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Shear lag and eccentricity effects of bolted connections in cold-formed steel sections

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Abstract

This paper examines the "three factors" approach previously presented by the senior author for determining the net section efficiency of a bolted cold-formed steel open profile. One objective is to ascertain that the net section efficiency is governed by three factors: the in-plane shear lag associated with stress concentration around a bolt hole that is also present in a flat sheet, the out-of-plane shear lag that is also present in a bi-symmetric I-section bolted at the flanges only, and the bending moment arising from the connection eccentricity with respect to the neutral axis. This paper presents the test results of 55 single and back-to-back channel braces bolted at the web including those connected with one row of bolts perpendicular to the axial load. The test results affirm the three factors approach, and it was found that the back-to-back channel braces were affected by local bending even though the connection eccentricity was nominally zero. The paper asserts the need to avoid snug-tightening laboratory test specimens and the importance of identifying the failure modes accurately.

Keywords

formed, cold, connections, bolted, effects, eccentricity, lag, sections, shear, steel

Disciplines

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Shear Lag and Eccentricity Effects of Bolted Connections in Cold-Formed Steel Sections

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1. Introduction

The net section tension capacity of a bolted steel profile such as a channel or an angle cannot be computed simply as the product of its net section area and its material tensile strength, which would otherwise imply a full net section efficiency. In reality, the net section efficiency is invariably less than unity due to a number of factors [1-5]. In order to account for the reduced net section efficiency of bolted steel profiles, constant reduction factors [6] and simple formulae [7- 8] have been incorporated into the code equations for determining the net section tension capacity. Regression analysis of laboratory test results is also popular in the literature [3-4, 9-13].

The equations specified in the design standards [6-8], mainly based on the work of Munse & Chesson [2], have been found to be unconservative for most specimens [3-5, 10-11, 14-15]. Equations derived using regression analysis, on the other hand, have been shown by Teh & Gilbert [5, 16] to have pitfalls if not handled properly.

Recently, Teh & Gilbert [5] proposed a design equation for determining the net section tension capacity of a channel brace bolted at the web. The equation incorporates the in-plane

shear lag associated with stress concentration around a bolt hole that is also present in a flat sheet, the out-of-plane shear lag that is also present in a bi-symmetric I-section bolted at the flanges only, and the bending moment arising from the connection eccentricity with respect to the neutral axis. It was shown through laboratory tests that the equation is significantly more accurate and reliable than the code equations and those derived in the literature using regression analysis.

The equation proposed by Teh & Gilbert [5] for a channel brace bolted at the web was modified by Teh & Gilbert [14] to suit an angle brace bolted at one leg. While the modified equation was shown to be accurate except for unequal angles bolted at the narrow leg, it was noted that a single angle brace bolted at one leg is subject to biaxial bending under the axial load. Furthermore, the net section efficiency of double angles, which were subject to bending in the symmetry plane only, was not found to be higher than that of single angles.

It is significant that the term $\bar{x}/L$ contained in the efficiency factor expression, considered by Teh & Gilbert [5, 14] to represent the interaction between the detrimental bending moment due to connection eccentricity $\bar{x}$ and the counteracting moment provided by the bolt couple acting at L distance apart, was considered by Munse & Chesson [2] to account for the out-of-plane shear lag. In the formulation of Teh & Gilbert [5, 14], the latter effect is a function of the ratio of the unconnected element width to the total element width only.

The AISC specification for structural steel buildings [7] determines the net section efficiency factor of a bolted profile to be the larger of the two values computed as a function of $\bar{x}/L$ and as the ratio of the connected element width to the total element width. Teh & Gilbert [5, 14], on the other hand, treats the two factors as cumulative. The AISC approach [7] means that, in theory, two sections having the same ratio may be found to have the same net section efficiency even if their connection eccentricities differ from each other.

The present work aims to ascertain that the net section efficiency of a bolted cold-formed steel open profile is governed by the three factors described by Teh & Gilbert [5]. The equation proposed by Teh & Gilbert [5] is verified against the laboratory test results of double channel braces bolted symmetrically back-to-back, for which the connection eccentricity $\bar{x}$ is nominally zero.

This paper also presents the test results of single channels connected at the web with a single line or row of bolts parallel or perpendicular to the axial load, complementing the tests of Teh & Gilbert [5] on single channels bolted in a rectangular pattern. In addition, the present aspect ratios are as high as 0.6. An aspect ratio is the ratio of the flange width to the web depth. The proposed equation was also verified against the test results of Pan [10] involving an aspect ratio as high as 0.75.

The paper includes some discussions on the needs to ensure that laboratory test specimens are not snug-tightened, and to accurately identify the actual failure mode of bolted connection specimens.

