Prediction of Bolted Connection Capacity for Block Shear Failures along Atypical Paths

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ABSTRACT

Failure modes such as bolt tear-out and the so-called alternate block shear path observed in tees are closely related to the classical block shear limit state, but they have not been addressed as such in current design specifications in North America. In previous work conducted at the University of Alberta, a unified block shear equation was proposed that provides accurate test-to-predicted block shear capacity ratios and results in consistent reliability indices over a variety of connection types. A total of 104 specimens that failed in bolt tear-out and 14 tees that failed on the alternate block shear path are considered from the literature, along with 12 new bolt tear-out tests conducted as part of this research program. It is shown that the unified block shear equation provides accurate and consistent results for these failure modes as well.

Keywords: block shear, bolt tear-out.

INTRODUCTION

Block shear is a well-documented failure mode that can occur in connections when a block of material in the connected region is displaced due to tension fracture on one plane of the block perimeter in combination with shear on one or more others. Bolt tear-out and failure in tees along the "alternate block shear" path can be considered block shear failures with atypical failure paths, and this paper investigates the suitability of different methods of predicting block shear capacity specifically for these modes.

Bolt tear-out failure generally occurs by shear tearing along the two planes adjacent to the bolt hole, and there is no tension fracture in the block of material due to the presence of the hole itself. This path is illustrated in Figure 1a. Bearing is a closely related failure mode and is considered to constitute failure by the excessive deformation of material behind the bolt. If connection deformation is not a design consideration, the ultimate strength of a bolted connection with a relatively small end distance and pitch that fails locally around the bolts would generally be governed by bolt tear-out rather than bearing. Nevertheless, capacities determined using the AISC bearing provisions for when hole deformation at the service load is not a design consideration (Equation J3-6b; AISC, 2005) are also examined for comparison to the block shear approach.

A failure mode observed by Epstein and Stamberg (2002) in tees connected by bolts through the flange only, which was termed "alternate block shear" failure by the researchers, is depicted in Figure 1b. This failure mode is similar to traditional block shear except that it has only one shear plane in the tee stem and tension fracture involves the entire flange.

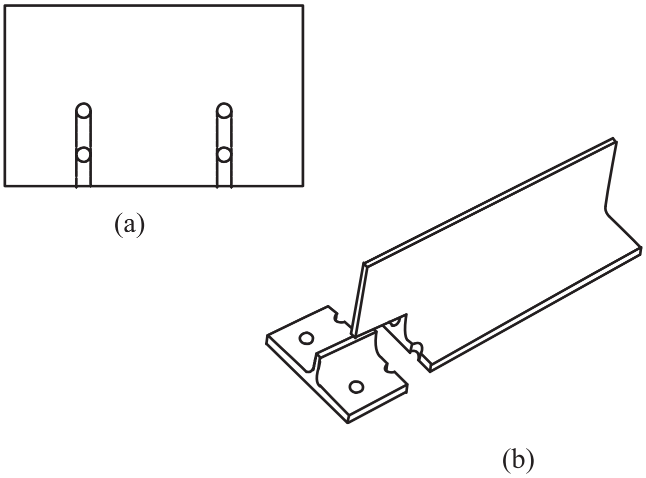

Fig. 1. Failure paths considered: (a) bolt tear-out; (b) alternate block shear path in tees.

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DESIGN EQUATIONS

CSA-S16-01 and AISC 360-05

The provisions in the current North American design standards, CSA-S16-01 (CSA, 2001) and the AISC *Specification* (AISC, 2005), for predicting the block shear capacity of tension members with concentrically loaded symmetrical blocks are essentially identical. The block shear capacity is taken as the lesser of:

