Performance of the Unified Block Shear Equation for Common Types of Welded Steel Connections

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ABSTRACT

Block shear describes a steel connection failure mode in which a combination of tensile and shear failures along perpendicular planes results in a block of material being displaced from a member. This behavior has been observed to govern the design of many bolted connections and has been well researched, resulting in design equations of various forms. Certain arrangements of welded connections are also susceptible to block shear. This paper reports on the potential application of a "unified" block shear equation, proposed by Driver et al. (2006) for bolted connections, to various welded connections. Experimental results for welded steel lap plate connections loaded concentrically in tension are examined. The results of these tests are used to describe the connection behavior and to evaluate the performance of several design equations, including the proposed "unified" equation. The application of the unified equation to welded connections of slotted hollow structural sections (HSS) and to coped beams supported at their ends by a clip angle welded to the beam web.

Keywords: block shear, coped beams, slotted HSS connections, welded connections.

Block shear describes a failure mode observed in some steel connections in which a combination of tensile and shear failures along perpendicular planes results in a block of material being displaced from a member. This behavior has been observed to govern the design of many bolted connections, because the removal of material for bolt holes outlines a block of reduced net area along which failure may occur. The problem of block shear in bolted connections has been well researched, resulting in design equations of various forms.

In addition to bolted connections, however, certain arrangements of welded connections are also susceptible to block shear. As described subsequently, this failure mode has been observed to govern in both experimental and analytical studies of welded connections, wherein a block of material defined by a perimeter adjacent to the weld toes defines the failure region. Research in this area is limited, and current North American design specifications do not explicitly address this case. The direct application of existing design criteria often results in inaccurate capacity predictions.

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This paper reports on the potential application of the "unified" block shear equation, proposed by Driver et al. (2006) for bolted connections, to various welded connections. Experimental results for welded steel lap plate connections loaded concentrically in tension are examined. The results of these tests are used to describe the connection behavior and to evaluate the performance of several existing design equations, including the unified equation. The application of the unified equation to welded connections of slotted hollow structural sections is also presented and discussed. Finally, the unified equation is considered for the case of coped beams supported at their ends through a clip angle welded to the beam web.

EXISTING BLOCK SHEAR DESIGN EQUATIONS

AISC 2005 Specification

Although the current block shear provisions in North American design specifications do not explicitly address the case of welded connections, the equations provided can reasonably be so applied. In the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005), hereafter referred to as AISC 2005, a nominal block shear capacity, R_n, is calculated to account for rupture along the tension face and simultaneous yielding or rupture of the shear face(s), as given by the lesser of the following two equations:

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

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \tag{2}$$

where

F_y = specified minimum yield stress
F_u = specified minimum tensile strength
A_{gv} = gross area subject to shear
A_{nv} = net area subject to shear
A_{nt} = net area subject to tension
U_{bs} = reduction coefficient; taken as 1.0 for uniform tensile stress and 0.5 for nonuniform tensile stress

If this method is applied to typical welded connections, Equation 1 governs because the absence of bolt holes causes the net and gross areas to be equal. A resistance factor of 0.75 is applied for load and resistance factor design.

Unified Block Shear Equation

Based on an extensive experimental database compiled from the literature by Kulak and Grondin (2001), Driver et al. (2006) demonstrated that a "unified" block shear equation can be used for a wide variety of bolted connection types to provide capacity predictions that are more consistent with experimental observations than other methods, thus resulting in more consistent structural reliability. The unified block shear equation takes the following form:

$$R_n = 0.6\frac{F_y + F_u}{2}A_{gv} + U_t F_u A_{nt} \qquad (3)$$

where

U_t = equivalent tensile stress factor

The equivalent stress factor accounts for nonuniform stress distributions that develop as a result of eccentric loading or asymmetrical blocks, and thus its value depends on the connection type being considered (Driver et al., 2006).

The unified equation has been adopted into the current edition of the Canadian standard, CAN/CSA-S16-09 *Design of Steel Structures* (CSA, 2009), hereafter referred to as CSA-S16-09. The standard contains a table of U_t values for a selection of connection types and specifies a value of 1.0 for symmetrical blocks with concentric loading. A resistance factor of 0.75 is used with the unified equation.

CONCENTRICALLY LOADED LAP PLATE CONNECTIONS

The case of concentrically loaded welded steel lap plate connections is examined because of its frequent occurrence in applications such as trusses and braced frames in buildings. Fifteen specimens tested to failure were considered: 11 from research reported by Topkaya (2007) and an additional 4 carried out by Oosterhof and Driver (2008) as part of the current research. The latter tests, described in the following section, serve to expand the available data set, providing a unique sample of materials and geometries designed to fail at significantly higher loads than those observed by Topkaya.

Experimental Program

A comprehensive description of the test specimens, material properties, test setup, instrumentation, and test procedures

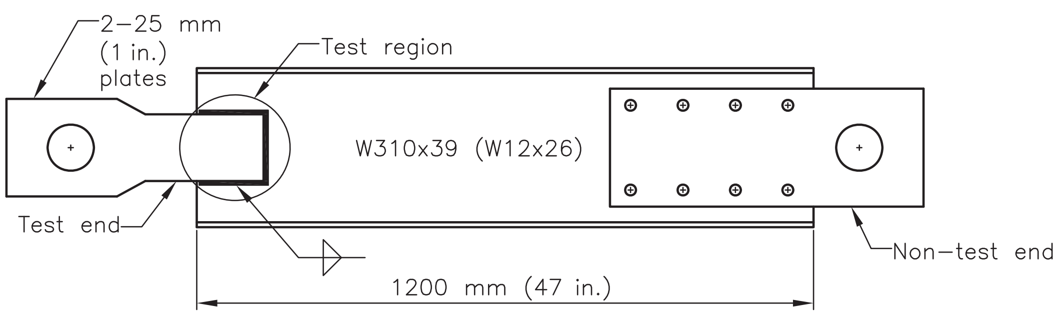


Fig. 1. General arrangement of test specimens.

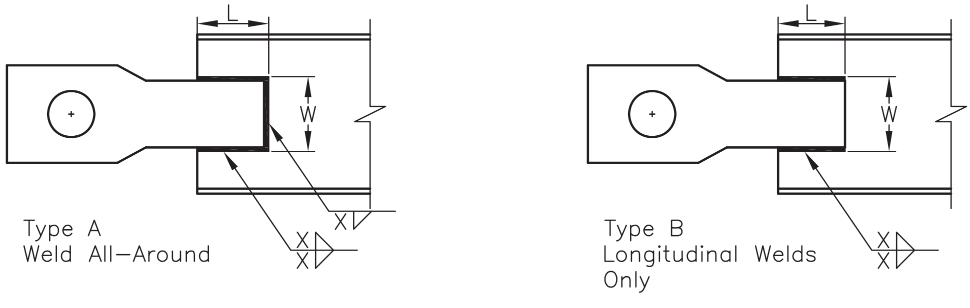


Fig. 2. Test region parameters.

Table 1. Summary of Welded Lap Plate Specimen Parameters and Test Results									
Specimen ID	Weld Type†	Weld Size (mm)	As-built		Block Material Thickness (mm)	Material Properties		Test Failure Load (kN)	Displacement at Failure (mm)
			Length, L† (mm)	Width, W† (mm)		F_y (MPa)	F_u (MPa)		
W1	A	10	146.5	150.4	6.2	379	472	1006	5.9
W2	B	12	144.7	152.2	6.2	379	472	941	4.1
W3	A	10	201.4	103.7	6.2	379	472	1005	6.0
W4	B	10	200.1	100.0	6.2	379	472	963	4.7
Data reported by Topkaya (2007)									
1	A	7	50	60.3	4	309	402	216	—
2	A	7	95.3	57.5	4	309	402	314	—
3	A	7	98.3	77.5	4	309	402	347	—
4	A	7	150.3	78.6	4	309	402	430	—
5	A	7	49.3	98.8	4	309	402	295	—
6	A	7	96	97.8	4	309	402	395	—
7	A	7	150	99.9	4	309	402	475	—
8	A	7	47.3	148.8	4	309	402	386	—
9	A	7	96	148.2	4	309	402	467	—
10	B	7	101.3	56.4	4	309	402	306	—
11	B	7	149.3	98.2	4	309	402	433	—
† Refer to Figure 2; Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa; 1 kip = 4.448 kN									

for the experimental program is reported by Oosterhof and Driver (2008). The following provides a summary of the main parameters.

