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Ultimate shear-out capacities of structural steel bolted connections

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Ultimate Shear-out Capacities of Structural Steel Bolted Connections

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Abstract:

Based on the previous research results of the authors, this paper presents an accurate and consistent equation for determining the ultimate shear-out capacity of a structural steel bolted connection. The equation is verified against independent laboratory test results obtained by other researchers around the world. Comparisons against alternative equations found in the design specifications and literature are also included. The paper explains why certain equations appear to be accurate for particular configurations, but are grossly inaccurate for others. The various assumptions embedded in the existing equations, some optimistic and others pessimistic, are described. It is shown that the current code equations lead to very significant errors on either side of conservatism, while the proposed equation is consistently accurate for all test specimens known to fail in shear-out. A resistance factor of 0.85 is recommended for the proposed equation in order to achieve a reliability index of 4.0. The use of the proposed equation instead of the current AISC specification's equation will facilitate structural designs that are more economical yet reliable.

Subject headings: bolted connections, structural design, structural steel, thin wall sections

Author keywords: bearing strength, connection capacity, shear-out, tear-out

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Introduction

In Section J3.10 of the AISC Specifications for Structural Steel Buildings (AISC 2010a), the shear-out (also termed tear-out) failure mode of a bolted connection is treated as a special case of the bearing failure mode. Conversely, according to Kim et al. (2008), the bearing failure mode is treated as a special case of the shear-out failure mode in the Japanese steel structures code (AIJ 2002). However, in reality, the shear-out failure mode depicted in Figure 1(a) is distinct from the bearing failure mode depicted in Figure 1(b). Photographs of laboratory specimens showing these two failure modes can be found in Teh & Clements (2012). The two failure modes are hence treated as distinct from each other in this paper.

Bolt hole deformation at service load is not a concern in the present work. Salih et al. (2011) have stated that the deformation based definition of failure has led to inconsistency since the failure loads depend on an often arbitrary selection of a limiting deformation. Aalberg & Larsen (2001) have also commented that the theoretical background to the deformation limit of 6.35 mm used in the AISC specification is unclear. In the present work, the shear-out strength is understood to be the ultimate test load achievable when the shear-out failure mode governs the load-carrying capacity of the bolted connection.

Based on the active shear planes described by Clements & Teh (2013) in the context of the block shear failure mode, and verified by Teh & Uz (2013) against independent laboratory test specimens subjected to pure shear, an accurate and consistent equation is proposed in this paper for determining the shear-out capacity of a structural steel bolted connection. The equation will be verified against independent laboratory test results obtained by other researchers around the world. This paper also compares the performance of the proposed equation against alternative equations found in the design specifications and literature.

It will be explained why certain equations appear to be accurate for particular connection configurations, but are grossly inaccurate for others. The various assumptions embedded in the existing equations, some optimistic and others pessimistic, are described. Discussions will be first presented for connections with a single row of bolts, such as that illustrated in Figure 2, including single-bolt connections.

Separate discussions are then presented for double-row bolted connections, for which a combined shear-out and bearing failure mode can be mistaken as a pure shear-out failure mode. It will be explained that whether an upstream bolt would fail in bearing or shear-out of the connected plate depends only on the spacing between itself and the bolt downstream, and not on the relative strength between the computed total shear-out capacity and bearing capacity for all bolts.

Based on the laboratory test results of single- and double-row bolted connection specimens failing in ultimate shear-out obtained by independent researchers around the world, a resistance factor will be computed for the most accurate and consistent equation.

Existing equations for ultimate shear-out capacity

When bolt hole deformation is not a concern, section J3.10 of the AISC Specifications for Structural Steel Buildings (AISC 2010a) specifies the shear-out capacity P_p of a bolted connection to be (for each line of bolts)

$$P_p = 1.5 L_{nv} t F_u \tag{1}$$

in which L_{nv} is considered to be the "net shear length" in the literature (Teh & Clements 2012). For a single-row bolted connection such as that illustrated in Figure 2, L_{nv} is the clear distance between the edge of the bolt hole and the plate end, denoted e_n in Figure 2. The variable F_u denotes the material tensile strength of the steel plate, and t is the plate thickness. It should be noted that there are two shear failure planes before each bolt, as illustrated in Figure 3 for the gross and active shear planes. The shear coefficient implicit in Equation (1) is therefore 0.75, which is significantly greater than the well-established value of 0.6.

The use of the clear distance between the edge of the bolt hole and the plate end, $L_{nv} = e_n$, is an approximation since the two shear failure planes cannot coincide with the centreline of the bolt hole, or with each other. The definition has been used in the current AISC specification for the sake of simplicity in dealing with circular and slotted bolt holes (AISC 2010b), as discussed in the section “Circular and slotted bolt holes”.

Section E6.1 of the North American Specification for the Design of Cold-formed Steel Structural Members (AISI 2012) specifies an ultimate shear-out capacity that is 20% lower

$$P_p = 1.2 L_{nv} t F_u \tag{2}$$

Equation (2) is based on the well-established shear coefficient of 0.6. In fact, the shear coefficient of 0.6 has always been used in the AISC specifications (AISC 1993, 2010a) for determining the ultimate strength of elements in shear, as found in section J4.2 of the latest specification. There is an inconsistency with regard to the shear coefficient between equation (J3-6b) of the current AISC specification (AISC 2010a), or Equation (1), and the shear rupture equation (J4-4) of the same specification.

As an aside, Equation (2) also coincides with the AISC specification (AISC 2010a) for determining the shear-out capacity when bolt hole deformation at service load is a concern. However, the cold-formed steel specification (AISI 2012) does not provide separate shear-out equations for considering and for neglecting bolt hole deformation at service load, although it does so for the bearing failure mode.

The earlier AISC specification (AISC 1993) makes use of the gross shear planes depicted in Figure 3(a) for a single-bolt connection, which are measured from the centre of the bolt hole to the plate end, and implicitly assumes a reduced shear coefficient of 0.5 in determining the ultimate shear-out capacity as given in equation (J3-1b) or (J3-2a) of the specification

$$P_p = L_{gv}\, t F_u \tag{3}$$

Cai & Driver (2010) proposed an alternative equation based on the gross shear planes that assumes partial shear strain hardening

$$P_p = L_{gv}\, t\left(\frac{F_u + F_y}{\sqrt{3}}\right) = 1.155\, L_{gv}\, t\left(\frac{F_u + F_y}{2}\right) \tag{4}$$

in which F_y is the yield stress of the steel plate. Equation (4) uses the von Mises coefficient.