$$P_r = \phi A_{nt} F_u + 0.60\phi A_{gv} F_y \tag{1}$$

$$P_r = \phi A_{nt} F_u + 0.60\phi A_{nv} F_u \tag{2}$$

Both design specifications provide an effective stress factor on the first (tension) term that is applied in certain circumstances when the stress distribution on the tension plane is nonuniform due to starkly asymmetrical loading on the block. Equation 1 applies when the net tension area, A_{nt}, reaches the ultimate tensile strength, F_u, and the gross shear area, A_{gv}, reaches the shear yield strength, $0.6F_y$. This phenomenon has been observed by many researchers (e.g., Franchuk et al., 2003). However, Equation 2, representing the development of the ultimate capacities of both the net tension area and net shear area, A_{nv}, is not supported by test observations. On the contrary, experimental evidence (e.g., Huns et al., 2002) indicates that tension fracture occurs well before shear fracture and, although the shear yield stress is exceeded, the ductility of the material in tension is inadequate to allow the ultimate shear strength to be reached concurrently with the ultimate tensile strength on the tension area of the block.

There is no equation in the current design specifications in North America given explicitly for bolt tear-out failure as depicted in Figure 1a. However, the AISC *Specification* Commentary (AISC, 2005) states that the bearing provisions can be applied to address the tear-out limit state. Alternatively, design equations for block shear could be used as shown, for instance, in a design example in the CISC *Handbook of Steel Construction* (CISC, 2006). This latter approach is clearly based on the assumption that bolt tear-out is a type of block shear failure. In this case, Equations 1 and 2 become (the lesser of):

$$P_r = 0.60\phi A_{gv} F_y \tag{3}$$

$$P_r = 0.60\phi A_{nv} F_u \tag{4}$$

These block shear equations imply that both gross section yield and net section rupture of the shear planes are possible failure modes, and therefore, both must be checked.

Unified Equation

Based on a large number of experimental results from the literature, Kulak and Grondin (2001) observed that equations existing at that time were inconsistent in predicting the capacities of connections failing in block shear. To address this deficiency, Driver et al. (2006) proposed a single unified block shear equation that has been shown to provide excellent results for a variety of member and connection types failing in block shear. It represents the observation from tests that rupture on the net tension area tends to occur well after yielding has taken place on the gross shear plane, but prior to shear rupture. The effective shear stress in the unified equation is taken as the average of the shear yield and shear ultimate stresses to reflect this fact. The equation also reflects the fact that failure of the shear planes occurs on the gross section adjacent to the holes (Driver et al., 2006). For tension members with symmetrical blocks, it takes the following form:

$$P_r = \phi A_{nt} F_u + \phi A_{gv} \left(\frac{F_y + F_u}{2\sqrt{3}} \right) \tag{5}$$

Effective stress factors were provided on both terms of the unified equation to allow for nonuniform stress distributions on the block perimeter, although the factor on the second (shear) term can generally be taken as 1.0. The unified block shear equation can be applied to the case of bolt tear-out simply by eliminating the tension component:

$$P_r = \phi A_{gv} \left(\frac{F_y + F_u}{2\sqrt{3}} \right) \tag{6}$$

It is postulated that the unified block shear equation can be adopted for a truly unified equation that is also suitable for predicting bolt tear-out failure at the ultimate limit state. It is investigated herein for use with this mode, as well as for the alternate block shear failure path in tees, where both the tension and shear terms are needed.

PREVIOUS RESEARCH

Although many bolt tear-out tests have been conducted on very high strength steels, due to their demonstrably different behavior, this study focuses on common grades of steel with yield strengths no greater than 550 MPa (80 ksi). Considering these grades only, Udagawa and Yamada (1998) conducted 146 tests on plates, and 31 of them failed by bolt tear-out. For these 31 tests, the number of bolt lines running in the direction of the applied load was one or two, while the number of bolt rows running in the direction perpendicular to the applied load varied from two to four. Kim and Yura