2. Equations for the net section tension capacity of a channel brace

Clause 3.2.2(3) of AS/NZS 4600:2005 Cold-formed Steel Structures [6] specifies the net section tension capacity of a bolted connection in a steel member to be

$$P_p = 0.85k_t A_n F_u \quad (1)$$

in which A_n is the net area of the section and F_u is the material tensile strength of the member. The variable k_t in the equation represents the net section efficiency factor, which is equal to unity for a connection that ensures uniform stress distribution over the net section.

The clause is adopted from AS 4100-1998 Steel Structures [17]. The explicit coefficient of 0.85 embedded into Equation (1) “is intended to account for sudden failure by local brittle behaviour at the net section” [18] and is therefore in a sense part of the resistance factor. The reason for the sudden brittle failure not being accounted for using a lower (formal) resistance factor is that a uniform resistance factor of 0.90 is applied to the net section fracture mode and the member yielding (over the gross section) mode under axial tension. The effective resistance factor actually applied to the net section fracture mode is therefore 0.765.

For the purpose of the present work, the explicit coefficient of 0.85 in Equation (1) is ignored since it is actually part of a safety factor rather than a net section efficiency factor. In accordance with Table 3.2 of the Australasian code [6], which specifies the values of k_t for various connection arrangements, Equation (1) is replaced by

$$P_p = A_n F_u \quad (2)$$

for a double channel brace symmetrically bolted back-to-back ($k_t = 1.0$), and

$$P_p = 0.85 A_n F_u \quad (3)$$

for a single channel brace bolted at the web ($k_t = 0.85$).

Section E5.2 of Supplement No. 2 to the North American Specification for the Design of Cold-formed Steel Structural Members 2007 [8] specifies the net section tension capacity of a channel brace bolted at the web to be

$$P_p = A_n F_u \max \left\{ 0.5, \min \left(0.9, 1 - 0.36 \frac{\bar{x}}{L} \right) \right\} \quad (4)$$

in which $\bar{x}$ is the distance between the web’s outer face and the section’s neutral axis (i.e. the connection eccentricity), and L is the connection length. These variables are defined in Figure

1. It will be seen that, for most practical channel connections, Equation (4) gives a net section efficiency factor equal to 0.9, which is over-optimistic for most channel sections.

The function “max” in Equation (4) means that the larger between the two values inside the outer brackets is to be used, while the function “min” means that the lesser between the two values inside the inner brackets is to be used.

Equation (4) was proposed by LaBoube & Yu [19] based on the laboratory test results of Holcomb et al. [12] and the original equation proposed by Munse & Chesson [2] to account for “shear lag” in a steel member where not all of its cross-sectional elements are bolted to the joining member. The original equation is still used in the current AISC specification [7] with a lower bound “shear lag factor” equal to the ratio of the connected width to the total width

$$P_p = A_n F_u \max \left(1 - \bar{x}/L, \frac{W_c}{W_c + 2W_f} \right) \quad (5)$$

in which W_c is the web depth and W_f is the flange width as defined in Figure 1. In practically all cases, the lower bound does not affect the outcome of Equation (5).

Teh & Gilbert [5] proposed the following equation for determining the net section tension capacity of a single channel brace bolted at the web

$$P_p = A_n F_u \left(\frac{1}{1.1 + \frac{W_f}{W_c + 2W_f} + \frac{\bar{x}}{L}} \right) \quad (6)$$

As explained by Teh & Gilbert [5], the constant of 1.1 in the denominator of Equation (6) accounts for the in-plane shear lag effect present in the steel sheet [16], the term $W_f/(W_c +$

2W_f) accounts for the out-of-plane shear lag effect of a channel brace bolted at the web, and the term $\bar{x}/L$ accounts for the detrimental bending moment effect due to the connection eccentricity $\bar{x}$ and for the counteracting bending moment effect that increases with the connection length L . It can be seen from the statistical analysis of test results found in the literature conducted by Pan [10] that the out-of-plane shear lag and the eccentricity terms are independent variables.

