Block Shear Equations Revisited…Again

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Shortly after block shear was first identified as a possible failure mode for coped beam connections, design equations to account for it were incorporated into allowable stress design (ASD) provisions. These equations never changed, partly due to ASD not being updated since 1989. However, load and resistance factor design (LRFD) treatment of block shear changed with each new Specification. Over the years, it was suggested that the effect of eccentricity was missing from block shear equations. On the surface it appears that the effect of eccentricity on the block shear strength of connections, as suggested by previous investigators, has now been incorporated into the latest unified Specification. For many connections, however, nothing has changed. It is the conclusion of this paper that additional important cases need to be shown in Commentary Figure C-J4.2 of the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005), hereafter referred to as the AISC *Specification,* for which block shear equations now incorporate a new factor to account for connection eccentricity. In particular, as a minimum, angles connected by only one leg or tees connected by their flanges should also be included with other connections for which block shear capacities are now reduced.

BACKGROUND: CODE TREATMENT UNTIL 2005

In 1978, destructive tests on coped beams with bolted web connections were performed with some exhibiting what has become known as block shear as the failure mode (Birkemoe and Gilmor, 1978). They proposed a design equation in which the ultimate shear strength is applied to the net shear area and ultimate tensile strength to the net tension area. Block shear occurs when the web, for this case, develops its ultimate strength along the perimeter bolt holes and a "block" of this web begins to fracture. Figure 1a shows this block shear path for a coped beam. The equation that Birkemoe and Gilmor proposed was:

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$$P_{bs} = \left(0.3A_{nv} + 0.5A_{nt}\right)F_u \tag{1}$$

where

F_u = ultimate strength of the material
A_{nv} = net shear area
A_{nt} = net tensile area

Over time, this concept has become more broadly applied to many other connection applications. This equation (with the symbols A_v and A_t used for net shear and tension areas, respectively) was first incorporated into allowable stress design (ASD) in the 1978 AISC *Specification* (AISC, 1978). The 1989 AISC Specification (AISC, 1989), the last revision of ASD, included the same provisions for block shear.

The block shear equations in the load and resistance factor design (LRFD) specifications have changed with each edition. Both the first edition (AISC, 1986a) and the second edition (AISC, 1993) contain two equations for the determination of the block shear rupture design strength, ϕR_n, given by

$$\phi R_n = \phi[0.6F_y A_{gv} + F_u A_{nt}] \tag{2a}$$

and

$$\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}] \tag{2b}$$

where, in addition to the symbols in Equation 1,

ϕ = 0.75
F_y = specified minimum yield strength
A_{gv} = gross area subject to shear
A_{gt} = gross area subject to tension

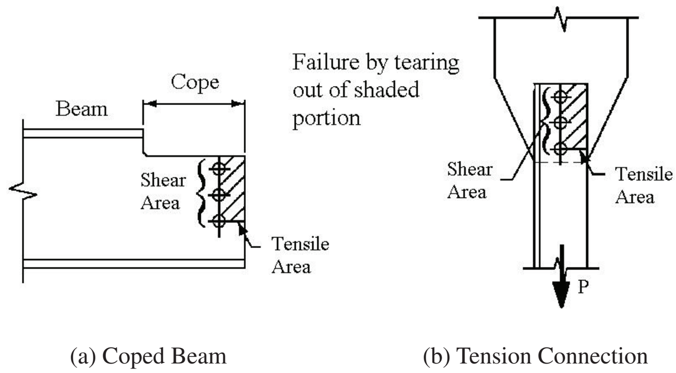

Fig. 1. Block shear failure paths (AISC, 2005).

Equation 2a represents block shear strength determined by rupture on the net tensile section combined with shear yielding on the gross section on the shear plane(s). Equation 2b represents block shear strength determined by rupture on the net shear area(s) combined with yielding on the gross tensile area. These equations are based on the work of Ricles and Yura (1983) as well as that of Hardash and Bjorhovde (1985). Except for slight differences in notation, these equations did not change from the first to the second edition of the AISC LRFD *Specification*, but, for some connections, which equation governed did change.