Description of Test Specimens

Figure 1 shows the general arrangement of the test specimens, which consisted of two lap plates fillet welded to either side of the web of a wide-flange section. The connection was designed to be loaded concentrically in tension and to fail by tearing of a block of material from the 5.8-mm (0.23-in.)-thick web of the wide-flange tension member, in the area labeled "test region" in the figure. The bolted "non-test end" of the specimen was designed with a wide gauge to induce a relatively uniform stress distribution, as would be expected in a longer tension member. Figure 2 defines the parameters in the test region that differentiate the test specimens, namely, connection geometry and weld arrangement. Connection block length-to-width ratios, L/W, of approximately 2:1 and 1:1 were chosen, with the lengths and widths being measured to include the legs of the fillet welds. Connection plates were either welded all-around (type A) or only longitudinally (type B). As-built dimensions are summarized in Table 1.

Material Properties

All material meets the requirements of ASTM A572 Grade 50 and CAN/CSA-G40.21 Grade 350W steel. Tension coupon tests were performed to determine the relevant material properties of the webs of the tension members—all cut from the same piece—with mean values obtained as follows: F_y = 379 MPa (55.0 ksi), F_u = 472 MPa (68.5 ksi) and E = 206,800 MPa (30,000 ksi). The mean rupture strain was 32.1% on a 50-mm gauge length.

Test Procedure

The specimens were loaded in tension in a 6600-kN (1.5 million-lb) universal testing machine (4000 kN in tension). All tests were conducted using stroke control at a rate of 0.5 mm/min (0.02 in./min). Specimens were loaded beyond their maximum capacity until the block of failed material was pulled completely apart from the web or until the applied load had decreased significantly from its peak value. Further details are provided by Oosterhof and Driver (2008).

Test Results

All four test specimens failed in block shear. Figure 3a shows typical whitewash flaking patterns formed during loading, providing an indication of strain and, indirectly, stress distributions. Rupture of the tension surface of the block preceded that of the shear surfaces in all specimens. Material rupture was typically initiated at the ends of the longitudinal welds (near the corners of the lap plates), as shown in Figure 3b. The tear then propagated toward the center of the specimen under continued loading. For all specimens, it was observed that tension failure occurred on a curved, and often irregular, surface. Figure 3c shows a specimen after complete rupture of the block.

Load versus block displacement curves for each test are shown in Figure 4. These curves show consistent loading and unloading patterns and clearly identifiable yield and ultimate loads. The observed failure mode was quite ductile, with block displacements at the ultimate load ranging from

(a) (b) (c)

Fig. 3. Typical yield and rupture patterns: (a) typical strain distribution pattern; (b) tear development on tension face; (c) failed connection.

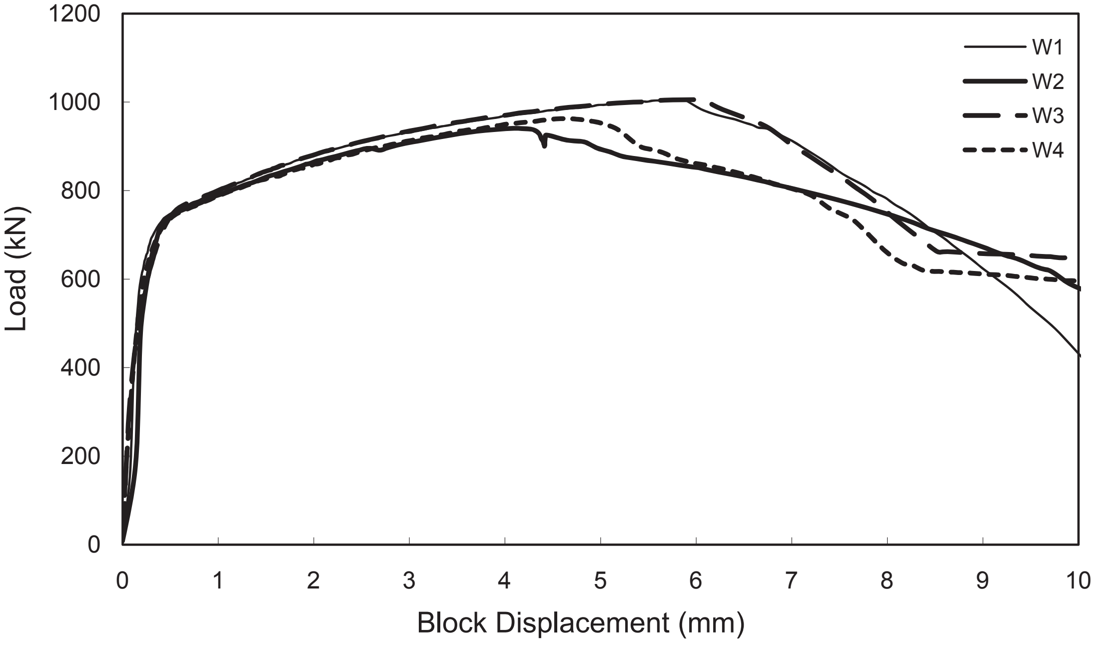


Fig. 4. Load versus block displacement curves for tested lap plate connections.

Table 2. Test-to-Predicted Ratios for Concentrically Loaded Lap Plate Connections							
Experimental Program	Number of Tests	AISC 2005		Topkaya's Equation		Unified Equation $U_t = 1.25$	
		Mean	COV	Mean	COV	Mean	COV
Oosterhof and Driver (2008)	4	1.14	0.029	0.93	0.029	0.99	0.032
Topkaya (2007)	11	1.26	0.042	1.01	0.042	1.07	0.045
Weighted Statistics	15						
Mean		1.20		0.97		1.03	
Coefficient of variation		0.036		0.036		0.039	
Maximum		1.34		1.07		1.15	
Minimum		1.10		0.90		0.94	

10 to 16 times those at first yield. The type A specimens exhibited slightly greater ductility than type B, although this small difference is not considered to be of consequence in this study. A summary of the ultimate failure loads is shown in Table 1, along with the results from the similar tests by Topkaya (2007).

Discussion of Observed Behavior

The curved failure surface observed on the tension faces of all failed members, as seen in Figure 3c, is an important characteristic differentiating the behaviors of welded and bolted connections. The block shear failure perimeter in bolted connections typically follows a straight line between bolt holes, because the removed material defines a reduced net area that is most susceptible to failure. Conversely, the curved failure surface observed in the welded connections increases the physical perimeter of the block, effectively engaging a greater area of material than considered in existing capacity equations. This is considered to be the primary reason the welded connections tested had significantly higher block shear capacities than those calculated by equations derived for bolted connections, as discussed in the following sections.