Table 1. Tests from Previous Research

Author (Year)	Section Type	Number of Tests			Mean T/P Ratio Criterion A (COV)		
		Total	Criterion A[a]	Criterion B[b]	S16-01/ AISC 360-05	AISC 360-05 Bearing	Unified Equation
Bolt Tear-out							
Udagawa & Yamada (1998)	Plate	31	0	31	—	—	—
Kim & Yura (1999)	Plate	19	9	19	1.24 (0.14)	0.97 (0.08)	0.95 (0.13)
Aalberg & Larsen (2001, 2002)	Plate	20	10	20	1.17 (0.13)	0.94 (0.08)	0.98 (0.12)
Puthli & Fleischer (2001)	Plate	9	0	9	—	—	—
Rex & Easterling (2003)	Plate	20	11	20	1.21 (0.07)	0.93 (0.11)	0.99 (0.08)
Udagawa & Yamada (2004)	Channel	5	0	5	—	—	—
Alternate Block Shear							
Epstein & Stamberg (2002)	Tee	14	14	14	1.08 (0.09)	—	1.05 (0.09)

a. Tests that meet the minimum end distance and bolt spacing requirements specified in North American design specifications ($F_y \le 550$ MPa; 80 ksi).

b. Includes tests that do not meet the minimum end distance and bolt spacing requirements specified in North American design specifications ($F_y \le 550$ MPa; 80 ksi).

(1999) carried out 19 tests on plates with one or two bolts in a single line parallel to the applied load and all of the tests failed by bolt tear-out. Aalberg and Larsen (2001, 2002) used the connection configurations of Kim and Yura (1999) and tested eight one-bolt connections and 12 two-bolt connections, and all specimens failed by bolt tear-out. Puthli and Fleischer (2001) completed 25 tests on plates that had two bolts in a row perpendicular to the applied load, and nine of them failed in the bolt tear-out mode. Rex and Easterling (2003) conducted 46 single bolt bearing tests, and 20 plates ultimately failed by bolt tear-out.

Udagawa and Yamada (2004) carried out 42 tests on web-connected channel sections, and five of them failed by bolt tear-out. All five specimens had a single bolt line in the web in the direction of the applied load, and the number of bolts varied from two to four.

Epstein and Stamberg (2002) conducted 50 tests on a wide variety of flange-connected tees cut from standard wide-flange shapes. Fourteen specimens failed along the alternate block shear path, all but one of which had two bolt rows aligned in the direction perpendicular to the applied load (the other had three).

There are a significant number of tests reported in the literature for which bolt tear-out is the ultimate failure mode. However, to ensure that the bolt tear-out mode governs the failure, most connection configurations tested do not meet the minimum end distance and bolt spacing requirements specified in North American design specifications, and many have only one or two bolts. Table 1 presents the number of test connections that failed by bolt tear-out from seven different research projects and the number of tests that meet each of criteria A and B, as defined in the table. Test-to-predicted values are provided for both the block shear and bearing design equations for criterion A.

The last row of Table 1 represents the tests that failed along the "alternate block shear" path in tees. Due to the relatively small eccentricity in flange-connected tees, an effective stress factor equal to 1.0 has been applied to the stresses on the tension plane in the predicted values.

Table 2. Measured Web Connection Dimensions

Specimen	Section	Hole Diameter, d_0 (mm)	Web Thickness, w (mm)	End Distance, e_1 (mm)	Pitch, p (mm)	Gage, g (mm)
A1G1	W310×60	20.6	7.48	28.3	54.3	140.6
A2G1	W310×60	20.6	7.52	29.3	54.2	139.9
A3R1	W310×39	20.4	6.30	28.2	53.8	178.8
A4R2	W310×39	20.6	6.22	28.3	54.1	178.6
A5E1	W250×49	20.5	7.55	31.0	54.1	140.1
A6E2	W250×49	20.5	7.51	47.7	54.1	140.0
A7G1	W310×60	20.8	7.43	28.6	53.8	139.7
A8G2	W310×60	20.8	7.44	27.1	54.1	177.8
A9R1	W310×39	20.7	6.54	27.6	53.6	178.2
A10R2	W310×39	20.8	6.55	27.1	54.3	178.2
A11E1	W250×49	20.6	7.30	28.3	53.7	140.2
A12E2	W250×49	20.7	7.34	44.0	54.3	139.9