In the first edition of the LRFD *Specification* (AISC, 1986a), these equations were found only in Chapter J of the Commentary where it stated that "the controlling equation is one that produces the *larger* force." The Commentary went on to explain that since block shear is a fracture or tearing phenomenon and not a yielding limit state, the proper formula is the one in which the fracture term is larger than the yield term. For ductile steels where F_u is considerably larger than F_y, this may be true for both equations. For steels having F_y as an appreciable portion of F_u, this may not be true for either equation. The Commentary went on to state that "where it is not obvious which failure plane fractures, it is easier just to use the larger of the two formulas." In fact, the tables in the first edition of the LRFD Manual (AISC, 1986b) stated that the equation to be used is the one producing the larger block shear strength.

The block shear equations in the second edition of the AISC LRFD *Specification* (AISC, 1993) were found in Chapter J of the Specification, as opposed to the Commentary. While the formulas were the same, the change in the second edition was contained in a check of the relative fracture strengths, F_uA_{nt}, as compared to $0.6F_uA_{nv}$, or since F_u is common to both terms, A_{nt} compared to $0.6A_{nv}$. The Specification then stated that when $F_uA_{nt} \geq 0.6F_uA_{nv}$, use Equation 2a and when $F_uA_{nt} < 0.6F_uA_{nv}$, use Equation 2b. The Commentary stated that "the proper equation to use is the one with the larger rupture term."

In both the first and second LRFD editions, the Commentary gave two extreme examples showing which of the two limiting states (shear yield/tension fracture or shear fracture/tension yield) was appropriate. One of the examples had a tension area much larger than the shear area while the other example reversed these areas. The later interpretation of using the limiting state with the larger rupture term was certainly justified on the basis of these examples. The same could not easily be said of the earlier treatment.

The first edition LRFD *Specification* specified to always use the larger strength found from Equation 2a or 2b, while the second edition either led the designer to the same equation or possibly to the equation that yields the smaller design strength. The effect was that, for some connection geometries, the second edition produced more conservative results (Epstein, 1996a).

The third edition of the AISC LRFD *Specification* (AISC, 1999) used the following equations for the determination of the block shear rupture design strength, ϕR_n:

$$\phi R_n = \phi[0.6F_yA_{gv} + F_uA_{nt}] \leq \phi[0.6F_uA_{nv} + F_uA_{nt}]$$

when $$F_uA_{nt} \geq 0.6F_uA_{nv} \tag{3a}$$

and

$$\phi R_n = \phi[0.6F_uA_{nv} + F_yA_{gt}] \leq \phi[0.6F_uA_{nv} + F_uA_{nt}]$$

when $$0.6F_uA_{nv} > F_uA_{nt} \tag{3b}$$

In Equation 3a, the term $0.6F_yA_{gv}$ shall not be taken greater than $0.6F_uA_{nv}$, and in Equation 3b, the term F_yA_{gt} shall not be taken greater than F_uA_{nt}. These two provisions didn't exist in the second edition LRFD. Equation 3a is the equivalent of using yielding of the shear plane and rupture of the tension plane. The additional provision, however, requires that the yield strength of the gross shear area be less than the ultimate strength of the net shear area. Conversely, Equation 3b implies rupture of the shear plane and yielding of the tension plane. Similar to Equation 3a, however, the third edition limited the yield strength of the gross tension area to less than or equal to the ultimate strength of the net tension area.

BACKGROUND: COPED BEAMS VERSUS ANGLE TENSION MEMBERS

"The block shear failure mode is not limited to the coped ends of beams." This statement first appeared in the first edition of the LRFD *Specification* (AISC, 1986a) and then, subsequently, in the 1989 ASD *Specification* (AISC, 1989). The examples shown in the Commentary included tension connections for angles as well as gusset plates (see Figure 1b). Prior to their inclusion, some argued that structural engineers should have recognized block shear as a possible failure mode for angles, despite the fact that many textbook examples did not consider block shear for angles in tension, even when it clearly was the governing failure mode.