Since the term $\bar{x}/L$ was intended by Munse & Chesson [2] to account for the out-of-plane shear lag effect, for a double channel brace bolted symmetrically back-to-back, this term vanishes only in the approach of Teh & Gilbert [5]. Equation (6) becomes, for such a brace not subject to bending under the axial load

$$P_p = A_n F_u \left(\frac{1}{1.1 + \frac{W_f}{W_c + 2W_f}} \right) \quad (7)$$

3. Test materials

The G450 sheet steel materials used in the laboratory tests, which have a trade name GALVASPAN[®], were manufactured and supplied by Bluescope Steel Port Kembla Steelworks, Australia. Two nominal thicknesses were used in the present work, being 1.5 mm and 3.0 mm. The average base metal thicknesses t_{base} , yield stresses F_y , tensile strengths F_u and elongations at fracture over 15 mm, 25 mm and 50 mm gauge lengths ε_{15} , ε_{25} and ε_{50} , and uniform elongation outside the fracture ε_{uo} of the steel materials as obtained from six 12.5 mm wide tension coupons are shown in Table 1. Tensile loadings of all coupons and bolted

connection specimens are in the direction perpendicular to the rolling direction of the G450 sheet steel. The tension coupon tests were conducted at a constant stroke rate of 1 mm/minute resulting in a strain rate of about 2×10^{-4} per second prior to necking.

The tensile strengths in the direction perpendicular to the rolling direction of 1.5 mm and 3.0 mm G450 sheet steels obtained in the present work, rounded to the nearest 5 MPa, are 6% and 10% higher than those obtained by Teh & Hancock [20] in the rolling direction. While Teh & Hancock [20] did not provide the elongations at fracture, it is believed that the rolling direction is associated with higher ductility. In any case, the G450 sheet steels used in the present work represent the grades of steel covered by AS/NZS 4600 [6] which are among those having the lowest ductility and for which the nominal tensile strength and yield stress may be fully utilised in structural design calculations [21]. The use of such low ductility steel ensures that the proposed design equation is not unsafe for more ductile steels.

The sheet steels were brake-pressed into channel sections, with the 1.5-mm sections having a corner radius of 2 mm and the 3.0-mm ones having a corner radius of 3 mm.

4. Specimen configurations and test arrangements

The back-to-back double channel specimens comprise sections having web depths of 80, 100 and 120 mm, with flange widths ranging from 20 to 50 mm, corresponding to the dimensions of the single channel specimens tested by Teh & Gilbert [5]. Such an arrangement enables the investigation of the significance of the term $\bar{x}/L$ found in Equation (6).

As with the single specimens tested by Teh & Gilbert [5], the back-to-back channel specimens had two rows of bolts arranged in a rectangular pattern, as depicted in Figure 1. However, the present work includes single channel specimens having a single line of bolts in

the direction of loading, as shown in Figure 2, to complement the single specimens of Teh & Gilbert [5]. The highest aspect ratio of the present single specimens is 0.6, composed of a channel section having a web depth of 50 mm and a flange width of 30 mm.

The bolts at the downstream ends (i.e. those closest to the member ends) were tightened as snug as possible with a wrench to prevent “global bending” of the back-to-back specimens, associated with the separation of the webs from the gusset plates. However, in order to ensure that friction did not contribute to the tension capacity, the bolts at the upstream end were only lightly tightened. As illustrated in Figure 3, only friction of the bolts at the upstream end would contribute to the tension capacity of the critical net section since the resultant of stresses at the critical section A-A resisting the tension load P does not include the friction of the downstream bolts. As will be discussed later, friction between the gusset plates (or the washer) and the bolted specimen is an important factor that has often been overlooked in the literature.

As demonstrated by Teh & Gilbert [5], channel braces bolted at the web that have a single row of bolts only perpendicular to the axial load tend to fail in either block shear or bearing, even for a channel section with an aspect ratio of 0.2. In order to obtain net section fracture, the aspect ratio has to be as low as 0.1, resulting in minimal eccentricity $\bar{x}$ as seen later. The possibility of applying Equation (7) to such connections was investigated.

The bolted connection specimens were tested to failure using an Instron 8033 universal testing machine at a stroke rate of 1 mm/minute. The test set-up is shown in Figure 4.

5. Experimental test results and discussions

In calculating the net section tension capacity P_p of a specimen, the measured values of the material properties and geometric dimensions such as the base metal thickness, the web depth, the flange width, the bolt hole diameter and the connection length, are used. However, for legibility, only the nominal values are shown in the tables following.

Only the code equations [6-8] and the equations proposed by the authors are discussed in this section. Equations proposed in the literature for determining the net section tension capacity of a channel brace [10, 12] have been previously discussed by Teh & Gilbert [5].

5.1. Double channel sections bolted back-to-back

Table 2 lists the relevant geometric dimensions and the test results of the back-to-back double channel specimens. An empty cell in the table indicates that the data in the above cell applies. The variable c denotes the actual net section efficiency factor, defined as the ratio of ultimate test load P_t to net section tension capacity P_p computed with the assumption of uniform stress distribution

$$c = \frac{P_t}{A_n F_u} \quad (8)$$

All specimens failed in net section fracture, as shown in Figure 5 for CB7.