For the purpose of the present work, the following equation based on yielding along the gross shear planes is included for comparison against laboratory test results

$$P_p = 1.2\, L_{gv}\, t F_y \tag{5}$$

Proposed equation for ultimate shear-out capacity

Clements & Teh (2013) have shown through contact finite element analysis that the shear failure planes are not the gross shear planes assumed in Equations (3) through (5), and do not coincide with the centreline of the bolt hole as implied by Equations (1) and (2). Rather, the so-called active shear planes lie midway between the two extremes, as depicted in Figure 3(b) for a single-bolt connection. The active shear planes, in conjunction with the well-established shear coefficient of 0.6, have been verified by Teh & Uz (2013) against independent laboratory test results obtained by other researchers (Gross et al. 1995, Orbison et al. 1999).

Teh & Uz (2013) have also shown that, when the applied load is resisted by shear only, the shear strain hardening reserve of the hot-rolled steel plate is largely exhausted at the ultimate limit state. The shear-out capacity should therefore be computed using the material tensile strength F_u rather than the yield stress.

Based on the results of Clements & Teh (2013) and Teh & Uz (2013), the following equation is proposed for determining the ultimate shear-out capacity of a bolted connection (for each line of bolts)

$$P_p = 1.2 L_{av} t F_u \tag{6}$$

Although the definition of the active shear planes in the present work is the same as that in Teh & Clements (2012), no approximation to the active shear length L_{av} is used in the present work, as evident in Figure 3(b). The reasons are two-fold.

Firstly, in the specimens tested by Teh & Clements (2012) and by other researchers against which the active shear planes have been verified (Teh & Yazici 2013, Teh & Uz 2013), the neglected portion of the active shear length was relatively insignificant compared to its total length. The neglected portion was equal to a quarter of the bolt hole diameter, while the total active shear length was typically multiple times the bolt hole diameter. In contrast, as will be seen in the next section, the total active shear length of the single-row bolted connection specimens in the present work are close to or even shorter than the bolt hole diameter. Secondly, unlike a specimen failing in block shear, there is no tensile resistance component contributing to the capacity of a specimen failing in shear-out, which would otherwise mask the error associated with the neglected portion of the active shear length.

Single-row bolted connection specimens

This section includes single-row two-bolt connections (see Figure 2) and single-bolt connections which failed in the shear-out mode. It should be noted that some researchers have included the specimens tested by others which had been identified by the original authors to have failed in modes other than shear-out, and such specimens are excluded from this section.

For consistency with the results reported by the original researchers, as much as possible the present work uses the values reported by the researchers for the dimensions and material properties of the test specimens. The measured values are always used for the material properties, which is important since measured yield stresses and tensile strengths can differ from the nominal values by more than 10%. The measured geometric dimensions are also used to 0.1 mm accuracy where available, otherwise the nominal values are used. However, unlike the material properties, the errors are typically within 5%.

The percentages of overestimation reported in this paper have been calculated using more precise professional factors than those shown in the following tables, which are given in two decimals. Therefore, an overestimation of either 14% (eg. 1/0.878) or 13% (eg. 1/0.883) may be reported for a professional factor given as 0.88 in the tables.

An empty cell in the following tables indicates that the data in the cell above applies.

Specimens tested by Puthli & Fleischer (2001)

Table 1 shows the results of Equations (1) through (6) for the nine shear-out specimens tested by Puthli & Fleischer (2001), which were single-row two-bolt connections similar to that illustrated in Figure 2. There was only one connection configuration relevant to the determination of the shear-out capacity. The clear end distance e_n was 21 mm, or 0.7 times the bolt hole diameter of 30 mm.

The results in Table 1 indicate that Equations (5) and (6) are the two most accurate for the shear-out specimens tested by Puthli & Fleischer (2001), each with a mean professional factor P_t/P_p equal to or only slightly larger than unity. However, since there was only one configuration tested, the apparently accurate results shown for the first nine specimens in Table 1 have to be interpreted with caution.

Nevertheless, the test results of Puthli & Fleischer (2001) show that Equations (1) and (2) are too conservative for these specimens, which had a relatively narrow clear end distance e_n. As stated previously, these two equations assume the net shear failure planes.

Despite the assumption of partial shear strain hardening only and the use of a shear coefficient equal to 0.577 instead of 0.6, Equation (4) overestimates the shear-out capacity of the first nine specimens by up to 13% (i.e. 1/0.89 for specimen 153x400b). This outcome is due to the use of gross shear planes.

Specimens tested by Kim & Yura (1999)

For each specimen, Kim & Yura (1999) provided the applied load corresponding to their deformation limit of 6.35 mm as well as the ultimate test load. As indicated in the introduction to this paper, the present work is only concerned with the ultimate test load, against which all equations are verified for accuracy.

The fact that the performance of Equation (5) for the first nine specimens, tested by Puthli & Fleischer (2001), was coincidental is evident from its results for the single-bolt specimens tested by Kim & Yura (1999), also shown in Table 1. The excessive underestimations by Equation (5) for the latter’s first three specimens, from 29% to 41%, despite the use of gross shear planes, is due to the neglect of shear strain hardening. On the other hand, the significant overestimation for specimen BO050R despite the neglect of shear strain hardening indicates that the actual shear failure planes are smaller than the gross shear planes.

As pointed out by Teh & Clements (2012), the error due to the use of gross shear planes and that due to the neglect of shear strain hardening may offset each other completely in certain cases. However, depending on the relative influence between the two factors in a configuration, they often lead to significant overestimations and underestimations, respectively. The neglect of strain hardening is more pronounced for materials with high ratios of ultimate tensile strength to yield stress F_u/F_y.