It was some angles found in the wreckage of the Hartford Civic Center roof, which had failed in block shear, that served as the impetus to investigate block shear in angles. Initial finite element investigations (Epstein and Thacker, 1991) were able to accurately determine block shear as the failure mode for these Civic Center angles. That study also investigated various similar geometries. Most importantly, the study also showed that there probably should be a substantial difference in the way in which block shear is treated for coped beams (where the load is applied to the connection in the plane of the web, which is also the block shear path) versus angles (where the load is applied eccentric to the failure plane). The first edition LRFD *Specification* (AISC, 1986a) included block shear equations in the Commentary (pages 186–188) where angles were shown.

Extensive tests of angles to determine effects of connection geometries and eccentricities were conducted at the University of Connecticut (Adidam, 1990; Epstein, 1992). The tested specimens were back-to-back angles with two rows of bolts in one leg of each angle. Among the specimens, the connection geometry was varied by using staggered and unstaggered bolt patterns, and alternating the position of the lead bolt in the staggered bolt patterns from the inside gage line to the outside gage line. It was concluded that the AISC ASD and LRFD *Specifications* provided inadequate safety factors for the block shear failure of angles. To alleviate this problem, it was suggested that the shear lag factor, U, be included in the tension terms of the code equations for block shear capacity.

The U factor, in essence the efficiency of the connection, is used to reduce the net area of angles in tension because not all of the tension area is effective in carrying the load. As the length of the outstanding leg increases, the eccentricity increases, and U decreases. The study of angles showed that as the outstanding leg increased in size, the capacity of the angle in net tension and the block shear capacity was reduced. These conclusions were also verified using nonlinear finite element models (Epstein and Chamarajanagar, 1996). The results justified applying the correction factor, U, to the block shear design equations. Another study investigated the efficiency of angles in tension rolled from high strength steel (Gross, 1994). Gross contended that the block shear equations for ASD and LRFD were not sufficient for high strength steel.

While the LRFD equations for block shear changed with each new edition, the changes did little to address the underlying problem of the eccentricity of the load to the plane in which block shear occurs. Over the years, several researchers have suggested modifications of the block shear equations to account for the reduction in capacity with increasing eccentricity. As was suggested previously (Adidam, 1990; Epstein, 1992), the simple empirical incorporation of the "shear lag" factor, U, into the tension path for block shear is probably all that is needed. For ASD, the resulting equation would be

$$P_{bs} = \left(0.3A_{nv} + 0.5UA_{nt}\right)F_u \tag{4}$$

where

$$U = 1 - \bar{x}/l \tag{5}$$

where

$\bar{x}$ = connection eccentricity

l = connection length

For LRFD, U could similarly be a factor for each tension area. Even for the simple case of pure tension fracture, not block shear, of a member with eccentricity, such as an angle, U is used to calculate an *effective* net area. Another way of treating the behavior is to think of it as combined tension and bending. Using AISC interaction equations, an equivalent reduction factor can be found (Epstein and D'Aiuto, 2002). Block shear is usually initiated with tension fracture. At the point where this occurs, the tension stresses, resulting from the load eccentricity, are more than the average load divided by the net area. Incorporating U into the block shear equations produces far more satisfactory results, not only for angles, but for flange connected tee sections as well (Epstein and Stamberg, 2002).

Before proceeding further, it should be pointed out that there has been disagreement with the idea that the block shear equations need modification to account for eccentricity. Grondin (2005) wrote that, "Although angles in tension represent a different case since both in-plane and out-of-plane eccentricities are present, the writer believes that these eccentricities are sufficiently small and the ductility of steel sufficiently large to minimize the effect of these eccentricities on the block shear capacity of angles in tension." Another study (Kulak and Grondin, 2001) concluded that, "Further research is required to investigate the effect of out-of-plane eccentricity on block shear failure." However, no change was recommended in the approach for angles, in part, because 12 of the 15 block shear failures previously documented (Epstein, 1992) involved connections with staggered gage lines. Stagger, however, was shown not to have an appreciable effect on capacity.