Table 2 shows the ratios of the ultimate test load P_t to the net section tension capacity P_p predicted by Equations (2) and (4) through (7). In applying Equations (4) through (6), the connection eccentricity $\bar{x}$ of the individual channel has been used.

It can be seen from the actual net section efficiency factors c in Table 2 that the assumption of uniform stress distribution in Equation (2) is unjustified. On the other hand, despite the use of the individual channel's eccentricity $\bar{x}$ to account for the out-of-plane shear lag effect, Equations (4) and (5) still lead to overestimations. In particular, Equation (4) suggests a net section efficiency factor equal to the upper bound value of 0.9 for all specimens except for CB10 since the term " $1 - 0.36 \bar{x}/L$ " is greater than 0.9 for these specimens.

Equation (7) results in an average professional factor equal to 0.98, with a standard variation of 0.084. Table 2 shows that Equation (7) fails to account for the effects of connection eccentricity and connection length. Although the eccentricity $\bar{x}$ is nominally zero, the back-to-back double channel specimens were subjected to significant local bending, as shown in Figure 6(a). There were therefore detrimental local bending effects, as well as counteracting bending effects from the bolt couples acting at L distance apart.

Equation (6) leads to the most reasonable if conservative estimates for the back-to-back double channel specimens. The use of the individual channel's eccentricity $\bar{x}$ (more than) captures the local bending phenomena shown in Figure 6(a), which can be compared to the global bending of the single channel specimen shown in Figure 6(b).

As indicated in Table 2, specimen CB12 had three rows of bolts in order to prevent bolt shear failure. Based on the rationale of Teh & Gilbert [5], who derived Equation (6), the number of bolt rows is irrelevant to the net section efficiency. This rationale is supported by the laboratory test results of Salih et al. [13] on angle braces bolted at one leg.

It is also evident from the results of Equation (7), which does not account for local or global bending, that the St Venant's effect of the downstream bolts on the stress distribution at the critical net section was significant. For 100-mm long connections, the St Venant's effect more

than offset the absence of connection eccentricity $\bar{x}$ in the equation such that the equation was found to be conservative for most of these specimens.

5.2. Single channels connected at the web with a single line of bolts

Table 3 lists the relevant geometric dimensions and the test results of the single channel specimens with a single line of bolts in the axial direction. An empty cell in the table indicates that the data in the above cell applies. The table shows the ratios of the ultimate test load P_t to the net section tension capacity P_p predicted by Equations (3) through (6). All the specimens failed in the net section fracture mode, as shown in Figure 2 for 1.5 and 3.0 mm specimens.

Equations (3) through (5) were found to be over-optimistic for the present single channel specimens with a single line of bolts, affirming the conclusion of Teh & Gilbert [5], who tested single channel braces with the bolting pattern depicted in Figure 1.

It can be seen from Table 3 that Equation (6) is reasonably accurate for the present single channel specimens with a single line of bolts, although the resulting coefficient of variation is significantly greater than that for the single channel specimens with a rectangular bolting pattern tested by Teh & Gilbert [5]. The latter is reproduced in Table 4.

5.3. Single channels connected at the web with a single row of bolts

Table 5 lists the geometric dimensions and the test results of the single channel specimens with a single row of bolts perpendicular to the axial load. An empty cell in the table indicates that the data in the above cell applies. The aspect ratio of these specimens, 0.1, is extremely low in order to obtain the net section fracture mode, as shown in Figure 7(a). In fact, there is minimal eccentricity $\bar{x}$.

As it transpired, the in-plane and out-of-plane shear lag terms in Equation (7) are sufficient to accurately determine the net section efficiency factors of the tested specimens.

5.4. Frictional forces between gusset plates and bolted specimen

An instructive set of test results were provided by Yip & Cheng [22], who tested single channel braces connected at the web with a single line of bolts in the axial direction, similar to the CSS specimens shown in Figure 2. Five of their specimens failed in net section fracture. The test net section efficiency factors of all these specimens were found to be higher than unity, with a median of 1.14.

Such test results are “anomalous” since the net section efficiency factor of a bolted channel brace cannot be greater than unity. The strain measurement results indicate compression stresses in the flanges [22], meaning the net section efficiency must be low.

Yip & Cheng [22] found that the ultimate test loads of the five specimens were significantly higher than their finite element predictions, with a maximum over-strength of more than 30%. They suggested that the discrepancies were due to the neglect of frictional forces between the gusset plates and the bolted specimens in their finite element models. In this regard, good agreements between laboratory test results and FEA predictions were obtained by Salih et al. [13], who modelled the friction between contact surfaces of their bolted connections.

After pre-loading a specimen so that the bolts bore against the gusset plates and the specimen, Yip & Cheng [22] snug-tightened the bolts. This procedure means that the frictional forces between the gusset plates and the bolted specimen contributed to the apparent net section tension capacity (even though the load reading was returned to zero following the pre-load).