Performance of Existing Capacity Equations

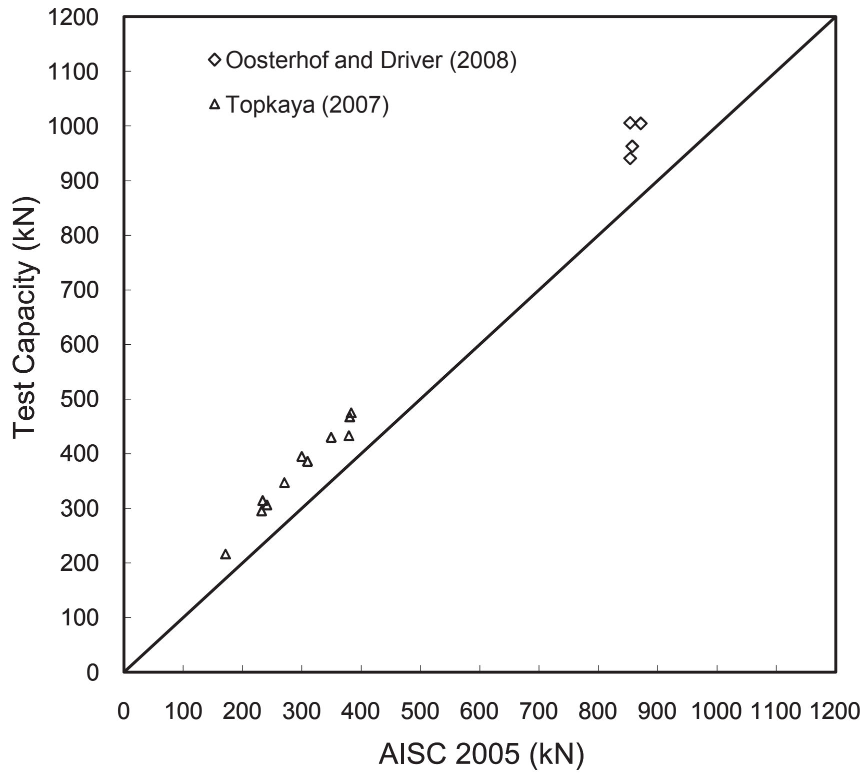
Table 2 summarizes the test-to-predicted ratios for the AISC 2005 provisions, the unified equation (discussed in the next section) and an equation proposed by Topkaya (2007) for block shear of welded connections. In applying these equations, the block perimeter is taken as lying adjacent to the weld toes that define the potential failure surface. The tests performed by Oosterhof and Driver (2008) resulted in significantly higher failure loads than those performed by Topkaya (2007), and, as indicated in Table 2, the larger-scale specimens have appreciably lower test-to-predicted ratios. Recognizing the significance of the apparent difference in behavior between the two sets of specimens, the weighted mean test-to-predicted ratios are also shown in Table 2, giving 50% weight to the 4 larger-scale tests and 50% to the 11 smaller-scale tests. Test-to-predicted ratios for the individual tests are presented by Oosterhof and Driver (2008).

Figure 5a shows that AISC 2005 consistently underestimates block shear capacity for this type of welded connection. While this is conservative, the inaccuracy is significant and may lead to inefficient designs.

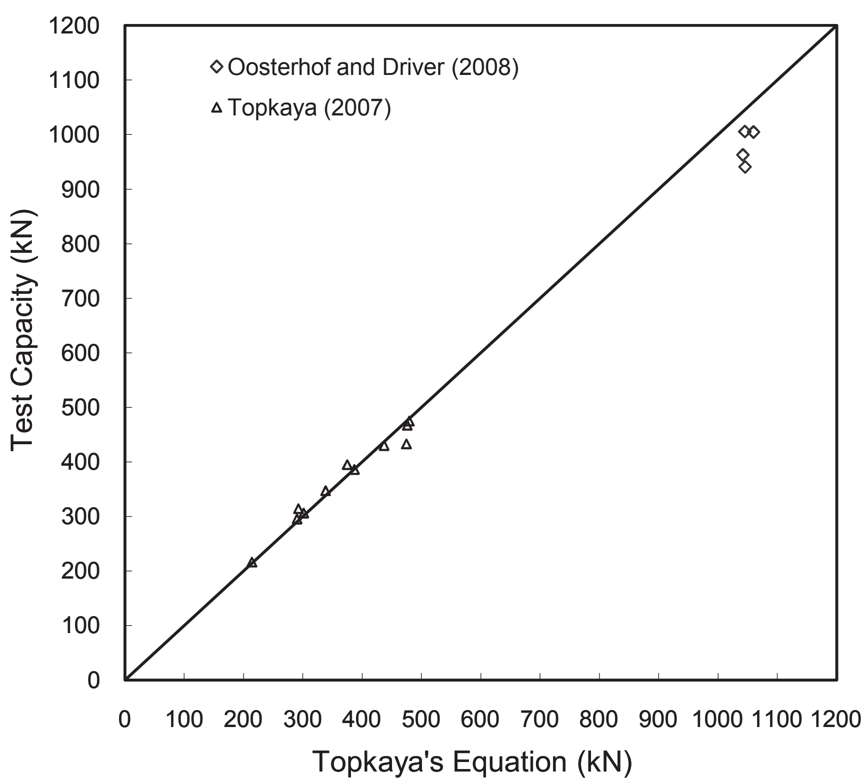
Having observed this incongruity, Topkaya (2007) proposed the following design equation based on physical tests and numerical analyses, which provides significantly improved results:

$$R_n = \frac{F_u}{\sqrt{3}} A_{gv} + 1.25 F_u A_{gt} \tag{4}$$

where all variables have been defined previously. (Note that $1/\sqrt{3}$ is expressed as 0.6 in Equations 1, 2 and 3.) According to Topkaya, the coefficient 1.25 in the second term of this equation accounts for triaxial stress effects on the tensile plane, resulting in significantly higher capacities than those predicted by preexisting design equations. However, Figure 5 suggests that while Equation 4 is accurate for the specimens tested in the research program of Topkaya (2007), it may overestimate the strengths of large-scale connections, as evidenced by the fact that the capacities of the four specimens tested by Oosterhof and Driver (2008) are overestimated by up to 10%.



(a)



(b)

Fig. 5. Comparisons of design equations and experimental results for welded lap plate connections.

Performance of Unified Equation

Equivalent Stress Factor

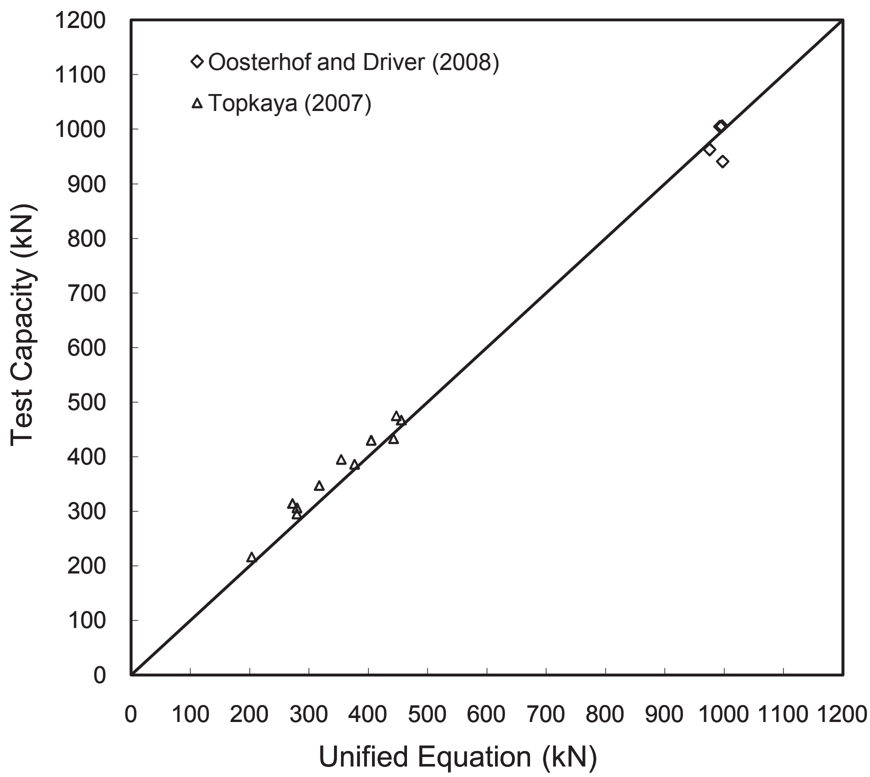
The unified equation (Equation 3) has been shown by Driver et al. (2006) to address inconsistencies in block shear strength predictions associated with existing design specifications. The equivalent stress factor, U_t, allows the equation to be adjusted to account for the presence of nonuniform stress distributions present among various connection configurations. For the welded connections studied in this investigation, Oosterhof and Driver (2008) propose that a value of $U_t = 1.25$ is appropriate for design, because it produces test-to-predicted ratios near unity for the tests considered. This proposal implies that the capacity of the tension plane, which was observed to initiate failure, is expected to increase from the nominal value by about 25% due to a combination of triaxial stress effects and a nonlinear failure path. (Test results show that these effects dominate over a possible capacity decrease that would be caused by the nonuniformity of the stress field.) The resulting capacity equation is consistent with Equation 4, proposed by Topkaya (2007), which also includes a factor to increase tension resistance by 25%.