BACKGROUND: TEES AND OTHER SECTIONS

A limited number of tests on tee sections in tension produced surprising results (Epstein, 1996b). The tees were connected through the flange and the stem was the outstanding element. A previously undocumented *alternate* failure path in block shear was discovered during the testing (see Figure 2). This led to a full investigation of structural tees in tension. Finite element analyses were then performed, and they supported this new block shear failure mode of structural tees in tension (Epstein and McGinnis, 2000). The results compared very well with previous tests in replicating the newly discovered alternate block shear failure path. It was found that the moments caused by the eccentricity were not equal to the axial load multiplied by the eccentricity. The reactions at the first and last bolts along a line of bolts created an opposing moment, which reduced the moment caused by the eccentricity. Again, it was concluded that the shear lag factor, U, should be incorporated in the tension terms of the block shear equations.

A series of tests on 50 structural tees in tension were performed (Epstein and Stamberg, 2002). The specimens were bolted through the flange, with the stem outstanding, and had varying eccentricities and connection lengths. The geometries of the specimens were chosen such that a progression from net section failure to block shear failure would occur. The tests exhibited behavior that matched what was indicated by finite element models. As connection lengths decreased or eccentricities increased, or both, the efficiencies of the connections decreased. The previously obtained theoretical results (Epstein and D'Aiuto, 2002) for these tests were compared to the values calculated using the then current (second edition) *Specification*. Even though the theory was originally based on block shear in tees, it was found to agree with the net tension failure results as well. In addition to incorporating a reduction factor in block shear equations, it was also suggested that, as a simplified design approach, a reduced lower bound for the shear lag factor, U, may be appropriate for net section failures.

THE 2005 SPECIFICATION TREATMENT

The shear lag coefficients have been significantly reduced in the 2005 AISC *Specification for Structural Steel Buildings*, which combines ASD and LRFD into one document (AISC, 2005). Block shear design capacity is given by the basic equation for tension rupture-shear yield as

$$\phi R_n = \phi[0.6F_y A_{gv} + U_{bs} F_u A_{nt}] \tag{6}$$

In Equation 6, the term $0.6F_yA_{gv}$ shall not be taken greater than $0.6F_uA_{nv}$, which represents the rupture-rupture limit state. This, in essence, is Equation 3a with the exception of U_{bs}. U_{bs} is a new term that was added to the block shear design equation to account for the effect of eccentricity on the block shear capacity of a connection. This equation is certainly similar to what was proposed previously (Epstein, 1992). Note that shear rupture-tension yield is no longer considered. Many past research studies noted that block shear failures usually initiated with a tension fracture.

The definition and limits for the variable, U_{bs}, went through revisions during the development of the 2005 *Specification*. In an earlier treatment, the following equation was used to calculate U_{bs}:

$$U_{bs} = 1, \text{ if } e/l \leq 1/3 \tag{7a}$$

or

$$U_{bs} = 1 - e/l, \text{ if } e/l > 1/3 \tag{7b}$$

where

e = eccentricity of the force tending to cause block shear rupture in the plane of the connection faying surface

l = length of the block subject to block shear rupture

The variable e is equivalent to $\bar{x}$ in Equation 5. Note that, however, l is not the same as in the Equation 5. The difference is that the term l in Equations 7a and 7b is equal to the length of the block subject to block shear and in the equation for U it is equal to the length of the connection. Therefore, for the same connection, U_{bs} will be larger than U because the length of the block subject to block shear will be greater than the length of the connection for standard connections.

In a subsequent treatment (which became the one adopted in the 2005 *Specification*) it was stated that U_{bs} is equal to 1.0 when the tension stress is uniform, and equal to 0.5 when the tension stress is nonuniform. This U_{bs} does not vary according to any parameter of a connection, such as connection length or eccentricity. This definition certainly simplifies calculations.

