25% (1/0.80 = 1.25) of the tested capacity.

Reliability analyses can only ensure that the same reliability index is achieved for a group of different design equations, but cannot achieve a consistent safety margin across the components being designed when an inaccurate equation is used.

The proposed equation, which makes use of the active shear area in conjunction with the shear ultimate stress, has been demonstrated to provide the most consistent and accurate results in determining the block shear capacities of bolted connections in hot-rolled steel gusset plates tested by various researchers around the world. A resistance factor equal to 0.85 is recommended for use with the proposed equation to ensure a reliability index of 4.0 is achieved.

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