It is shown in Figure 5 that the application of the unified equation to welded lap plate connections, as proposed by Oosterhof and Driver (2008), achieves considerably improved predictions of the test results as compared to AISC 2005 and slightly more consistent agreement for the two individual sets of test data than the equation proposed by Topkaya (2007).

Reliability Study

Principles of limit states design (or load and resistance factor design) include the statistical determination of failure probability, which can be controlled by the selection of a target reliability index, β. To achieve the target level of safety, an appropriate resistance factor, ϕ, is applied to reduce the predicted capacity. For this study, the resistance factor was calculated based on the method of Ravindra and Galambos (1978), using the same approach as that discussed in detail by Franchuk et al. (2004).

The relevant parameters required to calculate the resistance factor are the material, geometry and professional factors (ρ_M, ρ_G and ρ_P, respectively), and their associated coefficients of variation (V_M, V_G and V_P, respectively). The material and geometry factors are the ratios of mean



(c)

Fig. 5 (cont.). Comparisons of design equations and experimental results for welded lap plate connections.

Table 3. Summary of Reliability Parameters: Lap Plate Connections			
Parameter	Value		
ρ_M	1.05		
ρ_G	1.00		
ρ_P	1.03		
V_M	0.063		
V_G	0.050		
V_P	0.039		
ϕ	0.70	0.75	0.80
β	5.1	4.6	4.1

measured-to-nominal values. The professional factor is the mean test-to-predicted strength ratio for available test results, with the implied presumption that the tests are broadly representative of cases encountered in practice.

The predicted block shear resistance of a steel connection is a function of both the yield and ultimate strengths of the material, and in both cases the mean strengths are generally greater than the nominal values. For the material parameters used in the reliability study, it is conservative to use the values for the static yield strength because the material factor for yield strength is lower and the coefficient of variation higher than those for the ultimate strength. For plates ranging in thickness from 10 to 20 mm (0.4 to 0.8 in.), Schmidt and Bartlett (2002) propose values of 1.07 and 0.054 for the static yield strength material factor and associated coefficient of variation, respectively (values for thinner plates were not reported). For the static yield strength of webs of rolled W-shapes, they recommend values of the material factor and coefficient of variation of 1.05 and 0.063, respectively. Thus, because the two cases give similar material parameters, a slightly conservative approach that can be considered generally suitable for this type of lap plate connection is to use the values for the webs of rolled shapes, providing the lower material factor and the higher coefficient of variation.

The parameters affecting the block shear geometry factor are the material thickness and block perimeter. Block perimeter variability for these welded connections is likely increased somewhat from that of bolted connections, because it depends on both the fabricated size of the lap plates and the weld dimensions. Insufficient data are currently available to determine the overall geometry factor and coefficient of variation for welded lap plate connections. As such, values reported by Hardash and Bjorhovde (1984) for bolted gusset plates are used: geometry factor equal to 1.00 and coefficient of variation equal to 0.050. The proposed geometry factor is believed to be conservative; for example, Zhao and Hancock (1995) reported a value of 1.47 for the weld throat geometry factor in a similar reliability study indicating larger weld sizes than nominal (likely a combination of leg size and face reinforcement). Data reported by Callele et al. (2009) and others also reflect the tendency of welds to be deposited slightly oversized. The variability of the weld size as deposited may be somewhat greater than reflected in the value of the coefficient of variation used, although the weld leg size would constitute the smaller part of the total block perimeter. The geometry factor is also lower and the coefficient of variation higher than the values reported for bolted connections by Franchuk et al. (2004) of 1.017 and 0.039, respectively.

The professional factor is the mean test-to-predicted ratio, with the predicted values determined using measured material and geometric properties. It thus acts as an indication of the level of agreement between the design equation and experimental results. As shown in Table 2, the mean test-to-predicted ratio for the unified equation is 1.03, and the coefficient of variation of these ratios is 0.039.

Using the parameters discussed previously, reliability indices are determined for resistance factors of 0.70, 0.75 and 0.80 for comparison. For connections, the traditional target reliability index is 4.5, although, increasingly, values falling between 4.0 and 4.5 are being considered adequate. Reliability indices of 5.1, 4.6 and 4.1 are obtained for resistance factors of 0.70, 0.75 and 0.80, respectively. Considering the larger-scale tests alone, the reliability indices are 4.8, 4.3 and 3.9, respectively. Based on the results of the reliability analysis, the resistance factor of 0.75 that has been recommended for bolted connections is also considered appropriate for use with the unified equation for the design of concentrically loaded welded lap plate connections. A summary of reliability parameters is shown in Table 3.

SLOTTED HSS-TO-GUSSET PLATE CONNECTIONS

Hollow structural sections (HSS) are commonly connected to gusset plates by cutting a slot into the end of the HSS member, inserting the gusset plate and fillet welding, either along the longitudinal edges only or all around, as shown in Figure 6. These slotted HSS connections are ubiquitous in HSS truss and bracing members. It is possible for this type of connection to fail by block shear of the HSS member when loaded in tension, as most recently reported by Martinez-Saucedo and Packer (2009).

Comparisons to Welded Lap Plate Connections

Conceptually, a slotted HSS member welded to a gusset plate is similar to the case of the welded lap plate connections discussed earlier in that both involve concentric loading resulting in the failure of a nominally rectangular block of material defined by the weld toe perimeter on the connecting element. It was reported based on experimental programs completed by Zhao and Hancock (1995) and Zhao et al. (1999) that block shear failure initiated at the ends of the longitudinal welds farthest from the HSS end, suggesting a failure process similar to that observed in the welded lap plate connections. One notable difference between these two cases, however, is the boundary conditions near the planes of failure. That is, in slotted HSS connections, the gusset plate is connected to a closed HSS section with return walls. Additional significant differences between the two cases include the material properties of HSS sections and the removal of material for the slot from within the block perimeter.

Previous Studies on Slotted HSS-to-Gusset Plate Connections

Martinez-Saucedo and Packer (2009) examined the performance of the unified equation for the block shear failure mode in slotted HSS connections. They reported the results of eight experimental tests—two that failed in block shear, four that failed circumferentially and two that experienced local buckling. This study, along with others before it, identified the limitations of applying existing design equations (namely, Equations 1 and 2) for block shear to this case. Using the results of finite element analyses, Martinez-Saucedo and Packer (2009) recommended the use of the unified equation (Equation 3) with $U_t = 1.0$ for cases where block shear is the governing failure mode. A new design equation to calculate net section capacity for the circumferential failure mode, accounting for shear lag, was also presented.