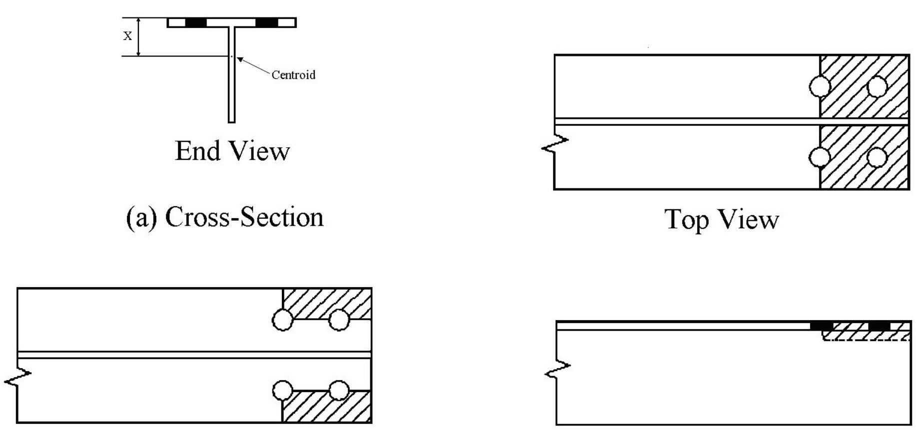

Fig. 2. Block shear failure path for tee sections.

THE EFFECT OF THE BLOCK SHEAR REVISIONS ON DESIGN CAPACITIES

In this section, the data from previous research on angles at the University of Connecticut (Epstein, 1992) will be used to examine the effect of the latest revision on block shear capacities (Equation 6). The majority (35) of the 38 geometries tested in the 1992 study showed block shear failures and only those are used herein for the comparisons. Since, with the 2005 *Specification*, ASD and LRFD treatments have coalesced, only LRFD will be presented in the following (Aleksiewicz, 2004).

Table 1 shows the material properties and geometries of the single angle specimens that were tested in the 1992 investigation (Epstein, 1992). In this table, specimen numbers are the same as used in 1992 and material properties of yield and ultimate were as measured. The bolt patterns refer to the number of bolts in the two gage lines and whether or not they were staggered (+ or – denotes stagger and which gage line had the lead bolt). For more specific details of the specimens, the reader is referred to the original paper (Epstein, 1992).

Also presented in Table 1 are the capacities, as calculated from the third edition of the LRFD *Specification* (LRFD, 1999), as well as those from the latest *Specification* (AISC, 2005). Equation 6, from the 2005 *Specification*, uses U_{bs} either equal to 1.0 (for uniform tensile stresses) or 0.5 (for nonuniform tensile stresses). Therefore, U_{bs} is determined by the type of connection, not the specific parameters of a connection such as connection length (l) or the amount of eccentricity (e or $\bar{x}$). Tension members not connected through all of their elements develop nonuniform tensile stresses due to shear lag. Also, eccentricity causes bending which creates nonuniform tensile stresses. Any connection that has either of these characteristics (angles have both) should, therefore, use the value of 0.5 for U_{bs}. The capacities for the 2005 *Specification* in Table 1 include 0.5 for U_{bs}.

Figure 3 shows the relationship between PF (professional factor, which is the ratio of the failure load determined by destructive testing divided by the corresponding design capacity based on measured material and geometric properties) versus $1 - e/l$ for the 1999 and 2005 *Specifications* for the data in Table 1. Least square "trend lines" (straight line fits of the data) are used for comparison. The 1999 data is similar to that of previous *Specifications* in that many Professional Factors are below 1.0 and as the eccentricity increased or the connection length decreased, the trend for Professional Factors decreased.

If the earlier treatment for U_{bs} in Equation 7 were used, there would be no difference in results from 1999 except for specimens #1, 3, 5, and 6, shown in Table 1, because only these had values of $1 - e/l < 0.67$. In Figure 3, because the factor $U_{bs} = 0.5$ has been applied to all block shear tension areas for the 2005 data, clearly all professional factors (PF) shown are increased when compared to the 1999 data. The trend line for 2005 clearly shows a marked improvement.

So, it appeared that the approach settled upon for block shear had been satisfactorily addressed. However, in the 2005 *Specification* Commentary Section J4.3 on block shear (see Figure 4), it becomes readily evident that there is no change for angles because "angle ends" are shown in the category of $U_{bs} = 1.0$ (Figure 4a).