Note: 1 in. = 25.4 mm

EXPERIMENTAL PROGRAM

The bolt tear-out experimental program conducted as part of this research included 12 specimens that were connected through the web only, using three different wide-flange sections meeting the requirements of both CSA-G40.21 Grade 350W and ASTM A572 Grade 50 steel. The three main variables were the gage, g (G in the specimen designation), number of bolt rows (R), and end distance, e_1 (E). The connection dimensional parameters are shown in Figure 2 and the associated measured values are listed in Table 2. As depicted in Figure 2, all specimens had four bolts except the two R2 specimens, which had six. Although the gage was varied, in no case did a block tear-out occur that encompassed the entire bolt pattern; rather, the webs tore along each individual bolt line.

The test setup is depicted schematically in Figure 3. All specimens were 1220 mm (48 in.) long. Specimens were

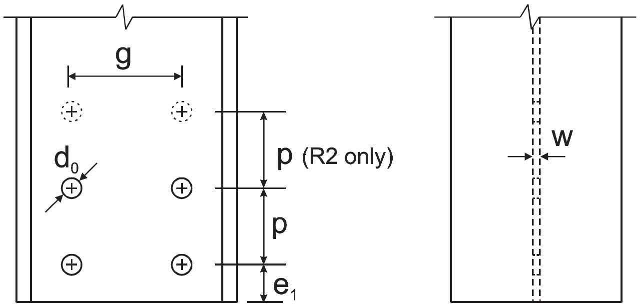


Fig. 2. Connection dimensional parameters.

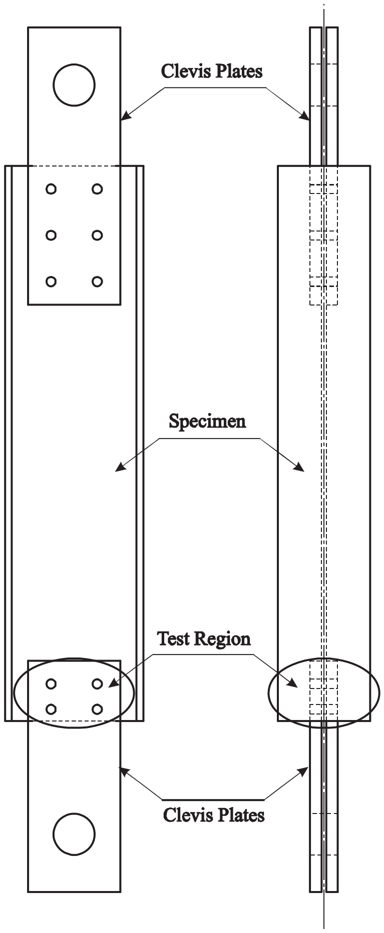


Fig. 3. Test setup.

Table 3. Summary of Test Results

Specimen	F_y (MPa)	F_u (MPa)	Peak Load (kN)	Unloading Point
A1G1	439	519	690.7	drop of 5% of the peak load
A2G1	439	519	723.8	right after the peak load
A3R1	379	472	634.1	right after the peak load
A4R2	379	472	912.7	right after the peak load
A5E1	343	487	697.7	right after the peak load
A6E2	343	487	775.8	after a sudden load drop
A7G1	411	494	665.1	drop of 5% of the peak load
A8G2	411	494	622.1	right after the peak load
A9R1	369	478	632.8	drop of 5% of the peak load
A10R2	369	478	766.1	drop of 5% of the peak load
A11E1	376	500	691.2	drop of 5% of the peak load
A12E2	376	500	792.6	drop of 5% of the peak load

Note: 1 ksi = 6.895 MPa; 1 kip = 4.448 kN

connected to clevis plates at both ends, which were in turn connected to the testing machine by pin connections. The clevis plates were designed to remain elastic so they could be reused. This would be a common scenario for cases where bolt tear-out of the internal plate is the governing mode of failure.