Research programs conducted by Zhao and Hancock (1995), Korol (1996), Zhao et al. (1999), Wilkinson et al. (2002), and Martinez-Saucedo et al. (2006) include tests of a total of 88 HSS connections. Ling et al. (2007) performed a study on existing test data, adding data of their own from an experimental program on very high strength tubes with a yield strength near 1400 MPa (203 ksi). They proposed modifications to existing design equations, but did not consider the unified equation, as is done later. Capacity prediction of very high strength steel members is beyond the scope of this paper.

The numerical study performed by Martinez-Saucedo et al. (2006) examines the behavior of these connections; however, it is not included in the reliability study discussed later because of the lack of fabrication variability inherent to finite element modeling.

Performance of AISC 2005 Equations

Table 4 shows test-to-predicted ratios comparing existing and proposed design equations to the test results of Zhao and Hancock (1995), Zhao et al. (1999), Wilkinson et al. (2002), and Martinez-Saucedo et al. (2006). [The 11 test results from Korol (1996) are not included in Table 4 or the subsequent reliability study because measured ultimate and yield strengths were not available.] The predicted values used to determine the ratios presented in the table are based on a

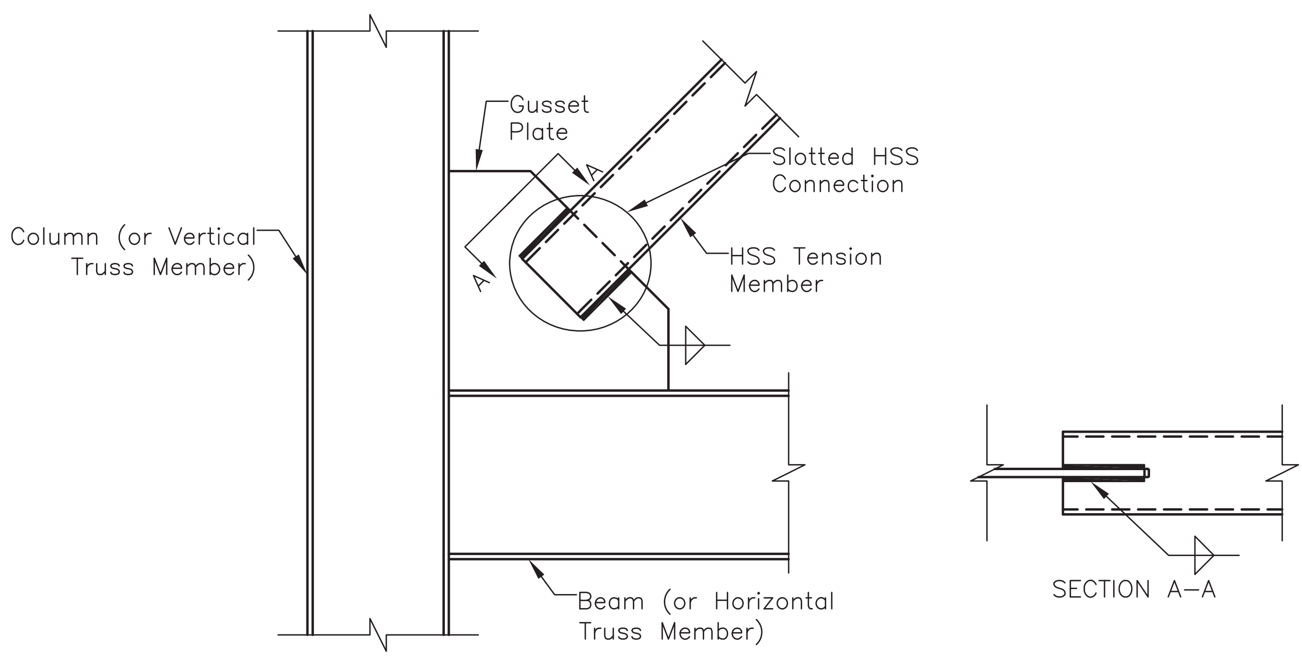


Fig. 6. Schematic of slotted HSS connection.

Table 4. Test-to-Predicted Ratios for Slotted HSS Connections									
Experimental Program	Number of Tests	AISC 2005[a]		Unified Equation[a] $U_t = 1.0$		Unified Equation[a] $U_t = 1.25$		Unified Equation[b] $U_t = 1.25$	
		Mean	COV	Mean	COV	Mean	COV	Mean	COV
Zhao and Hancock (1995)									
Weld type A†	24	1.06	0.05	1.02	0.05	0.97	0.05	1.13	0.05
Weld type B†	24	1.13	0.08	1.10	0.07	1.06	0.07	1.06	0.07
Zhao et al. (1999)									
Weld type A†	12	1.06	0.06	1.04	0.04	0.97	0.04	1.18	0.04
Weld type B†	12	1.12	0.07	1.09	0.06	1.05	0.06	1.05	0.06
Wilkinson et al. (2002)									
Weld type B†	3	1.06	0.04	1.01	0.03	0.98	0.03	0.98	0.03
Martinez-Saucedo et al. (2006)									
Weld type A†	1	0.89	—	0.89	—	0.85	—	0.97	—
Weld type B†	1	1.01	—	1.01	—	0.99	—	0.99	—
Combined Statistics	77								
Mean		1.09		1.06		1.01		1.09	
Coefficient of Variation		0.070		0.065		0.070		0.072	
Maximum		1.31		1.26		1.22		1.29	
Minimum		0.89		0.89		0.85		0.92	

† Refer to Figure 2; [a] Net tension area including slot width for type A; [b] Net tension area excluding slot width

block that includes the weld leg dimension, and the material properties used are those measured and reported by the respective researchers. These studies included both type A and type B specimens (i.e., welded all-around or longitudinally only, as defined in Figure 2). For type B specimens, the area subject to tension includes the weld leg dimensions of the two longitudinal welds only, because of the presence of the slot. The slot width is included as part of the tension area for type A specimens, assuming that the weld material effectively bridges the slot.

The design equations from AISC 2005 are again found to generally underestimate connection capacity, as was the case for lap plate connections. The mean test-to-predicted ratio for all tests is 1.09, which is closer to unity than the value of 1.20 obtained for the lap plate tests. As was suggested earlier, underestimation of the tensile component of the block capacity in welded connections appears to be the main reason for the high test-to-predicted ratios. Because typical slotted HSS connection geometry causes a decreased portion of load to be carried on the tension face compared to lap plate connections, the lower mean test-to-predicted ratio, albeit with a value still greater than unity, is expected.

Performance of the Unified Equation

Equivalent Stress Factor

Two equivalent stress factors for use in the unified equation (Equation 3) are considered: the recommendation of $U_t = 1.0$ by Martinez-Saucedo and Packer (2009) and the recommendation made for lap plate connections of $U_t = 1.25$ to account for the expected increased contribution of the tension face. Results in Table 4 show that while both versions of the unified equation improve capacity predictions, $U_t = 1.25$ achieves test-to-predicted ratios closer to unity with a similar coefficient of variation. This supports the intuitive similarity between the cases of the slotted HSS and lap plate connections. Therefore, for the design of slotted HSS connections against block shear failure, the unified equation as shown in Equation 3, with $U_t = 1.25$, does appear appropriate and is therefore considered in the following comparisons and the reliability study discussed in the next section. However, because the contribution of the tension face is relatively small for typical geometries, using a value of $U_t = 1.0$ is a reasonable and slightly more conservative alternative that results in similar capacity predictions and level of safety.

Observing the results in Table 4, a consistent correlation between test-to-predicted ratio and weld type is evident. Type A specimens in each experimental program have a lower mean test-to-predicted ratio than do type B specimens, for all capacity equations considered. However, with the exception of the single type A test of Martinez-Saucedo et al. (2006), which is discussed later, ratios for both weld types are found to be sufficiently close to unity when the unified equation is used.