It is not apparent if there is a change for tees, which also, as previously noted, have capacities (net tension and block

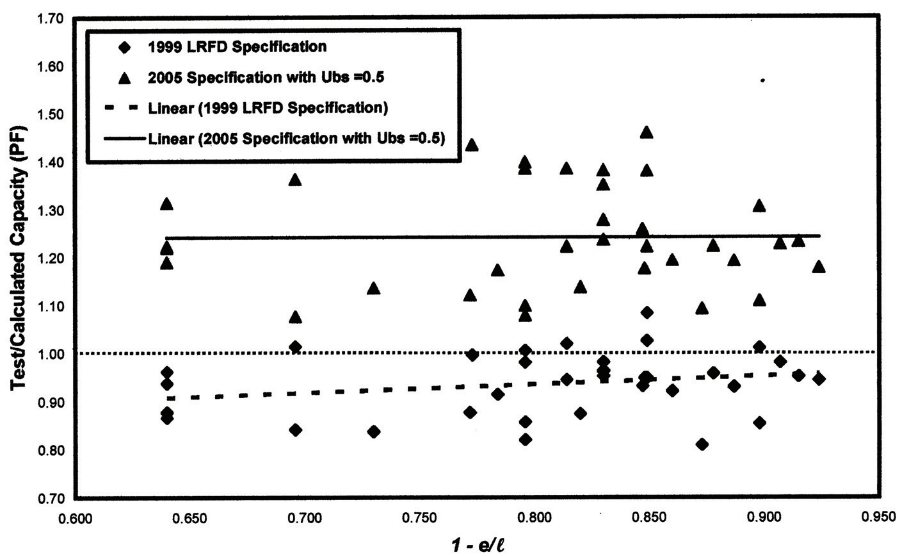

Fig. 3. 1998 LRFD vs. 2005 (with U_{bs} *= 0.5) block shear treatment for tested angles.*

Table 1. Geometry, Material Properties and Block Shear Capacities										
#	Member	Bolt Pattern	F_y ksi	F_u ksi	$1 - e/l$	P_{test} kips	1999 LRFD		2005 LRFD	
							kips	PF	kips	PF
1	6x6x5⁄16	2/2+	51.9	73.9	0.640	182.5	157.9	0.87	111.8	1.22
2	6x6x5⁄16	2/2-	51.4	77.0	0.730	204.2	182.8	0.84	134.8	1.14
3	6x6x5⁄16	2/2	51.0	75.5	0.640	188.7	150.8	0.94	107.7	1.31
4	6x6x5⁄16	2/3-	53.0	77.2	0.820	242.7	208.1	0.87	160.0	1.14
5	6x6x5⁄16	3/2+	49.3	73.6	0.640	204.9	175.1	0.88	129.1	1.19
6	6x6x5⁄16	2/3	51.4	75.0	0.640	259.7	202.4	0.96	159.6	1.22
7	6x6x5⁄16	3/3	51.6	74.8	0.784	237.1	194.3	0.92	151.6	1.17
9	6x4x5⁄16	2/2+	51.0	72.4	0.796	202.7	154.9	0.98	109.7	1.39
10	6x4x5⁄16	2/2-	46.8	68.2	0.847	203.9	164.1	0.93	121.5	1.26
11	6x4x5⁄16	2/2	50.3	71.0	0.796	194.2	144.8	1.01	104.2	1.40
12	6x4x5⁄16	2/3-	55.5	80.0	0.898	247.1	216.9	0.85	167.0	1.11
13	6x4x5⁄16	3/2+	50.5	70.2	0.796	189.1	172.8	0.82	129.0	1.10
14	6x4x5⁄16	2/3	49.4	68.9	0.796	219.8	192.2	0.86	152.8	1.08
15	6x4x5⁄16	3/3	46.5	64.9	0.878	218.6	171.1	0.96	134.0	1.22
17	6x3½x5⁄16	2/2+	48.3	74.5	0.830	198.2	154.1	0.96	107.6	1.38
18	6x3½x5⁄16	2/2-	52.5	76.6	0.873	198.8	184.2	0.81	136.4	1.09
19	6x3½x5⁄16	2/2	52.1	78.2	0.830	199.3	155.3	0.96	110.6	1.35
20	6x3½x5⁄16	2/3-	50.3	68.5	0.915	238.5	187.8	0.95	145.1	1.23
21	6x3½x5⁄16	3/2+	49.5	69.4	0.830	216.1	170.1	0.95	126.8	1.28
22	6x3½x5⁄16	2/3	48.0	69.1	0.830	250.6	191.4	0.98	151.9	1.24
23	6x3½x5⁄16	3/3	45.6	69.3	0.898	236.5	175.4	1.01	135.8	1.31
25	5x5x5⁄16	2/2+	44.3	62.0	0.696	154.1	114.0	1.01	84.8	1.36
26	5x5x5⁄16	2/2-	44.6	61.5	0.772	155.8	133.2	0.88	104.2	1.12
27	5x5x5⁄16	2/3-	45.1	63.2	0.848	194.9	153.9	0.95	124.2	1.18
28	5x5x5⁄16	3/2+	50.4	70.1	0.696	169.6	151.1	0.84	118.1	1.08
29	5x3½x5⁄16	2/2+	47.9	71.6	0.814	174.1	128.0	1.02	94.3	1.38
30	5x3½x5⁄16	2/2-	45.0	67.8	0.860	171.8	139.8	0.92	107.9	1.19
31	5x3½x5⁄16	2/3-	45.2	68.2	0.907	208.8	159.6	0.98	127.5	1.23
32	5x3½x5⁄16	3/2+	48.8	72.6	0.814	189.9	150.7	0.94	116.5	1.22
33	5x3x5⁄16	2/2+	42.5	59.4	0.849	149.4	109.2	1.03	81.2	1.38
34	5x3x5⁄16	2/2-	43.1	61.0	0.887	161.5	130.2	0.93	101.5	1.19
35	5x3x5⁄16	2/3-	42.5	62.6	0.924	187.2	148.6	0.94	119.1	1.18
36	5x3x5⁄16	3/2+	42.2	61.1	0.849	163.0	128.7	0.95	100.0	1.22
37	5x3x5⁄16	1/2-	46.1	65.4	0.849	173.3	119.9	1.08	89.1	1.46
38	5x3x5⁄16	2/1+	44.1	61.8	0.773	126.8	95.4	1.00	66.3	1.43