Bolts used in the tests were ASTM Grade A490, with a diameter, d_b, of 19.1 mm (¾ in.). The pitch, as a fixed parameter, was nominally 54 mm (2⅛ in.) because both CSA-S16-01 and AISC 360-05 specify that the pitch should not be less than $2.7d_b$. The minimum end distance for the bolts is 25 mm (1 in.) for gas-cut edges. (CSA-S16-01 also specifies that the end distance should not be less than $1.5d_b$ for connections that have either one or two bolts in a line in the direction of the applied force, but this was neglected because it does not apply to both design specifications considered.) All bolt holes were drilled and of standard size, namely, 20.6 mm (13⁄16 in.). Bolts had standard thread lengths that excluded the threads from the shear planes and were tightened to the snug-tight condition as defined in CSA-S16-01 (CSA, 2001).

Ancillary material tensile tests were conducted as per ASTM standard A370 (ASTM, 2007). Three coupons were fabricated from the web of each section in the direction of the applied load. Mean test results for each set of coupons are listed in Table 3.

Specimens were tested in tension in a universal testing machine (MTS 6000). The load was applied quasi-statically under stroke control. One of two typical unloading points was chosen as the terminus of each test: "right after the peak load" and "drop of 5% of the peak load." The former was selected in order to observe the load-carrying mechanism right at the peak load, whereas the latter was chosen to ensure that the ultimate strength of the connection had indeed been captured. Cai and Driver (2008) provide a complete description of the test set-up and experimental procedures.

TEST RESULTS

Test results are summarized in Table 3. All specimens failed by bolt tear-out of the web.

Two kinds of fractures were observed in the bolt tear-out failures: shear tears on one or both shear planes adjacent to the hole, as shown in Figure 4a, or a single tensile splitting crack initiating at the free edge near the hole centerline, as shown in Figure 4b. Tensile splitting cracks were caused by the development of transverse tensile stress as the material behind the bolt shank deformed into an arch shape. Most specimens eventually exhibited either shear tears or splitting cracks, although it is believed that splitting cracks did not occur until well after the peak load had been reached.

From the test results, it is evident that two shear planes adjacent to each bolt participate in resisting the peak load in bolt tear-out failure, despite the subsequent occurrence of tensile splitting in some specimens. In addition, the ductility of the material behind the end bolt hole, as illustrated in Figure 4c, is sufficient to allow the shear stress in the two shear planes to be developed well beyond the yield stress—but not necessarily up to the ultimate stress—before the ultimate load is reached. The specimen shown in Figure 4c was unloaded right after reaching the peak load, although no tearing of any kind had yet initiated. This observation is consistent with the contention that the load-carrying mechanism at the ultimate strength consists of the two shear planes next to the bolt holes, and that tensile splitting adjacent to the center of the last hole is a post-peak phenomenon.

The predicted capacity for each test specimen, with the assumption that two shear planes at each bolt carry the peak load, was calculated using the CSA-S16-01 and AISC 360-05 block shear equations and the unified equation. The predicted capacities and the resulting test-to-predicted ratios are shown in Table 4. For comparison, values derived from the S16/AISC block shear provisions but considering only the case that assumes gross section failure on the shear planes (Equation 3), as well as those from the AISC bearing provisions, are also provided in Table 4.