Since the first paper in the series on the subject was written by the senior author [16], a point is made that the bolts were not tightened to the extent that the frictional forces contributed significantly to the net section tension or block shear capacity, as also made in the section “Specimen configurations and test arrangements”. Rogers & Hancock [23] tightened the bolts by hand to a torque less than 10 Nm to ensure that the connection was able to slip under minimal loading. However, in the literature of cold-formed steel bolted connections, a torque of at least 100 Nm has been applied [11].

The provision in steel design specifications that bolts must be installed to a snug tight level should not be a cause to ignore potentially significant frictional forces in an experiment. It is prudent to prevent frictional forces from contributing to the net section tension or block shear capacity of a test specimen, if only to avoid anomalous test results and incorrect conclusions. As an aside, Yip & Cheng [22] and Chung & Ip [24, 25] have found that the friction between the interfaces of a bolted connection contributes significantly to the bearing resistance too.

5.5. Net section fracture or block shear failure?

Pan [10] tested single channel braces bolted at the web with the rectangular pattern depicted in Figure 1. Some of the specimens had the same web and flange dimensions as the CH specimens listed in Table 4. The SSC400 sheet steel used by Pan [10] was however significantly more ductile than the G450 sheet steel used by Teh & Gilbert [5].

Table 6 lists the relevant geometric dimensions and the results of Group A specimens tested by Pan [10]. An empty cell in the table indicates that the data in the above cell applies. The variable W_T is the total nominal sheet width, equal to $W_c + 2 W_f$. The measured tensile strength of the material is 450 MPa (rounded to the nearest 5 MPa from the reported 447.77 MPa). The base metal thickness was assumed to be 2.4 mm in the calculations since it was

not reported. The nominal bolt hole diameter of 14.3 mm was also used in the calculations. In determining the connection eccentricity $\bar{x}$, a corner radius of 2.5 mm was assumed in the computer program ColdSteel [26]. Each of the test results in Table 6 is the average of three specimens having the same nominal configuration, except for the last entry in which case it is the average of two 120 mm by 40 mm specimens only.

Equation (6) results in reasonably accurate estimates for the specimens having a web depth W_c of 80 mm, with aspect ratios ranging from 0.5 to 0.75. However, the results of the deeper specimens are not so encouraging. A close examination of the test results of the 100-mm and 120-mm deep specimens revealed that they were likely to have been in error.

The test results would indicate that the net section efficiency factors c decreased with decreasing aspect ratios for a constant flange width W_f of 40 mm, as shown in Table 6. However, the reverse should be true provided the failure modes were all net section fracture. As demonstrated by Teh & Gilbert [5], for a constant flange width W_f of 40 mm, the net section efficiency should increase with increasing web depths from 80 mm to 120 mm as the aspect ratios decrease. The reason is that the out-of-plane shear lag effect and the connection eccentricity $\bar{x}$ decrease over this variation.

The eagle-eyed reader may also notice the incidental “symmetry” of the test net section efficiency factors c in Table 6 about the middle specimen, which would imply that the channel braces having the same total width W_T had the same net section tension capacity irrespective of their aspect ratios (ranging from 0.33 to 0.75). Furthermore, the test net section efficiency factors in Table 6 could be approximated as k/W_T , with the constant k equal to 106 (mm). Such a direct inverse relationship is highly unlikely as it does not account for the effects of out-of-plane shear lag and connection eccentricity. It is even unlikely for bolted connections in flat sheets [16].

One possible explanation for the “anomaly” of the test results of the 100-mm and 120-mm deep specimens is that a failure mode other than net section fracture was involved. Sometimes a block shear failure, an example of which is shown in Figure 7(b), could be mistaken for a net section fracture mode, an example of which is shown in Figure 7(a). The reason why the specimen in Figure 7(b) is considered to fail in block shear can be found in Teh & Clements [27], while the reason why it was not bearing can be found in Clements & Teh [28]. The likelihood of block shear failure increases with increasing web depths, especially if the bolt spacing in the transverse direction to loading does not increase commensurately.

Misidentifications of failure modes in the literature of cold-formed steel bolted connections have been documented by Rogers & Hancock [29], who also described the methodology for identifying various failure modes (other than block shear failure).

5.6. Resistance factor (or capacity reduction factor)

For the sake of simplicity, it is intended that a uniform resistance factor is applied to Equation (6) for single and double channel braces connected at the web (whether symmetrically or not). However, in order to prevent the results of the double channel specimens bolted symmetrically back-to-back from skewing the resistance factor higher, these specimens were not included in the determination of the resistance factor. The overall average ratio of the ultimate test load P_t to the net section tension capacity P_p predicted by Equation (6) for the forty one CSS and CH specimens listed in Tables 3 and 4 is 1.01, with a standard deviation of 0.064.