shear) that are significantly reduced when only the flange is connected. Further, since W shapes connected only by the flanges are usually treated as half tees, one would assume that their block shear strength would also be compromised. It is not clear what value of U_{bs} should be used for this case.

A User Note in Section J4.3 of the 2005 *Specification* states that, "The cases where U_{bs} must be taken equal to 0.5 are illustrated in the Commentary"(Figure 4 herein). So, are only coped beam connections having significant in-plane eccentricities affected by the new treatment for block shear?

There is a recent study of the treatment of block shear equations that are presented in various design standards (Driver, Grondin and Kulak, 2006). In this study, several standards were compared for many tests of gusset plates, angles, tees and coped beams. It was concluded that, except for gusset plates, there should be a reduction factor (comparable to U_{bs}) applied to the tension term of the block shear equation. (It was also proposed to modify the shear term with a combination of yield and ultimate stresses).

In particular, for coped beams, Driver et al. (2006) proposed a reduction factor of 0.9 for the tension term in coped beams where there is one row of bolts (as shown in Figure 4a wherein U_{bs} is 1.0). For coped beams with two rows of bolts (as shown in Figure 4b wherein U_{bs} is 0.5), they proposed 0.3. In reality, the factor used for the single row coped beam connection is not that critical since the tension term is only a fraction of the shear term. For a two bolt connection (in one row) on a coped beam, the tension term is as large a percentage of the total as possible. But even then, the reduction in capacity would only be approximately 15% if U_{bs} = 0.5 instead of the 1.0 factor given in the 2005 AISC *Specification*. For longer connections (three or more bolts), this decrease is much less.