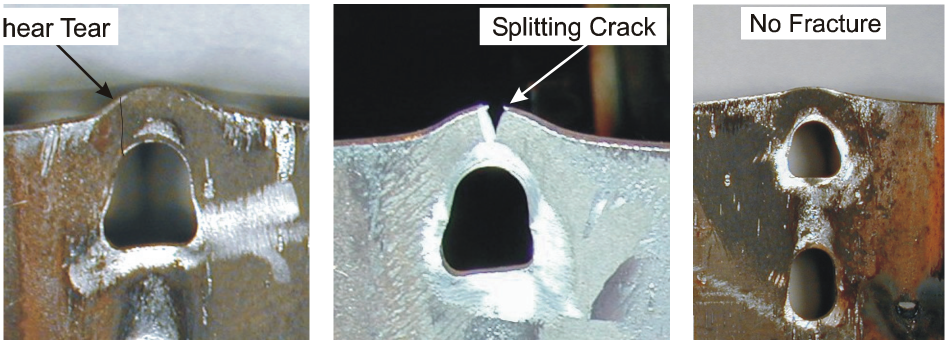
The equations in CSA-S16-01/AISC 360-05 give a mean test-to-predicted ratio and coefficient of variation of 1.46 and 0.10, respectively, while the unified equation results in corresponding values of 1.08 and 0.09. A mean test to-predicted ratio much closer to 1.0, combined with a slightly lower coefficient of variation, indicates that the unified equation better represents the behavior of these connections than does the set of two block shear equations from the North American design specifications. By considering only the S16/AISC equation that assumes gross section failure on the shear planes, the test to-predicted ratio is improved from 1.46 to 1.18, but the coefficient of variation is increased slightly. It should be noted that this modified block shear approach gives similar results to the AISC bearing treatment for bolt tear-out failure. The unified equation also provides somewhat improved results over the AISC bearing provisions, which give a mean test-to-predicted ratio of 1.17, with a coefficient of variation of 0.10. The mean test-to-predicted ratio considering these new tests as well as all criterion A specimens from the literature for the unified equation is 1.00, with a coefficient of variation of 0.11, as compared to 1.28 and 0.14, respectively, for the CSA-S16-01/AISC 360-05 equations.

RELIABILITY ANALYSES

In general, an appropriate reliability index, β, which represents the probability of failure of a member or connection, can be achieved by selecting a suitable resistance factor, ϕ, for design. These two parameters are related by the bias coefficient and the coefficient of variation of resistance, which can be determined by the relevant material, geometric, professional and discretization (in this case related to the need for an integer number of bolts) parameters. A summary of the relevant reliability parameters is shown in Table 5. Details of the procedures used in the reliability analysis presented in this paper are outlined by Cai and Driver (2008).

A total of 130 test results have been collected from the literature and this research project, including those from tests on plates, channels, tees, and wide-flange shapes with various connection configurations and conventional yield strengths (not greater than 550 MPa; 80 ksi). The reliability study considers all 130 tests conforming to criterion B, although only the 56 that conform to criterion A meet the minimum end distance and pitch requirements in North American design specifications.

Tables 1 and 4 show the mean test-to-predicted (T/P) ratios and the coefficients of variation (COV) for different research projects using CSA-S16-01/AISC 360-05 and the unified equation. It is evident that the block shear equations in CSA-S16-01/AISC 360-05 generally give high test-to-predicted ratios, while the test-to-predicted ratios for the unified equation are much closer to 1.0. The coefficients of variation for the two methods are similar. The AISC bearing provisions seem to provide low test-to-predicted ratios for the test results taken from the literature, and high values for the tests conducted as part of this research. The reason for this is unclear.



(a) Shear tear *(b) Tensile splitting crack* *(c) Ductility at a hole*

Fig. 4. End material adjacent to bolt hole.

Table 4. Summary of Test-to-Predicted Ratios

Specimen	Predicted Capacity			Test-to-predicted Ratio		
	S16-01/ AISC 360-05 Block Shear (kN)	AISC 360-05 Bearing (kN)	Unified Equation (kN)	S16-01/ AISC 360-05 Block Shear*	AISC 360-05 Bearing	Unified Equation
A1G1	481.2/650.6	601.5	683.1	1.44/1.06	1.15	1.01
A2G1	492.9/661.4	616.1	694.4	1.47/1.09	1.17	1.04
A7G1	451.1/603.4	563.9	639.2	1.47/1.10	1.18	1.04
A8G2	441.2/595.5	551.5	630.9	1.41/1.04	1.13	0.99
A3R1	366.5/469.6	458.1	507.4	1.731.35	1.38	1.25
A4R2	599.3/772.0	749.1	834.0	1.52/1.18	1.22	1.09
A9R1	376.2/470.0	470.3	519.1	1.68/1.35	1.35	1.22
A10R2	628.9/787.2	786.1	869.3	1.22/0.97	0.97	0.88
A5E1	479.7/528.9	599.6	615.7	1.45/1.32	1.16	1.13
A6E2	623.9/629.8	779.9	733.2	1.24/1.23	0.99	1.06
A11E1	448.1/540.5	560.2	605.8	1.54/1.28	1.23	1.14
A12E2	592.2/651.1	740.2	729.8	1.34/1.22	1.07	1.09
Mean (COV)	—	—	—	1.46/1.18 (0.10/0.11)	1.17 (0.10)	1.08 (0.09)