Figure 7a shows the relationship between test capacities and capacity predictions using the unified equation with $U_t = 1.25$ for the block shear failures of slotted HSS connections. Generally, the data plot very close to the diagonal line that represents a test-to-predicted ratio of unity. The result of the type A test of Martinez-Saucedo et al. (2006) (test-to-predicted ratio of 0.85) is considered significant because it represents one of only two large-scale test results available. The other of these tests (type B) shows excellent agreement with the unified equation capacity prediction (test-to-predicted ratio of 0.99). These two test specimens are similar in geometry and failure load, but the capacity prediction for the type A test is significantly greater because the area of the tension surface of the block includes the width of the slot bridged by the weld metal. This results in a significant overprediction of capacity in this case.

However, using this same approach for the other 36 tests considered, test-to-predicted ratios near, albeit slightly below, 1.0 were achieved for type A specimens. Based on these observations, it appears that depositing a type A weld around the end of the gusset plate that completely and consistently engages the full tension face of the block (i.e., with no reduction for the slot) may not be readily achievable—a situation that is clearly exacerbated if the slot is significantly longer than the length of gusset plate inserted into the HSS. Moreover, it is possible that engaging the full tension face of the block is more difficult to achieve for large-scale connections than for smaller ones.

Until more large-scale tests similar to those conducted by Martinez-Saucedo et al. (2006) are available, as a conservative approach it is considered prudent to exclude the width of the slot (while still including the width of the fillet weld legs at the end of the longitudinal welds) when calculating the net area in tension for both types A and B connections. As indicated in Table 4, this increases the test-to-predicted ratio using the unified equation for the test in question from 0.85 to 0.97 and the ratio for all tests from 1.01 to 1.09. This effect is shown graphically in Figure 7b. The fact that the mean test-to-predicted ratio for all type A specimens increases to 1.16 indicates that entirely neglecting the slot width is very conservative and could be revisited when additional large-scale type A test data are available.

Table 5. Summary of Reliability Parameters: Slotted HSS Connections

Parameter	Value		
ρ_M	1.18		
ρ_G	1.00		
ρ_P	1.09		
V_M	0.097		
V_G	0.05		
V_P	0.072		
ϕ	0.70	0.75	0.80
β	5.5	5.0	4.6

Reliability Study

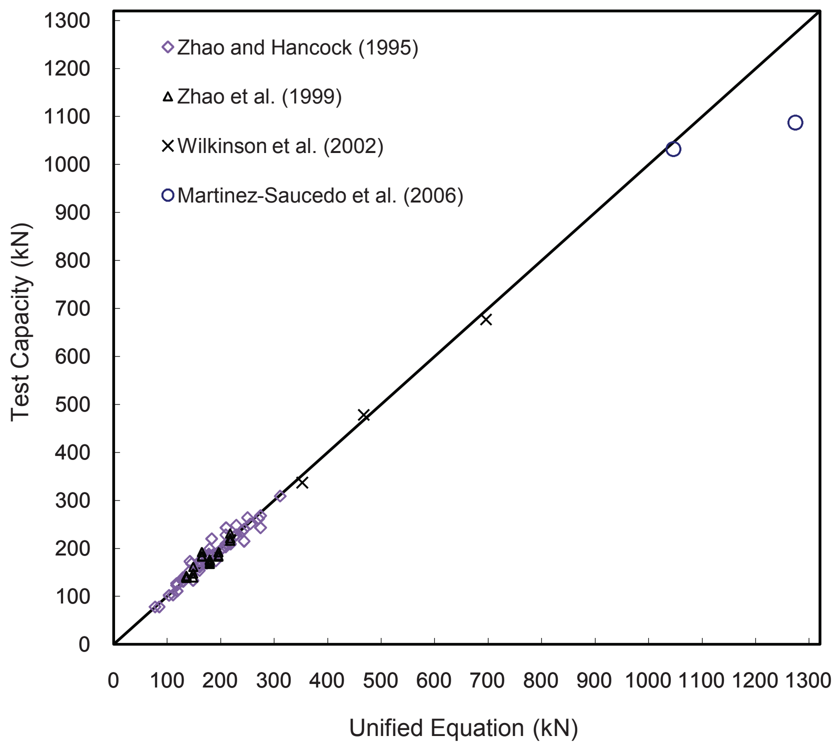
A reliability study was completed to examine the performance of the unified equation (with $U_t = 1.25$) for predicting the capacity of slotted HSS connections. The method used is discussed in the section on lap plate connections.

Material parameters for the HSS members are taken from Schmidt and Bartlett (2002). For ultimate strength, recommended values of the material factor and associated coefficient of variation are 1.18 and 0.063, respectively; for yield strength, the recommended values are 1.35 and 0.097, respectively. It is conservative to take the lower-bound material factor and upper-bound coefficient of variation; thus, values of 1.18 and 0.097 are selected.

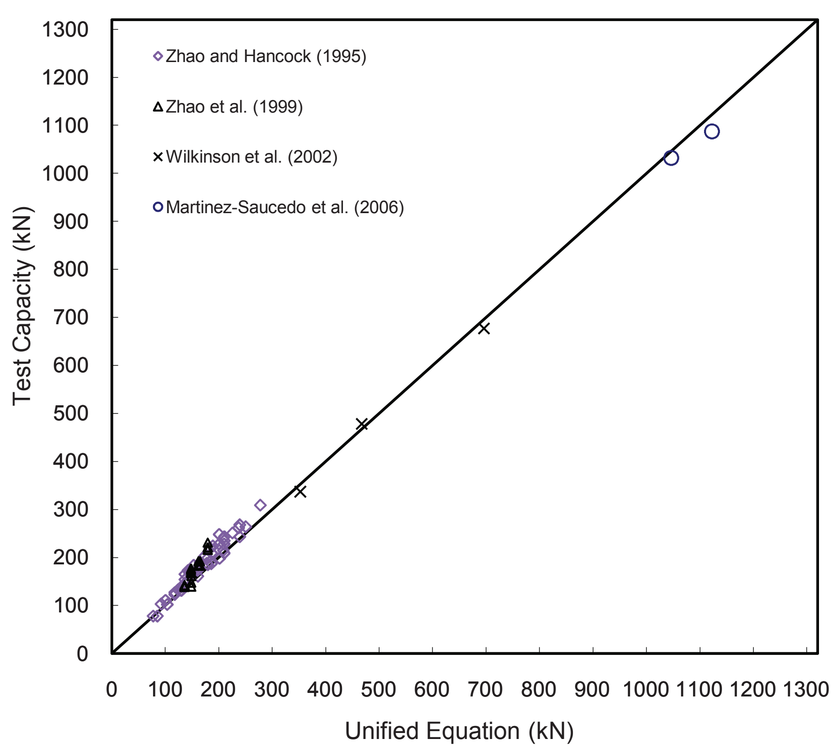
The relevant geometric parameters for slotted HSS connections are the HSS wall thickness and block perimeter. For HSS thickness, Schmidt and Bartlett (2002) report values of the geometry factor and associated coefficient of variation of 0.973 and 0.011, respectively. No statistics are currently available on appropriate geometric parameters that include the variability of the block perimeter of slotted HSS connections. As discussed in the section on lap plate connections, values of 1.00 and 0.05 are assumed to be appropriate—in the absence of better statistical data—for the geometric parameters.

The professional factor is 1.09 (the mean test-to-predicted ratio for the unified equation, excluding the slot width for the determination of the net area in tension, as reported in Table 4), and the associated coefficient of variation is 0.072.