The aforementioned phenomenon is also true to a lesser extent for Equation (4), which also uses the gross shear planes but assumes partial shear strain hardening. Although it has a mean professional factor equal to unity for the single-bolt specimens of Kim & Yura (1999), it overestimates the capacity of specimen BO050R by 14% and underestimates that of specimen AO100 by 12%. The resulting coefficient of variation is much larger than that of Equation (6), as shown in Table 1.

Likewise but for a different reason, Equation (1) overestimates the capacity of specimen BO200R by 13% but underestimates that of specimen AO050R by 38%. The former specimen had the longest end distance, while the latter had the shortest end distance. The use of an inflated shear coefficient equal to 0.75 tends to overestimate the capacity, while the use of net shear planes tends to underestimate same. The neglected portion of the active shear length is more significant for short end distances.

The conservatism of Equation (2) is obvious in this case. It underestimates the shear-out capacities of specimens AO050R and BO050R by 72%. Unlike Equation (1), the use of net shear planes is not compensated by the use of an inflated shear coefficient higher than 0.6.

It is interesting to note that Equation (3), found in the 1993 AISC specification (AISC 1993), turned out to be much more accurate than its replacement, Equation (1), which first appeared in the 1999 specification (AISC 1999), the same year as the publication of Kim & Yura (1999). The significantly less accurate equation was favoured due to its perceived simplicity (AISC 2010b), as described in the section “Circular and slotted bolt holes”.

In any case, Equation (6) is again shown to be accurate for the shear-out specimens tested by Kim & Yura (1999).

Specimens tested by Aalberg & Larsen (2001)

The third series of specimens in Table 1 were tested by Aalberg & Larsen (2001). Some of these specimens were composed of very high strength steels, with ultimate tensile strengths greater than 870 MPa and up to 1440 MPa. Four of the specimens (two nominal geometrical configurations only) tested by Aalberg & Larsen (2001) are not included in Table 1 and are discussed in the next section.

As for the specimens tested by Puthli & Fleischer (2001) and Kim & Yura (1999), Equation (6) is reasonably accurate for the twelve single-bolt specimens tested by Aalberg & Larsen (2001). Although Equation (3) has a mean professional factor equal to unity, it overestimates the capacity of specimen W1100-1 by 13%, which was accurately determined using Equation (6). The resulting coefficient of variation is somewhat higher than that of Equation (6).

The reason for the inconsistency of Equation (3) is opposite to that for Equation (1), and is similar to those for Equations (4) and (5). It uses the gross shear planes but assumes a reduced shear coefficient of 0.5.

Equation (1) overestimates the capacity of specimen W700-3 by 16% but underestimates that of specimen W700-1 by 31%. It is interesting to note that the capacity of specimen W700-1 was instead overestimated by Equations (4) and (5) by about 25%. The assumptions of net shear planes in Equation (1) and of gross shear planes in Equation (4) and (5) are more pronounced for short end distances. On the other hand, Equations (4) and (5) underestimate the capacity of specimen S355-3a by 10% and 26%, respectively, which had much larger end distances and happened to be accurately determined using Equation (1). Such inconsistent outcomes are due to the contrasting assumptions of these equations, as listed in Table 2.

The largest overestimations for the single-bolt specimens of Aalberg & Larsen (2001) are those by Equations (4) and (5) for W1100-1, equal to 26%. The ultimate load of this specimen was instead underestimated by Equations (1), (2) and (3) by 25%, 56%, and 13%, respectively. Equation (6), in contrast to all other equations, accurately determines the shear-out capacity.

As for all other series, Equation (2) is too conservative for the specimens tested by Aalberg & Larsen (2001) shown in Table 1. There is no optimistic assumption in the equation, as evident from Table 2.

Specimen tested by Rex & Easterling (2003)

Of the 48 specimens tested by Rex & Easterling (2003), only one was identified by them to have failed in shear-out, which they termed tear-out. The results for this specimen are shown in the last row of Table 1. Equation (6) is again the most accurate among all equations, with excessive underestimations by Equations (1), (2) and (5), and a significant overestimation by Equation (3).

Outlier specimens of Aalberg & Larsen (2001)

Table 3 shows the results for four single-bolt specimens tested by Aalberg & Larsen (2001), for which the ultimate test loads were found to be unusually high or low compared to the estimates across the six equations, especially with regard to Equation (6) that has been shown in the preceding section to give consistently accurate estimates. These four specimens are two pairs of two nominal geometrical configurations, as evident from Table 3.

The results for specimens S355-1a and S355-1b, which had the same nominal configuration, show the largest underestimations ever by Equations (1), (2) and (6). The reason for this outcome is uncertain, although catenary action by the narrow strip downstream of the bolt is a distinct possibility. Specimen AO050R in Table 1, which also had the same nominal end distance equal to half the bolt diameter, is the next most underestimated by these equations.

The ultimate test loads of specimens W700-4 and W1100-4 were significantly lower than their shear-out capacities predicted by all equations except for Equation (2), which has been shown in the preceding section to be excessively conservative. It is noteworthy that the professional factors of every equation for the two specimens, which had the same nominal geometrical configuration but different material properties, are similar to each other.

However, the reason for the overestimations by Equation (6) in this case is not the geometrical configuration, since specimens S355-4a and S355-4b in Table 1 also had this nominal configuration yet their shear-out capacities were accurately predicted by Equation (6). The only significant variable is that the material tensile strengths of W700-4 and W1100-4 are extremely high, at 871 MPa and 1440 MPa, respectively. It was likely that full shear strain hardening could not be achieved throughout the entire shear planes prior to fracture in these specimens, which had the longest active shear length, due to the lack of material ductility. In light of these results, similar specimens are not included in the next section.

Double-row bolted connection specimens

The bearing capacity of a bolt provided by the connected steel plate is most commonly expressed as

$$P_b = Cd\,tF_u \tag{7}$$

in which C is the bearing coefficient and d is the bolt diameter. According to Section J3-6b of the AISC Specifications for Structural Steel Buildings (AISC 2010a), C is equal to 3.0 when deformation at the bolt hole is not a concern, which is the case in the present work. This value is 20% higher than the corresponding value of 2.5 specified in Eurocode 3 (ECS 2005).