*The second number considers Equation 3 only.

Table 5. Parameters for Reliability Analyses

Reliability Parameter	Plates	Shapes	
		Web Failure	Web and Flange Failure
ρ_M (block shear)	1.07	1.05	1.03
V_M (block shear)	0.054	0.063	0.063
ρ_M (bearing)	1.19	1.13	—
V_M (bearing)	0.034	0.044	—
ρ_G	1.04	1.02	0.98
V_G	0.025	0.038	0.042
ρ_d	1.04	1.04	1.04
V_d	0.033	0.033	0.033

Table 6 presents the reliability indices for the design equations considered, with the values associated with connections that would be permitted by the design specifications shown in blue. Widely accepted target values for the reliability index range from 4.0 to 4.5 for connections. The resistance factor specified in CSA-S16-01 for block shear failure is 0.9, resulting in reliability indices that vary from 3.2 to 5.3. In AISC 360-05, the resistance factor is 0.75 for block shear, resulting in reliability indices that vary from 4.3 to 6.6. For comparison, the AISC bearing provisions result in reliability indices that vary from 5.1 to 5.8 for the bolt tear-out case. The unified equation, with a resistance factor of 0.75, provides an appropriate level of safety, with reliability indices ranging from 4.2 to 4.7. The greatly improved consistency over the various connection types indicates that the unified equation provides a better representation of the bolt tear-out failure behavior than the current block shear equations and is less conservative than the bearing provi-

Table 6. Reliability Indices Provided by Design Equations

Section	Number of Tests	Reliability Index β[c]			
		S16 01 Block Shear $\phi = 0.90$	AISC 360-05 Block Shear $\phi = 0.75$	AISC 360-05 Bearing $\phi = 0.75$	Unified Equation $\phi = 0.75$
Bolt Tear-out					
Plates	30[a]	4.4	5.5	5.1	4.3
Plates	99[b]	4.3	5.3	5.0	4.0
Channels (Web Failure)	5[b]	4.9	6.3	6.0	4.3
W-Shapes (Web Failure)	12[a,b]	5.3	6.6	5.8	4.7
Alternate Block Shear					
Tees (Web and Flange)	14[a,b]	3.2	4.3	—	4.2

a. Criterion A
b. Criterion B
c. Blue indicates a reliability index associated with connections that would be permitted by the design specification.

sions. Moreover, even if the specimens that violate the North American minimum end distance and pitch requirements are included, the unified equation still gives acceptable levels of safety, as shown in Table 6.

In the specific case of tees failing along the alternate block shear path, the reliability index obtained for CSA-S16-01 is unacceptably low. Conversely, those for the 2005 AISC *Specification* and the unified equation are both considered adequate with no effective stress factor applied.

SUMMARY AND CONCLUSIONS

Twelve full-scale tests designed specifically to investigate bolt tear-out failure have been completed on wide-flange tension members. Along with tests conducted by other researchers, a total of 116 bolt tear-out test results were analyzed. In addition, 14 tees that failed along the so-called alternate block shear path were investigated. It was found that the block shear equations in CSA-S16-01/AISC 360-05 generally provide highly conservative capacity predictions for bolt tear-out. Improved results, similar to those arising from the use of the AISC bearing equation, are achieved using the S16/AISC block shear provisions if only the equation that considers gross section failure on the shear planes is used. With the resistance factor of 0.9, CSA-S16-01 provides inconsistent reliability indices, and an unacceptably low reliability index was revealed in the case of failure of tees along the alternate block shear path. With the resistance factor of 0.75, AISC 360-05 generally provides high and inconsistent reliability indices. On the other hand, with a resistance factor of 0.75, the unified equation achieves appropriate and consistent levels of safety for the atypical block shear paths considered.