Using these factors, reliability indices of 5.5, 5.0 and 4.6 are obtained for resistance factors of 0.70, 0.75 and 0.80, respectively. Based on these results, the resistance factor of 0.75 recommended for lap plate connections is also considered appropriate for design of slotted HSS connections. The associated reliability index of 5.0 is considered sufficiently high to account for the uncertainty in the selection of geometric parameters. (Note that the reliability index obtained by including the slot width in the predicted capacities of type A specimens is 4.5.) A summary of all reliability parameters is shown in Table 5.



(a)



(b)

Fig. 7. Comparisons of design equations and experimental results for slotted HSS connections: (a) including slot width in net area in tension; (b) excluding slot width from net area in tension.

Table 6. Test-to-Predicted Ratios for Coped Beams with Welded Clip Angles								
	Including Web Buckling Tests				Excluding Web Buckling Tests			
	AISC 2005	Unified Equation			AISC 2005	Unified Equation		
		$U_t = 0.3$	$U_t = 0.7$	$U_t = 0.9$		$U_t = 0.3$	$U_t = 0.7$	$U_t = 0.9$
Mean	1.02	1.38	1.08	0.98	1.07	1.40	1.12	1.02
Coefficient of variation	0.10	0.09	0.09	0.09	0.07	0.07	0.06	0.07
Maximum	1.18	1.60	1.23	1.11	1.18	1.55	1.22	1.11
Minimum	0.81	1.14	0.91	0.80	0.91	1.23	0.97	0.88

COPED BEAMS WITH WELDED CLIP-ANGLE CONNECTION

Another common type of welded connection that can be susceptible to block shear is that of a coped beam with a clip-angle connection to its web, as depicted in Figure 8. An important distinction between this case and those discussed earlier is the eccentric loading condition on the block, resulting in stress concentrations that reduce connection capacity.

Previous Studies on Block Shear Capacity of Coped Beams

The block shear behavior of coped beams with bolted connections has been researched by Franchuk et al. (2003, 2004). The results of that study led to the recommendation of the unified equation for design, with $U_t = 0.9$ for one row of bolts and $U_t = 0.3$ for two rows. Yam et al. (2006a, 2006b) and Wei et al. (2010) examined the case of coped beams with welded double-clip-angle connections, including 22 physical tests and a parametric study using finite element modeling. A total of 10 tests failed by block shear. Although eight of the test beams exhibited local web buckling at failure, the researchers report that prior to this, a significant "block shear type" deformation occurred in all cases, indicating that the block shear capacity had been approached, or perhaps even reached. As such, these test results are included in this study. Results excluding the tests in which web buckling occurred are also reported for comparison, although it should be noted that all of the tests where web buckling did not occur had small connection eccentricities, making it impractical to assess the effect of eccentricity on block shear capacity using only these data. One test failed in the weld and three were not loaded to failure; these four tests are not included in the following discussion.

Performance of Existing Capacity Equations

Table 6 shows test-to-predicted ratios comparing existing and proposed design equations to the test results from Yam et al. (2006a) and Wei et al. (2010). AISC 2005 does not state whether U_{bs} should be taken as 1.0 or 0.5 in Equation 1 for this case. Taking this connection to be similar to the case of "single-row beam end connections," as shown in the AISC 2005 Commentary, values shown in Table 4 assume a value of 1.0 for U_{bs}.

Based on experimental results and subsequent finite element analyses, Yam et al. (2006b) proposed the use of Equation 1 from AISC 2005 with the inclusion of additional factors on both the shear and tension terms that are functions of the connection geometry and, in the case of the tension term, the ultimate strength of the material. These terms were developed from their test results to obtain a mean test-to-predicted ratio near unity and to minimize the coefficient of variation. Although the equation proposed by Yam et al. (2006b) achieves excellent agreement with their test results, it is desirable to use a simpler approach that is consistent with that used for other types of connections; thus, the performance of the unified equation is examined in the following section. Furthermore, the previous research did not perform a reliability study to quantify the statistical level of safety achieved by the design equation.

Performance of the Unified Equation

Equivalent Stress Factor

The unified equation has been shown by Franchuk et al. (2004) to provide good results for bolted connections in

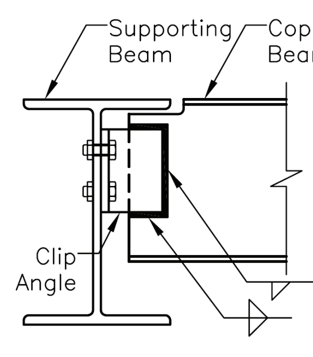


Fig. 8. Schematic of coped beam with a welded clip-angle connection.

Table 7. Summary of Reliability Parameters: Coped Beams with Welded Clip Angles						
Parameter	Including Web Buckling Tests			Excluding Web Buckling Tests		
U_t	0.3	0.7	0.9	0.3	0.7	0.9
ρ_M		1.05			1.05	
ρ_G		1.00			1.00	
ρ_P	1.38	1.08	0.98	1.40	1.12	1.02
V_M		0.063			0.063	
V_G		0.050			0.050	
V_P	0.09	0.09	0.09	0.07	0.06	0.07
ϕ		0.75			0.75	
β	6.0	4.4	3.7	6.5	4.9	4.2

coped beams using a U_t value of 0.9 for one row of bolts and 0.3 for two rows of bolts (consequently, these values have been adopted into CSA-S16-09). Wei et al. (2010) also calculate the predicted capacity of the specimens using the unified equation with these values of U_t, although they do not perform a reliability study to quantify the level of safety achieved, nor do they include specimens where web buckling occurs after significant block shear deformation. Considering the geometry of typical welded clip-angle connections, it is expected that a value between these limits would be appropriate to account for stress concentrations and to provide accurate capacity predictions for these connections. It was found that for the tests of Yam et al. (2006a) and Wei et al. (2010), a test-to-predicted ratio of unity can be achieved using the unified equation (Equation 3) with a value of $U_t = 0.86$ for all tests, or $U_t = 0.93$ excluding tests showing web buckling at failure; both of these results are similar to the value proposed previously for single-row bolted connections (0.9). It should be noted, however, that connection geometries with greater eccentricities than those accounted for in the limited available test data may require a reduction of the U_t value. The available test data include tension face lengths between 60 mm (2.4 in.) and 110 mm (4.3 in.). As expected, the two lowest test-to-predicted ratios from all 18 tests correspond to two of the specimens with the largest weld eccentricities.

Reliability Study

A reliability study was completed to examine the performance of the unified equation for predicting the capacity of coped beams with welded clip angle connections. Material and geometric parameters are taken from the reliability study on lap plate connections, due to the similarity in relevant design parameters, and are shown in Table 7.

The extreme values of U_t found appropriate in the unified equation for bolted coped beam connections, 0.9 and 0.3, were examined for applicability to welded connections. The reliability study is completed both including and excluding the tests in which web buckling took place, because significant block shear deformations were observed even when web buckling occurred. As reported in Table 6, for $U_t = 0.9$, the coefficient of variation for all the tests is 0.09. This coefficient of variation is significantly higher than those observed for the other types of connections considered in this study, resulting in a lower reliability index, β. While neglecting the tests where web buckling was present improves this value, the mean test-to-predicted ratios for the two cases are similar for all values of U_t considered, which suggests that the block shear capacity of these specimens was effectively reached prior to buckling. For $U_t = 0.3$, the professional factor for both groups is 1.4; this is clearly a gross underprediction of capacity. In order to achieve an acceptable value of β with a resistance factor, ϕ, of 0.75 (the value used for bolted connections and the welded connections discussed above), an intermediate value of $U_t = 0.7$ was also considered.