Nevertheless, assuming that the bearing coefficient C is equal to 3.0, the threshold value for the shear plane length l_s beyond which a shear-out failure would not take place before bearing failure can be estimated as follows

$$1.2l_s\,tF_u > 3.0d\,tF_u \Rightarrow l_s > 2.5d \tag{8}$$

in which the shear plane length is represented by a generic variable l_s, even though it has been demonstrated in an earlier section to be the active shear length L_{av}.

Leaving out the net section tension fracture mode and the block shear failure mode for the connected steel plate, the shear-out failure mode governs the single-bolt connection in Figure 4(a) while the bearing failure mode governs the single-bolt connection in Figure 4(b). This assertion is likely to be obvious to all readers.

However, it is less obvious that the upstream bolt (the lower one) in Figure 4(c) would not fail in shear-out even if the total shear length, $l_{s1} + l_{s2}$, is less than 5.0 d (there are two bolts). Irrespective of the downstream shear length l_{s1}, the plate material resisting the upstream bolt would fail in bearing rather than shear-out because the available shear-out capacity of the material between the two bolts is greater than the bearing capacity. This fact is best explained using the free body diagram shown in Figure 5 for the material between the two bolt holes.

The upward action resulting from the bearing stresses at the lower (upstream) bolt hole is resisted by the shear stresses acting downward along the two shear planes. Whether the ultimate limit state of bearing failure at the upstream bolt hole is able to take place or not depends only on the length of the two shear planes relative to the threshold value represented by Equation (8). It does not depend on the shear planes beyond the downstream (upper) bolt.

If the downstream shear length l_{s1} in Figure 4(c) is smaller than 2.5 times the bolt diameter, which is often the case, and the upstream shear length l_{s2} is larger than the threshold value, which is likely the case in practice since the preferred minimum bolt spacing is three times the bolt diameter (AISC 2010a), then the bolted connection would undergo the combined shear-out and bearing failure mode, which is outside the scope of this paper.

Therefore, for a double-row bolted connection to fail in pure shear-out, the bolt pitch (spacing in the direction of loading) cannot be greater than approximately 2.5 times the bolt diameter. It should also be noted that connections with three or more rows of bolts generally fail in neither shear-out nor bearing, since the most likely failure modes are net section tension fracture and block shear failure. The net section tension capacity does not increase with the increase in the number of bolt rows, while the shear-out and bearing capacities increase with the number of bolt rows (and bolts), as demonstrated by Teh & Gilbert (2012).

Table 4 shows the results of the first six equations for double-row bolted connections which failed in shear-out. The definitions of the net, gross and active shear lengths for such connections are given in Figure 6. However, the earlier AISC specification (AISC 1993) has different definitions for the upstream and downstream bolts, as depicted in Figure 7.

According to the 1993 AISC specification (AISC 1993), the ultimate shear-out capacity of a double-row bolted connection (for each line of bolts) is

$$P_p = L_{mv} \, t F_u \tag{9}$$

in which the mixed shear length L_{mv} is defined in Figure 7. The shear length for the upstream bolt, l_{mv2}, happens to be equal to its active shear length in magnitude. Both Equations (3) and (9) are included in Table 4 and discussed in the following.

Pursuant to the finding discussed in the preceding section, Table 4 does not include specimens with tensile strength of 870 MPa or greater. The (nominal) bolt pitch of all specimens was 40 mm, except for the last specimen, A123, for which it was 50 mm.

It can be seen from Table 4 that the results of Equations (1), (2) and (5) for the double-row bolted connection specimens are largely in line with those for the single-row ones listed in Table 1, discussed previously.

However, the optimistic assumption of Equation (3), in the form of the gross shear planes, is exacerbated for the double-row bolted connections. It overestimates the shear-out capacity of specimen AT0510R by 19%, and in general overestimates the ultimate loads of the double-row bolted connection specimens. This outcome is despite the assumption of a reduced shear coefficient equal to 0.5.

Similarly, due to the use of gross shear planes, Equation (4) overestimates the ultimate load of specimen "124.8" by 18% despite the assumption of partial shear strain hardening only. It overestimates the capacities of five double-row bolted connection specimens in the order of 15%.

Equation (9), which is in accordance with the earlier AISC specification (AISC 1993), is generally conservative, with an underestimation of 19% for specimen “122.6”. On average, it underestimates the shear-out capacities by 10%.

Equation (6) turns out to give a mean professional factor equal to unity for each of the double-row bolted connection series tested by Kim & Yura (1999), Aalberg & Larsen (2002) and Udagawa & Yamada (1998, 2004), with reasonably low coefficients of variation.

The ranges of professional factors given by the seven equations for the single- and double-row bolted connection specimens are shown in Table 2. It can be seen that Equation (6) is by far the most accurate one for determining the ultimate shear-out capacity of a bolted connection. It is the only one that is accurate within 10% on either side of conservatism for each of the single- and double-row bolted connection specimens tested by independent researchers around the world. The mean professional factor of Equation (6) for the single- and double-row specimens is 1.01 with a coefficient of variation equal to 0.047.

Resistance factor

Although the authors have some reservations concerning the reliability analysis procedure as often conducted in the literature due to the potential misuse, including the determination of the resistance factor, this section has been included based on the current practice in the field. 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)

$$\phi = \left(0.0062\beta^2 - 0.131\beta + 1.338\right) M_m F_m P_m e^{-p} \tag{10}$$

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 professional factor.

The exponential term p in Equation (10) is computed from

$$p = \alpha_R \beta \sqrt{V_m^2 + V_F^2 + V_P^2} \tag{11}$$

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 professional factor.

It was found that, in order to achieve the target reliability index β of 4.0 (AISC 2010b), a resistance factor ϕ of 0.85 is required for Equation (6).

Teh & Yazici (2013) have described some of the pitfalls in interpreting the accuracy or reliability of a design equation from the computed resistance factor, which are not recapitulated in this paper. In the present case, the computed resistance factors for Equations (1) and (3) are 0.81 and 0.80, which are almost the same. However, an examination of their professional factors in Tables 1 and 4 reveals that they perform very differently.