The following conclusions can be drawn from the test results of this research project, along with those from the literature:

1. The unified equation gives more accurate connection strength predictions and consistent reliability indices compared to the design equations in North American specifications for block shear failure with atypical failure paths, and the unified equation is therefore recommended for all block shear failures, regardless of whether the failure paths are classical or atypical.
2. In spite of the occurrence of tensile splitting cracks at the end bolts of some specimens that failed by bolt tear-out, the laboratory tests and strength calculations indicate that two shear planes adjacent to each bolt line carry the load approximately until the peak shear stress implied by the unified equation is reached.
3. For the bolt tear-out failure mode, based on the accurate results obtained using the unified equation model, the average stress on the shear planes at failure appears to exceed the shear yield stress but does not reach the ultimate shear stress, as has been observed in several previous research projects for cases of classical block shear failure.

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REFERENCES

Aalberg, A. and Larsen, P.K. (2001), "Bearing Strength of Bolted Connections in High Strength Steel," *Proceedings*, 9th Nordic Steel Construction Conference, Helsinki, Finland, pp. 859–866.

Aalberg, A. and Larsen, P.K. (2002), "The Effect of Steel Strength and Ductility on Bearing Failure of Bolted Connections," *Proceedings*, 3rd European Conference Steel Structures, Coimbra, Portugal, pp. 869–878.

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

ASTM (2007), *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*, A370-07, ASTM International, West Conshohocken, PA.

Cai, Q. and Driver, R.G. (2008), "End Tear-out Failures of Bolted Tension Members," *Structural Engineering Report No. 278*, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, AB, Canada.

CISC (2006), *Handbook of Steel Construction*, Canadian Institute of Steel Construction, Willowdale, ON, Canada.

CSA (2001), *Limit States Design of Steel Structures*, CSA-S16-01, 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, pp. 210–222.

Epstein, H.I. and Stamberg, H. (2002), "Block Shear and Net Section Capacities of Structural Tees in Tension: Test Results and Code Implications," *Engineering Journal*, American Institute of Steel Construction, Vol. 39, No. 4, pp. 228–239.

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, pp. 871–881.

Huns, B.B.S., Grondin, G.Y. and Driver, R.G. (2002), "Block Shear Behaviour of Bolted Gusset Plates," *Structural Engineering Report No. 248*, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, AB, Canada.

Kim, H.J. and Yura, J.A. (1999), "The Effect of Ultimate-to-Yield Ratio on the Bearing Strength of Bolted Connections," *Journal of Constructional Steel Research*, Vol. 49, No. 3, pp. 255–269.

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.

Puthli, R. and Fleischer, O. (2001), "Investigations on Bolted Connections for High Strength Steel Members," *Journal of Constructional Steel Research*, Vol. 57, No. 3, pp. 313–326.

Rex, C.O. and Easterling, W.S. (2003), "Behavior and Modeling of a Bolt Bearing on a Single Plate," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 129, No. 6, pp. 792–800.

Udagawa, K. and Yamada, T. (1998), "Failure Modes and Ultimate Tensile Strength of Steel Plates Jointed with High Strength Bolts," *Journal of Structural and Construction Engineering*, Architectural Institute of Japan, Vol. 505, pp. 115–122, in Japanese.

Udagawa, K. and Yamada, T. (2004), "Ultimate Strength and Failure Modes of Tension Channels Jointed with High Strength Bolts," *Proceedings*, 13th World Conference on Earthquake Engineering Conference, Vancouver, BC, Canada.