Section F1.1 of the North American specification [30] specifies that the resistance factor ϕ of a design equation is determined as follows

$$\phi = C_{\phi} (M_m F_m P_m) e^p \quad (9)$$

in which C_{ϕ} is the calibration coefficient equal to 1.52 in the case of the Load and Resistance Factor Design (LRFD), M_m is the mean value of the material factor equal to 1.10 according to Table F1 of the North American specification [30], F_m is the mean value of the fabrication factor equal to 1.00, and P_m is the mean value of the professional factor equal to 1.01 as stated in the preceding paragraph.

The power p of the natural logarithmic base e in Equation (9) is

$$p = -\beta_0 \sqrt{V_M^2 + V_F^2 + C_p V_P^2 + V_Q^2} \quad (10)$$

in which V_M is the coefficient of variation of the material factor equal to 0.08, V_F is the coefficient of variation of the fabrication factor equal to 0.05, V_P is the coefficient of variation of the professional factor equal to 0.065, C_p is the correction factor equal to 1.08, and V_Q is the coefficient of variation of load effects equal to 0.21. All these values are determined in accordance with Section F1.1 of the North American specification [30].

It was found that in order to achieve the target reliability index β_0 of 3.5 in the LRFD, Equation (9) yields a resistance factor of 0.73.

A resistance factor ϕ equal to 0.70 (rounded down to the nearest 0.05) in conjunction with Equation (6) is therefore recommended for the LRFD approach for determining the net section tension capacity of a cold-formed steel channel brace bolted at the web only, whether single or double (symmetrically or un-symmetrically connected back-to-back). This value would be the same as that found by Teh & Gilbert [5] for the CH specimens in Table 4 if the statistical variables recommended in Table F1.1 of the North American specification [30] are used in the calculation, and is higher than the current value of 0.65 used in the specification.

Only two channel brace specimens with a single row of bolts perpendicular to the axial load were tested to net section fracture, and no reliability analysis has been used to determine the resistance factor to be applied to Equation (7). However, considering that only channel braces with extremely low aspect ratios will fail in net section fracture when connected with a single row of bolts, it appears from the results shown in Table 5 that it is reasonable to apply the same capacity factor of 0.7 to Equation (7) for such cases.

6. Conclusions

Laboratory test results of fifty five channel braces bolted at the web which failed in net section fracture have been presented in this paper. The single channel specimens were bolted with four bolts in a rectangular pattern, a single line of bolts in the axial direction, or a single row of bolts perpendicular to the axial load. The results affirm the design equations in which the net section efficiency of a bolted cold-formed steel open profile is reduced by three distinct factors: the in-plane shear lag associated with stress concentration around a bolt hole that is also present in flat sheets, the out-of-plane shear lag that is also present in a bi-symmetric I-section bolted at the flanges only, and the bending moment arising from the connection eccentricity with respect to the neutral axis.

Even though the connection eccentricity of a double channel brace bolted symmetrically back-to-back is zero, local bending can reduce the net section efficiency significantly. It is proposed that the same design equation is applied to single and double channel braces bolted at the web so that the three factors are always accounted for.

A slightly modified equation, in which the bending effect is neglected, can be applied to channel braces having a single row of bolts perpendicular to the axial load. If the aspect ratio

is 0.1 or lower, then the net section fracture mode may govern the strength limit state. Otherwise, the net section fracture mode is irrelevant to the channel brace.

One important aspect that has often been overlooked in the literature is the contribution of frictional forces between the gusset plates and the bolted specimen to the apparent net section tension or block shear capacity. Snug-tightening of bolts, while mandated in the construction field, should not be used in experimental tests unless the contribution of the frictional forces is being researched or otherwise accounted for. Neglect of this aspect has led to anomalous results that significantly overstate the true capacities.

Provided that net section fracture is the governing failure mode, the net section efficiency of a channel brace increases with decreasing aspect ratios for a given flange width or a given web depth. Test results to the contrary may indicate a failure mode other than net section fracture.

It is recommended that a resistance factor of 0.70 be applied to the two equations proposed in this paper in order to ensure a reliability index of not less than 3.5 in the LRFD approach of the North American specification for the design of cold-formed steel structures.