The mean professional factor of Equation (1) is 1.09 with a coefficient of variation equal to 0.122, while the corresponding values for Equation (3) are 0.97 and 0.058. The "overall" statistical results of Equation (3) appear to be excellent, but Tables 2 and 4 paint a different outcome. The choice of specimen configurations affects not only the average (mean) value, but also the coefficient of variation. It is also for this reason that no coefficient of variation is given for the specimens of Puthli & Fleischer (2001) in Table 1.

The authors believe that the appropriateness of a design equation should be as much as possible assessed firstly on its fundamental merits. Simplicity is of course important, but the design equation should ideally not lead to excessive errors on either side of conservatism in particular instances. Table 2 is useful in detecting such instances.

It should also be noted that a resistance factor equal or close to unity does not signify the accuracy of the equation. For example, the computed resistance factor for Equation (2) is 1.01, but it is clear from Tables 1 and 4 that the equation is grossly inaccurate for the hot-rolled steel specimens studied in the present work.

The resistance factor for a hypothetical equation that has a mean professional factor of unity and zero coefficient of variation is 0.86. In any case, the proposed Equation (6) will lead to structural designs that are more economical with consistent reliability, compared to all the other equations considered in this paper.

Circular and slotted bolt holes

As mentioned in an earlier section, the current AISC specification (AISC 2010a, b) replaces Equation (3) used in the earlier specification (AISC 1993) with Equation (1) for the sake of simplicity in dealing with circular and slotted bolt holes. The use of distances measured from the centres of bolt holes in Equation (3), which is tantamount to using the gross shear length as illustrated in Figures 3(a), is problematic when it comes to slotted bolt holes.

The bolted connection with a circular hole shown in Figure 8 and the one with a slotted hole have the same distance between the hole centre and the plate edge. Therefore, Equation (3) would give the same shear-out capacity for the two connections. However, it is obvious that the shear-out capacity of the slotted connection must be lower than the other. This “anomaly” is resolved in Equation (1) through the use of clear distances measured from the edges of bolt holes.

In light of the significantly inaccurate results given by the current specification Equation (1) as summarised in Table 2, it is noteworthy that the proposed Equation (6) does not have the problem of Equation (3) described in the preceding paragraph.

Conclusions

Seven alternative equations for determining the ultimate shear-out capacity of a bolted connection have been described, where bolt hole deformation at service load is not a concern. The optimistic and pessimistic factors embedded in the equations have been listed and discussed with respect to their effects on the accuracy of the corresponding equations. The existing code equations can lead to considerable errors on either side of conservatism.

The use of gross shear planes mostly leads to overestimations of the shear-out capacities by up to 25%, even when significant strain hardening is ignored or only partially accounted for. However, this assumption can be offset to a significant extent by the use of a reduced shear coefficient equal to 0.5, although overestimations close to 20% are still possible.

On the other hand, the use of net shear planes is generally too conservative, and is excessively so in most cases. However, the use of an inflated shear coefficient equal to 0.75 (in conjunction with the net shear planes) can result in significant overestimations of the ultimate shear-out capacities. The current AISC specification results in professional factors ranging from 0.88 to 1.38 for the specimens tested by Kim & Yura (1999), and is quite inaccurate for most specimens tested by other researchers around the world.

The proposed equation, based on the active shear planes and the well-established shear coefficient of 0.6, is the only one that is consistently accurate for the shear-out specimens tested by independent researchers around the world except for low ductility high-strength steel specimens. Furthermore, it does not lead to potential anomalies for slotted bolt holes. A resistance factor of 0.85 is recommended for use with the equation in order to achieve the target reliability index of 4.

Relative to the current AISC specification's equation, which has a resistance factor of 0.75 as specified in the code, the use of the proposed equation will facilitate structural designs that are more economical yet reliable.

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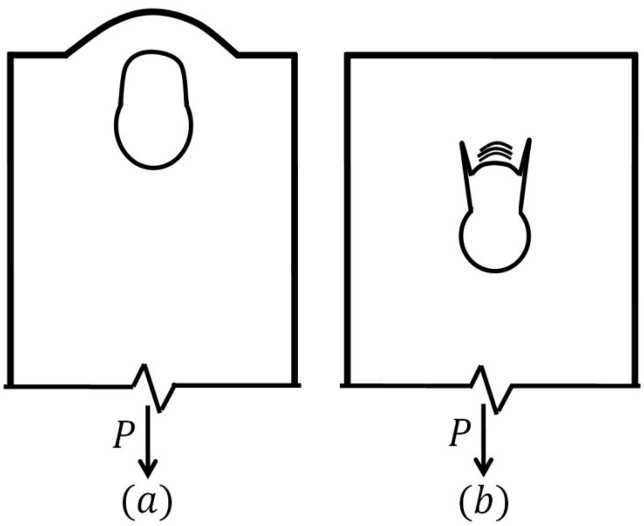