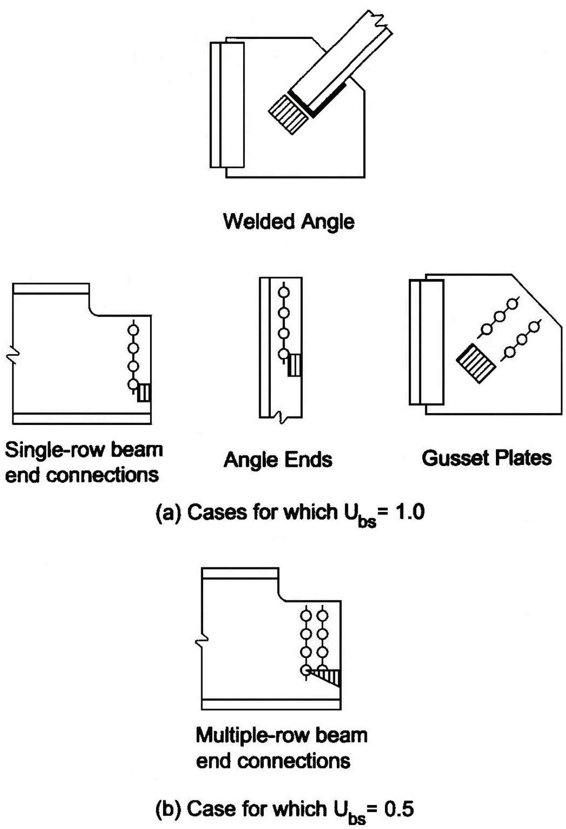

Fig. 4. Values of U_{bs} *as shown in Figure C-J4.2 of the 2005 AISC Specification Commentary (AISC, 2005).*

What about angles (and tees)? Even for one row of bolts in an angle or the stem of a tee, it was reported (Orbison, Wagner and Grondin, 1999) that the smaller the edge distance, the smaller the professional factor. Their tests were for fairly long connections (four bolts) in only one row. Therefore, the tension terms for these tests are a small percentage of the shear term. Even so, reduced edge distance reduced the professional factors, somewhat. The reason for this is fairly obvious. It is not that the stresses along the tension path are nonuniform; it is that their magnitude is increased because of the greater eccentricity of the load and, therefore, the increased effect of the resulting moment on the tension stresses. With only one row of bolts, however, similar to coped beams, it can be argued that using U_{bs} = 1.0 (as shown in Figure 4a) is satisfactory since, again, the tension term is a small fraction of the total.

But what happens with two rows of bolts in one leg of an angle? Note that all of the tests shown in Figure 3 had two rows of bolts. Further note that most of these tests were not included in the study of Driver et al. (2006) because there was stagger between the gage lines. Clearly, the tension term for these tests represents a larger percentage of the block shear capacity than when there is only a single row of bolts. So, the specifying of U_{bs} as significantly less than 1.0 appears to be appropriate.

As far as tees are concerned, Driver et al. show a reduction factor of 0.9. However, it does not appear that this was meant to apply to both situations of web-(referred to as "stem" by Driver et al.) connected and flange-connected tees because they do not include the latter in their list of investigations. For stem-connected tees, there is in-plane eccentricity, but for flange-connected tees there is out-of-plane eccentricity and the block shear failure for these usually extends into the web. For the flange-connected case, based on many tests (Epstein and Stamberg, 2002), it does not appear that the proposed reduction is sufficient. For tees that fail in block shear along the alternate path, the tension term is a significant fraction of the total block shear. Driver et al. did not report on these block shear failures when presenting their "unified" block shear equation.

CONCLUSIONS

Block shear treatment by AISC has undergone several revisions since 1978, where it first appeared, through the latest, 2005 equations. In the opinion of the authors, either the definition of U_{bs} must change or the figures in the Commentary need to include more cases where U_{bs} is not equal to 1.0. As a minimum, two gage lines in one leg of an angle as well as tees connected by their flanges and, probably, W sections connected by their flanges should be added to the cases in Figure 4b. The statement in the 2005 *Specification* that, "Where the tension stress is uniform, U_{bs} = 1.0; Where the tension stress is nonuniform, U_{bs} = 0.5 in., in conjunction with the limited cases shown in the Commentary, is not particularly helpful in pointing to cases where the use of U_{bs} = 1.0 may not be conservative. It is not that the tension stresses need to be nonuniform, but the magnitude of the stresses, as influenced by the eccentricity of the load, is what reduces the efficiency of the connection.

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