A summary of all reliability parameters is shown in Table 7 for both groups of test specimens. For a resistance factor of 0.75, and considering all of the test results, reliability indices of 6.0, 4.4 and 3.7 are obtained using U_t values of 0.3, 0.7 and 0.9, respectively, with the unified equation. The reliability index obtained for $U_t = 0.9$ is well below the target value—even though it is associated with a professional factor near unity—due to the relatively large scatter in the experimental results. An acceptable reliability index is achieved for $U_t = 0.7$, which generally provides conservative capacity estimates. Based on the limited test data available, a U_t value of 0.7 is currently recommended for use with the unified equation for predicting the block shear capacity of coped beams with welded clip-angle connections, with a resistance factor of 0.75. However, given that the only available test results where web buckling was not observed are from connections with small eccentricities, and because considering only these tests leads to a conclusion that $U_t = 0.9$ provides an adequate level of safety, further testing may reveal that a value of $U_t = 0.9$ for very small eccentricities that decreases with increasing eccentricity is most appropriate—an approach that is currently used with the unified equation for bolted coped beam connections.

SUMMARY AND CONCLUSIONS

The results of an experimental study on the block shear behavior of welded lap plate connections are presented and discussed. When compared to experimental results, the block shear equations in AISC 2005 are found to be excessively conservative when applied to welded connections. The unified equation (Equation 3), with an equivalent stress factor $U_t = 1.25$, is recommended for the prediction of the block shear capacity of concentrically loaded welded lap plate connections. A resistance factor of 0.75 is shown to be appropriate.

The application of the unified equation to welded slotted HSS connections is also examined. Based on results from experimental programs by Zhao and Hancock (1995), Zhao et al. (1999), Wilkinson et al. (2002) and Martinez-Saucedo et al. (2006), the unified equation (Equation 3) with an equivalent stress factor of $U_t = 1.25$ and a resistance factor of 0.75 is again shown to be appropriate for the prediction of the block shear capacity. As a slightly more conservative alternative, a value of $U_t = 1.0$ [proposed by Martinez-Saucedo and Packer (2009)] provides similar results due to the relatively small influence of the tension surface on the overall block shear capacity for typical geometries. It is recommended that the area of steel removed from the tension plane by the slot be excluded in the determination of the net area in tension for both type A and B connections.

The performance of the unified equation for the case of coped beams with a welded clip-angle connected to the beam web is also considered, using experimental data reported by Yam et al. (2006a) and Wei et al. (2010). Although the use of the unified equation with an equivalent stress factor of $U_t = 0.9$ gives test-to-predicted ratios close to unity, a reliability study has revealed that due to the large scatter in the test data, this value provides an inadequate level of safety over the range of connection eccentricities studied. Consequently, until further test results are available, an equivalent stress factor of $U_t = 0.7$ is considered appropriate for design, with a resistance factor of 0.75.

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REFERENCES

AISC (2005), *Specification for Structural Steel Buildings*, ANSI/AISC 360-05, American Institute for Steel Construction, Inc., Chicago, IL.

Callele, L.J., Driver, R.G. and Grondin, G.Y. (2009), "Design and Behavior of Multi-orientation Fillet Weld Connections," *Engineering Journal*, American Institute of Steel Construction, Vol. 46, Fourth Quarter, pp. 257–272.

CSA (2009), *Limit States Design of Steel Structures*, CAN/CSA-S16-09, Canadian Standards Association, Toronto, ON, Canada.

Driver, R.G., Grondin, G.Y. and Kulak, G.L. (2006), "Unified Block Shear Equation for Achieving Consistent Reliability," *Journal of Constructional Steel Research*, Vol. 62, No. 3, pp. 210–222.

Franchuk, C.R., Driver, R.G. and Grondin, G.Y. (2003), "Experimental Investigation of Block Shear Failure in Coped Steel Beams," *Canadian Journal of Civil Engineering*, Vol. 30, No. 5, pp. 871–881.

Franchuk, C.R., Driver, R.G. and Grondin, G.Y. (2004), "Reliability Analysis of Block Shear Capacity of Coped Steel Beams," *Journal of Structural Engineering*, Vol. 130, No. 12, pp. 1904–1913.

Hardash, S.G. and Bjorhovde, R. (1984), "Gusset Plate Design Utilizing Block-Shear Concepts," Department of Civil Engineering and Engineering Mechanics, University of Arizona, Tuscon, AZ.

Korol, R.M. (1996), "Shear Lag in Slotted HSS Tension Members," *Canadian Journal of Civil Engineering*, Vol. 23, pp. 1350–1354.

Kulak, G.L. and Grondin, G.Y. (2001), "Block Shear Failure in Steel Members—A Review of Design Practice," *Engineering Journal*, American Institute of Steel Construction, Vol. 38, No. 4, pp. 199–203.

Ling, T.W., Zhao, X.L., Al-Mahaidi, R. and Packer, J.A. (2007), "Investigation of Block Shear Tear-out Failure in Gusset-plate Welded Connections in Structural Steel Hollow Sections and Very High Strength Tubes," *Engineering Structures*, Vol. 24, No. 4, pp. 469–482.

Martinez-Saucedo, G., Packer, J.A. and Willibald, S. (2006), "Parametric Finite Element Study of Slotted End Connections to Circular Hollow Sections," *Engineering Structures*, Vol. 28, pp. 1957–1971.

Martinez-Saucedo, G. and Packer, J.A. (2009), "Static Design Recommendations for Slotted End HSS Connections in Tension," *Journal of Structural Engineering*, Vol. 135, No. 7, pp. 797–805.

Oosterhof, S.A. and Driver, R.G. (2008), "Block Shear Behaviour of Concentrically Loaded Welded Steel Lap Plate Connections," *Proc. Annual Conference, Canadian Society for Civil Engineering*, Vol. 3, pp. 2014–2023.

Ravindra, M.K. and Galambos, T.V. (1978), "Load and Resistance Factor Design for Steel," *Journal of the Structural Division*, Vol. 104, ST9, pp. 1337–1353.

Schmidt, B.J. and Bartlett, F.M. (2002), "Review of Resistance Factor for Steel: Data Collection," *Canadian Journal of Civil Engineering*, Vol. 29, pp. 98–108.

Topkaya, C. (2007), "Block Shear Failure of Gusset Plates with Welded Connections," *Engineering Structures*, Vol. 29, No. 1, pp. 11–20.

Wei, F., Yam, M.C., Chung, K.F. and Grondin, G.Y. (2010), "Tests on Block Shear of Coped Beams with a Welded End Connection," *Journal of Constructional Steel Research*, Vol. 66, pp. 1398–1410.

Wilkinson, T., Petrovski, T., Bechara, E. and Rubal, M. (2002), "Experimental Investigation of Slot Lengths in RHS Bracing Members," *Advances in Steel Structures*, Vol. 1, pp. 205–212.

Yam, M.C., Zhong, Y.C., Lam, A.C.C. and Iu, V.P. (2006a), "An Investigation of the Block Shear Strength of Coped Beams with a Welded Clip Angle Connection—Part I: Experimental Study," *Journal of Constructional Steel Research*, Vol. 63, pp. 96–115.

Yam, M.C., Zhong, Y.C., Lam, A.C.C. and Iu, V.P. (2006b), "An Investigation of the Block Shear Strength of Coped Beams with a Welded Clip Angle Connection—Part II: Numerical Study," *Journal of Constructional Steel Research*, Vol. 63, pp. 116–134.

Zhao, X.L. and Hancock, G.J. (1995), "Longitudinal Fillet Welds in Thin Cold-Formed RHS Members," *Journal of Structural Engineering*, Vol. 121, No. 11, pp. 1683–1690.

Zhao, X.L., Al-Mahaidi, R. and Kiew, K.P. (1999), "Longitudinal Fillet Welds in Thin-Walled C450 RHS Members," *Journal of Structural Engineering*, Vol. 125, No. 8, pp. 821–828.