Figure 1 Two distinct failure modes: (a) Shear-out (or tear-out); (b) Bearing

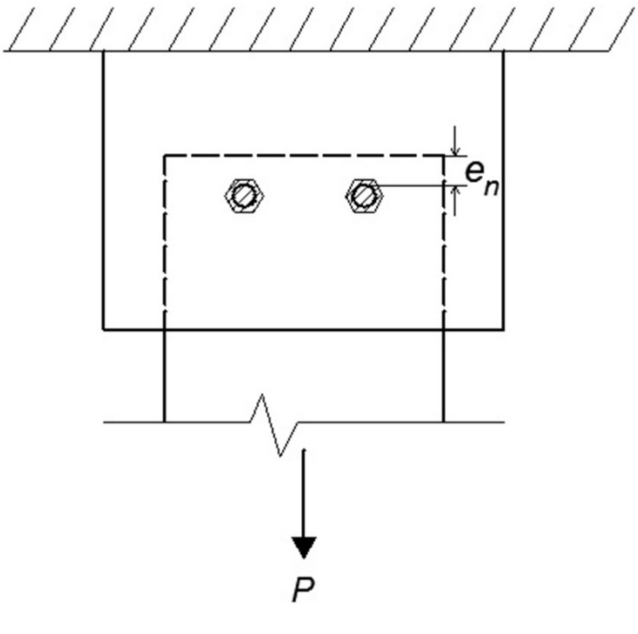

Figure 2 A single-row bolted connection

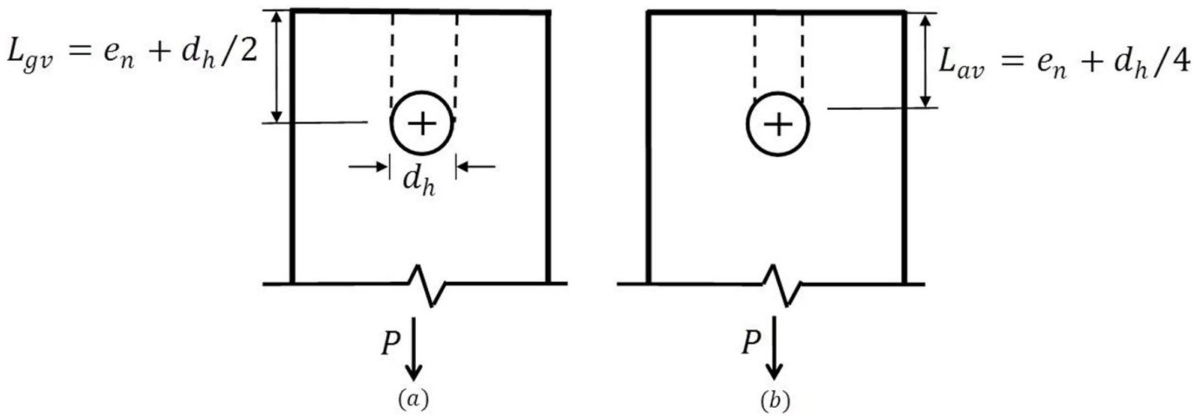

Figure 3 Definitions of shear failure planes: (a) Gross; (b) Active

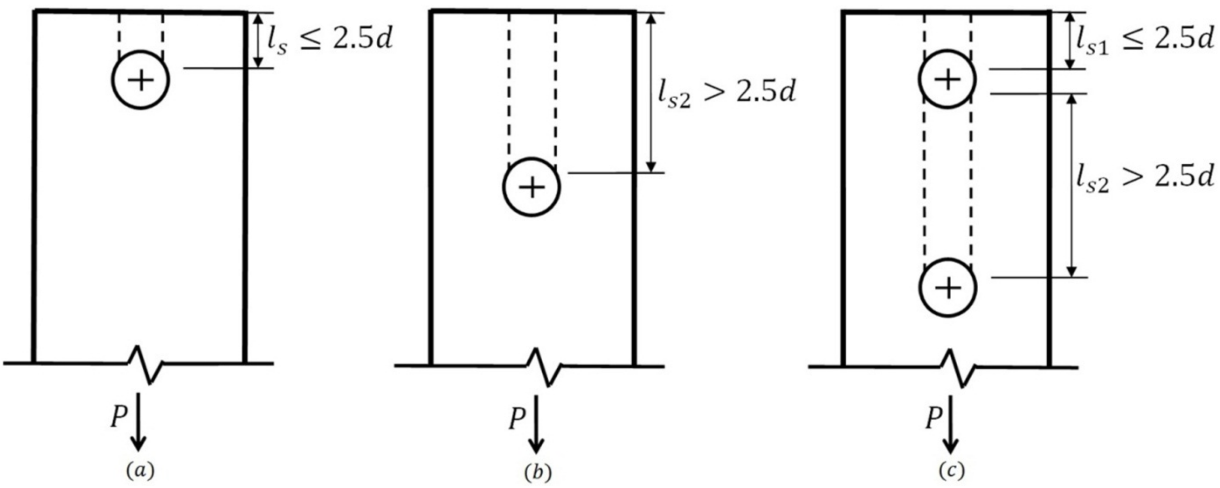

Figure 4 Threshold shear lengths: (a) Shear-out; (b) Bearing;; (c) Shear-out and bearing

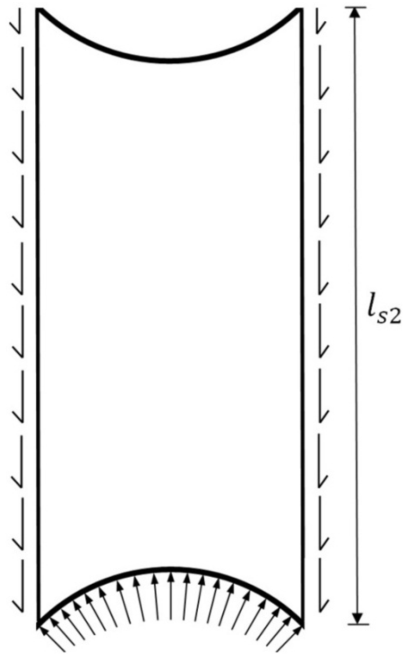

Figure 5 Free body diagram for the plate material downstream from an upstream bolt