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461 **Notation**

462 A_n = net area of considered section

463 c = test net section efficiency factor

464 C_p = correction factor

465 C_ϕ = calibration coefficient

466 d = bolt diameter

467 F_m = mean value of fabrication factor

468 F_u = tensile strength of steel material

469 F_y = yield stress of steel material

470 k_t = net section efficiency factor according to AS/NZS 4600:2005

471 L = connection length

472 M_m = mean value of material factor

473 P_m = mean value of professional factor

474 P_p = predicted failure load

475 t = nominal sheet thickness

476 t_{base} = base metal thickness

477 V_F = coefficient of variation of fabrication factor

478 V_M = coefficient of variation of material factor

479 V_P = coefficient of variation of professional factor

480 V_Q = coefficient of variation of load effects

481 W_c = web depth

482 W_f = flange width

483 $\bar{x}$ = connection eccentricity

- 484 β_0 = target reliability index
- 485 ε_{15} = elongation at fracture over a gauge length of 15 mm
- 486 ε_{25} = elongation at fracture over a gauge length of 25 mm
- 487 ε_{50} = elongation at fracture over a gauge length of 50 mm
- 488 ε_{uo} = uniform elongation outside fracture zone
- 489 ϕ = resistance factor (or capacity reduction factor)

Figure 1 Geometric dimensions of a channel member bolted at the web

(a) 1.5 mm CSS specimen

(b) 3.0 mm CSS specimen

Figure 2 Specimens with a single line of bolts in the axial direction

Figure 3 Contribution of bolt friction to tension capacity

Figure 4 Tension test arrangement (back-to-back specimen)

Figure 5 Net section fracture of one half of a back-to-back channel specimen

(a) Local bending

(b) Global bending

Figure 6 Local (CB12) and global bending of channel braces

(a) Net section fracture

(b) Block shear failure

Figure 7 Net section fracture and block shear failure

Table 1 Average material properties

	t_{base} (mm)	F_y (MPa)	F_u (MPa)	F_u / F_y	ϵ_{15} (%)	ϵ_{25} (%)	ϵ_{50} (%)	ϵ_{u0} (%)
1.5 mm	1.48	605	630	1.04	21.3	18.0	12.0	6.8
3.0 mm	2.95	530	580	1.09	29.3	22.0	15.3	8.1

Table 2 Results of double channel bolted back-to-back specimens ($t = 3.0$ mm)

Spec	d_h (mm)	W_c (mm)	W_f (mm)	$\bar{x}$ (mm)	L (mm)	A_n (mm ²)	c	P_t/P_p				
								(2)	(4)	(5)	(6)	(7)
CB1	17	100	30	6.90	50	779	0.74	0.74	0.83	0.86	1.06	0.96
CB2					75	791	0.85	0.85	0.94	0.93	1.17	1.09
CB3					100	774	0.89	0.89	0.99	0.95	1.20	1.14
CB4			40	10.2	50	898	0.69	0.69	0.76	0.86	1.05	0.91
CB5					75	893	0.76	0.76	0.85	0.88	1.11	1.00
CB6					100	904	0.82	0.82	0.91	0.91	1.16	1.08
CB7			50	13.9	50	1017	0.65	0.65	0.72	0.90	1.05	0.87
CB8					75	1024	0.68	0.68	0.76	0.84	1.05	0.92
CB9					100	1011	0.73	0.73	0.81	0.85	1.09	0.98
CB10	13	80	40	11.4	40	832	0.65	0.65	0.73	0.91	1.06	0.88
CB11					60	827	0.69	0.69	0.77	0.85	1.06	0.93
CB12*					80	831	0.77	0.77	0.85	0.89	1.14	1.03
CB13	17	120		9.30	50	1008	0.70	0.70	0.78	0.87	1.04	0.91
CB14					75	1017	0.72	0.72	0.80	0.82	1.03	0.94
CB15					100	1016	0.79	0.79	0.87	0.87	1.09	1.02
							Mean	0.74	0.82	0.88	1.09	0.98
							COV	0.095	0.094	0.041	0.049	0.084

*Three rows of bolts were used to prevent bolt failure.

Table 3 Results of single channel specimens with a single axial line of bolts ($d_h = 17$ mm)

Spec	W_c (mm)	W_f (mm)	t (mm)	$\bar{x}$ (mm)	L (mm)	A_n (mm ²)	c	P_t/P_p			
								(3)	(4)	(5)	(6)
CSS1	50	20	1.5	5.13	50	112	0.67	0.79	0.74	0.74	0.95
CSS2			3.0	5.76		232	0.74	0.87	0.82	0.84	1.06
CSS3			1.5	5.13	75	112	0.78	0.91	0.86	0.83	1.08
CSS4			3.0	5.76		228	0.85	1.00	0.95	0.92	1.18
CSS5			1.5	5.13	100	111	0.79	0.92	0.87	0.83	1.07
CSS6			3.0	5.76		233	0.87	1.02	0.96	0.92	1.19
CSS7		30	1.5	8.92	50	141	0.63	0.74	0.70	0.76	0.96
CSS8			3.0	9.60		291	0.61	0.72	0.68	0.76	0.95
CSS9			1.5	8.92	75	142	0.67	0.79	0.75	0.76	1.00
CSS10			3.0	9.60		291	0.69	0.82	0.77	0.79	1.02
CSS11			1.5	8.92	100	144	0.65	0.76	0.72	0.71	0.95
CSS12			3.0	9.60		292	0.75	0.88	0.83	0.82	1.08
							Mean	0.85	0.80	0.81	1.04
							COV	0.117	0.117	0.082	0.081