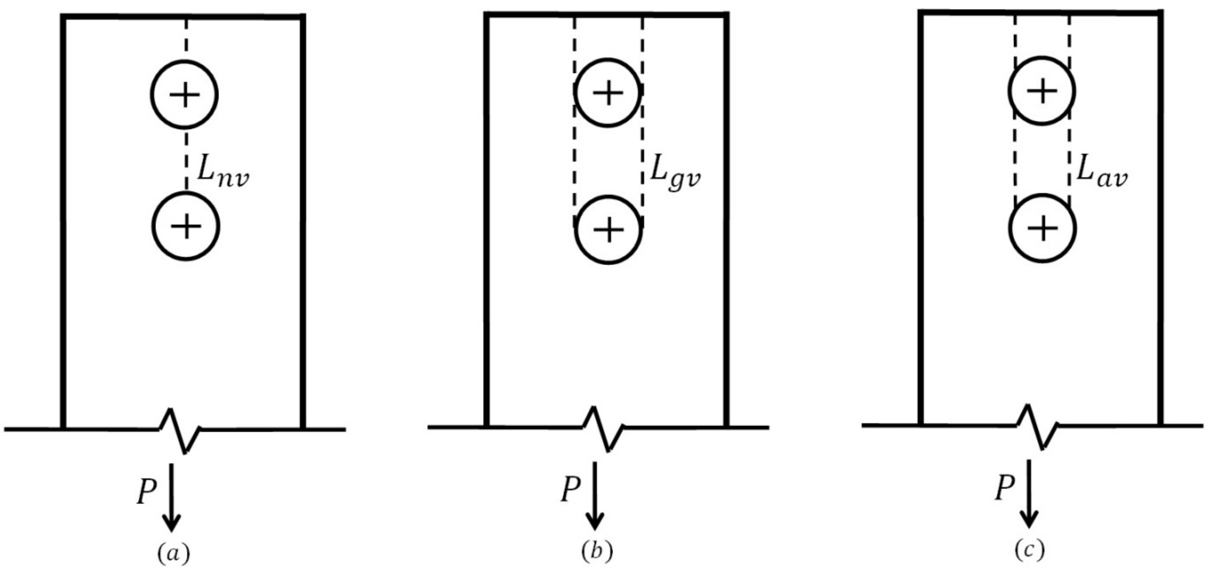

Figure 6 Definitions of net, gross and active shear lengths for a double-row bolted connection

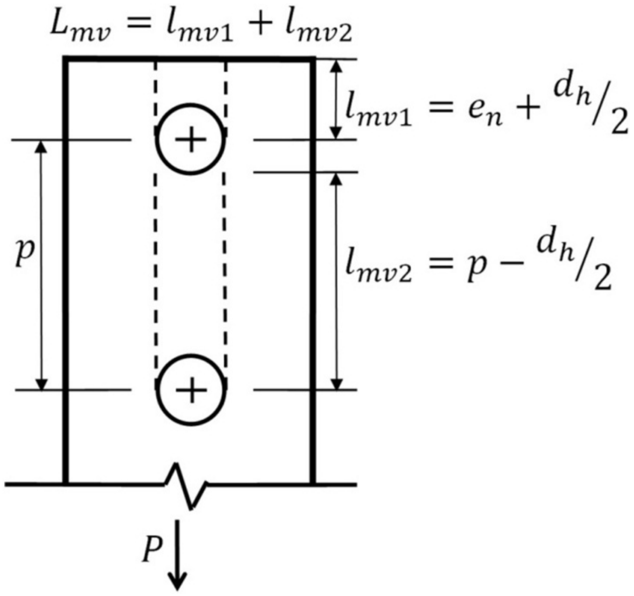

Figure 7 Mixed shear planes implied by AISC (1993)