Table 4 Results of single channel specimens with the rectangular bolting pattern [5]

Spec	d_h (mm)	W_c (mm)	W_f (mm)	t (mm)	$\bar{x}$ (mm)	L (mm)	A_n (mm ²)	c	P_t/P_p			
									(3)	(4)	(5)	(6)
CH1	17	100	30	1.5	6.28	50	190	0.67	0.79	0.75	0.77	0.95
CH2				3.0	6.90		381	0.73	0.86	0.81	0.85	1.04
CH3				1.5	6.28	75	185	0.72	0.85	0.80	0.79	0.99
CH4				3.0	6.90		386	0.78	0.91	0.86	0.85	1.07
CH5				1.5	6.28	100	188	0.74	0.88	0.83	0.79	1.00
CH6				3.0	6.90		389	0.79	0.93	0.88	0.85	1.07
CH7			40	1.5	9.56	50	220	0.61	0.72	0.68	0.76	0.93
CH8				3.0	10.2		449	0.64	0.76	0.71	0.81	0.98
CH8_2							449	0.66	0.78	0.74	0.83	1.01
CH10						75	450	0.69	0.81	0.76	0.79	1.00
CH10_2							449	0.66	0.77	0.73	0.76	0.96
CH11				1.5	9.56	100	217	0.67	0.79	0.75	0.75	0.95
CH12				3.0	10.2		450	0.70	0.83	0.78	0.78	1.00
CH12_2							451	0.71	0.83	0.79	0.79	1.01
CH13			50	1.5	13.2	50	248	0.59	0.69	0.65	0.79	0.95
CH14				3.0	13.9		502	0.61	0.71	0.67	0.83	0.98
CH15				1.5	13.2	75	248	0.62	0.72	0.68	0.75	0.94
CH16				3.0	13.9		507	0.64	0.76	0.71	0.78	0.98
CH17				1.5	13.2	100	246	0.63	0.74	0.70	0.73	0.94
CH18				3.0	13.9		501	0.64	0.76	0.71	0.75	0.96
CH19	13	80	40		11.4	40	408	0.62	0.73	0.69	0.87	1.02
CH20						60	392	0.65	0.76	0.72	0.80	0.99
CH21						80	407	0.66	0.77	0.73	0.77	0.98
CH22			20		4.60	40	290	0.76	0.89	0.84	0.86	1.05
CH23						60	287	0.82	0.96	0.91	0.89	1.10
CH24						80	294	0.86	1.01	0.95	0.91	1.13
CH25	17	120	40		9.30	50	506	0.71	0.84	0.79	0.87	1.06
CH26						75	500	0.73	0.86	0.81	0.83	1.04
CH27						100	502	0.75	0.88	0.83	0.82	1.04
Mean									0.81	0.77	0.81	1.00
COV									0.097	0.097	0.059	0.051

Table 5 Results of single channel specimens with a single row of bolts ($d_h = 17$ mm)

Spec	W_c	W_f	t	$\bar{x}$	A_n	c	P_t/P_p	
	(mm)	(mm)	(mm)	(mm)	(mm ²)		(3)	(7)
CSP1	100	10	1.5	1.50	125	0.839	0.988	0.988
CSP2			3.0	2.17	259	0.838	0.986	0.990
						Mean	0.987	0.989
						COV	0.002	0.002

Table 6 Results of Group A specimens tested by Pan [10] ($t = 2.4$ mm, $d_h = 14.3$ mm, $L = 40$ mm)

W_c (mm)	W_f (mm)	W_T (mm)	$\bar{x}$ (mm)	c	P_t/P_p			
					(3)	(4)	(5)	(6)
80	60	200	19.2	0.53	0.63	0.65	1.03	1.00
	50	180	15.0	0.58	0.68	0.66	0.86	0.98
	40	160	11.1	0.65	0.76	0.72	0.90	1.05
100		180	9.96	0.58	0.69	0.65	0.78	0.92
120		200	9.05	0.53	0.62	0.59	0.68	0.81