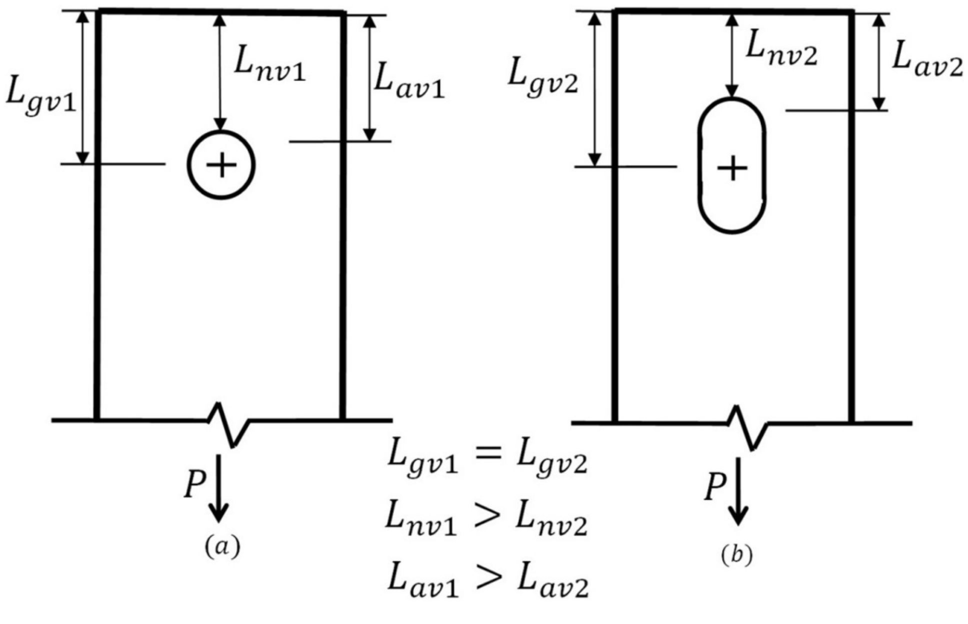

Figure 8 Problem with Equation (3) in using L_{gv}

Table 1 Results for single-row bolted connections

Specimen	d_h (mm)	e_n (mm)	t (mm)	F_y (MPa)	F_u (MPa)	P_t/P_p of Equations					
						(1)	(2)	(3)	(4)	(5)	(6)
144x400	30	21	17.5	524	645	1.15	1.44	1.01	0.96	1.03	1.06
153x400a						1.09	1.36	0.95	0.91	0.98	1.00
153x400b						1.06	1.33	0.93	0.89	0.95	0.98
162x400a						1.10	1.38	0.97	0.92	0.99	1.02
162x400b						1.09	1.36	0.95	0.91	0.97	1.00
171x450						1.08	1.36	0.95	0.91	0.97	1.00
162x550						1.14	1.43	1.00	0.95	1.02	1.05
171x550						1.13	1.41	0.99	0.94	1.01	1.04
180x550						1.14	1.43	1.00	0.96	1.03	1.05
Puthli & Fleischer (2001) – One configuration					Mean	1.11	1.39	0.97	0.93	1.00	1.02
AO050,R	21	9.1	4.7	267	430	1.38	1.72	0.96	1.03	1.29	1.09
AO100		18.8				1.09	1.36	1.05	1.12	1.41	1.06
AO150		28.5				0.94	1.17	1.03	1.10	1.38	0.99
BO050,R		9.1	4.8	483	545	1.37	1.72	0.96	0.88	0.90	1.09
BO100,R		18.8				1.08	1.36	1.04	0.96	0.98	1.06
BO150,R		28.6				0.96	1.20	1.06	0.97	0.99	1.02
BO200,R		38.4				0.88	1.10	1.04	0.95	0.98	0.97
Kim & Yura (1999)					Mean	1.10	1.38	1.02	1.00	1.13	1.04
					COV	0.183	0.183	0.042	0.086	0.191	0.046
S355-2a	22	18.9	5.0	388	539	1.09	1.37	1.04	1.04	1.20	1.06
S355-2b		18.8				1.10	1.38	1.04	1.05	1.21	1.07
S355-3a		28.2	4.9			1.01	1.26	1.09	1.10	1.26	1.05
S355-3b		28.3				1.01	1.26	1.09	1.09	1.26	1.05
S355-4a		37.8				0.91	1.14	1.06	1.07	1.23	0.99
S355-4b		37.7				0.93	1.16	1.08	1.08	1.25	1.01
W700-1		9.6	4.8	830	871	1.31	1.64	0.91	0.81	0.80	1.04
W700-2		19.1				1.02	1.28	0.97	0.86	0.85	0.99
W700-3		28.9				0.86	1.08	0.94	0.83	0.82	0.91
W1100-1		9.8	5.2	1340	1440	1.25	1.56	0.88	0.79	0.79	1.00
W1100-2		18.4				1.02	1.27	0.96	0.86	0.86	0.98
W1100-3		28.4				0.87	1.09	0.95	0.85	0.85	0.92
Aalberg & Larsen (2001)					Mean	1.03	1.29	1.00	0.95	1.03	1.01
					COV	0.135	0.135	0.072	0.133	0.207	0.053
2	27	11.5	6.5	414	690	1.28	1.60	0.88	0.96	1.23	1.01
Rex & Easterling (2003) – One shear-out specimen						1.28	1.60	0.88	0.96	1.23	1.01

Table 2 Optimistic and pessimistic assumptions and their outcomes

Equation	**Optimistic Factor**	**Pessimistic Factor**	P_t/P_p
(1)	Shear coefficient = 0.75	Net shear planes	0.86 - 1.38
(2)	N/A	Net shear planes	1.08 – 1.72
(3)	Gross shear planes	Shear coefficient = 0.5	0.84 – 1.09
(4)	Gross shear planes	Partial strain hardening	0.79 – 1.12
(5)	Gross shear planes	No strain hardening	0.79 – 1.41
(6)	N/A	N/A	0.91 – 1.09
(9)	Mixed shear planes	Shear coefficient = 0.5	1.02 – 1.19

Table 3 Results for outlier specimens of Aalberg & Larsen (2001)

Specimen	d_h **(mm)**	e_n **(mm)**	t **(mm)**	F_y **(MPa)**	F_u **(MPa)**	P_t/P_p **of Equations**					
						(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
S355-1a	22	9.2	5.0	388	539	1.45	1.81	0.99	1.00	1.14	1.13
S355-1b		9.3				1.43	1.79	0.98	0.99	1.14	1.12
W700-4		37.5	4.8	830	871	0.80	1.00	0.93	0.82	0.81	0.87
W1100-4		37.7	5.2	1340	1440	0.80	1.00	0.93	0.83	0.83	0.87

Table 4 Results for double-row bolted connections (bolt pitch = 40 mm)

Specimen	d_h (mm)	e_n (mm)	t (mm)	F_y (MPa)	F_u (MPa)	P_t/P_p of Equations						
						(1)	(2)	(3)	(4)	(5)	(6)	(9)
AT0510,R	21	9.3	4.7	267	430	1.17	1.47	0.84	0.90	1.13	0.95	1.02
AT1510,R		27.8				1.03	1.29	0.92	0.99	1.24	0.97	1.07
BT0510,R		8.8	4.8	483	545	1.33	1.67	0.94	0.86	0.88	1.07	1.14
BT1510,R		27.8				1.08	1.35	0.97	0.89	0.91	1.01	1.12
Kim & Yura (1999)					Mean	1.16	1.44	0.92	0.91	1.04	1.00	1.09
					COV	0.114	0.114	0.062	0.060	0.165	0.053	0.052
S355--5a	22	9.4	5.0	388	539	1.29	1.61	0.88	0.89	1.02	1.01	1.08
S355--5b		9.5				1.33	1.66	0.91	0.92	1.06	1.04	1.11
S355--8a		28.3				1.06	1.32	0.93	0.93	1.07	0.97	1.08
S355--8b		28.3				1.07	1.34	0.94	0.95	1.09	0.99	1.09
Aalberg & Larsen (2002)					Mean	1.19	1.48	0.92	0.92	1.06	1.00	1.09
					COV	0.119	0.119	0.025	0.025	0.025	0.029	0.015
121.4	18	7.0	12.0	278	443	1.25	1.56	0.97	1.03	1.28	1.06	1.15
122.4		14.0				1.18	1.47	1.01	1.07	1.34	1.07	1.18
123.4		22.2				1.07	1.33	0.99	1.06	1.32	1.02	1.14
124.4		30.7				0.98	1.23	0.98	1.04	1.29	0.98	1.10
121.6		6.7		477	601	1.26	1.58	0.97	0.94	1.02	1.07	1.16
122.6		16.0		473	604	1.17	1.46	1.03	1.00	1.09	1.08	1.19
123.6		23.0		493	615	1.05	1.31	0.98	0.95	1.02	1.01	1.12
124.6		31.6		478	600	1.01	1.26	1.01	0.97	1.05	1.01	1.13
121.8		6.0		622	742	1.20	1.50	0.92	0.86	0.91	1.01	1.10
122.8		15.0				1.08	1.35	0.93	0.88	0.93	0.99	1.09
123.8		23.0		674	775	0.98	1.23	0.92	0.86	0.88	0.95	1.05
124.8		31.0				0.92	1.15	0.92	0.85	0.88	0.92	1.03
A121	18	31.0		333	480	0.93	1.16	0.92	0.94	1.10	0.92	1.04
[a]A123	22	39.0		339	483	0.98	1.23	0.99	1.00	1.17	0.98	1.11
Udagawa & Yamada (1998, 2004)					Mean	1.07	1.34	0.97	0.96	1.09	1.00	1.11
					COV	0.108	0.108	0.039	0.080	0.152	0.053	0.045

a: Bolt pitch = 50 mm