An Experimental Study of Block Shear Failure of Angles in Tension

HOWARD I. EPSTEIN

INTRODUCTION

Structural tension members are designed to resist yielding of the gross section or rupture of the minimum net section, taking into account the effects of stagger and shear lag. The shear lag effect must be considered when all the components of a tension member are not transmitting the load to the connection. For short connections, an angle connected by one leg may fail in a combination of tension perpendicular to and shear parallel to the loaded axis. This type of failure has been termed block shear when investigated for beam web connections.[1,2,3]

Most of the international codes for steel design have not considered this type of failure.[4] The current AISC ASD[5] and LRFD[6] specifications do incorporate formulae, in their respective commentaries, to calculate block shear failure capacities. The current ASD Specification is based on the work of Birkemoe and Gilmor[1] and is given by

$$P = 0.3A_{nv}F_u + 0.5A_{nt}F_u \tag{1}$$

where F_u is the ultimate strength, and A_{nv} and A_{nt} are the net shear and tensile areas, respectively.

In 1985, Hardash and Bjorhovde,[7] reported on tests conducted on gusset plates in tension and suggested a different approach to calculate block shear strength. They recommended that the yield strength on the gross section on one plane be added to the fracture strength of the net section on the perpendicular plane. The first edition of the LRFD Specification[6] uses this approach to calculate nominal block shear strength. The two equations given are

$$P_n = 0.6F_yA_{vg} + F_uA_{nt} \tag{2}$$

$$P_n = 0.6F_uA_{ns} + F_yA_{tg} \tag{3}$$

where F_y is the yield strength, A_{vg} and A_{tg} are the gross shear and tension areas, respectively, and A_{ns} and A_{nt} are the net shear and tension areas, respectively. As explained in the original paper and in the commentary, the larger of Eqs. 2 or 3 is to be used as the nominal block shear strength. The LRFD resistance factor ϕ to be used in conjunction with these equations is given as 0.75.

Howard I. Epstein is professor of civil engineering at the University of Connecticut, Storrs, CT.

The commentaries for ASD as well as LRFD indicate that block shear failure is not limited to the coped web of a beam. Both clearly illustrate other possibilities including gusset plates and angles when used in tension and connected by only one leg. Finite element studies[8,9] have indicated that the state of stress for block shear in such angles is significantly different from that in the beam webs, the tests on which the current code equations are based. These angles are investigated in the present study.

Since the size of the outstanding leg appears to effect the eccentricity of the loading, and since the block shear equations in the codes do not include this factor, it was decided to make the outstanding leg one of the parameters to be studied in the experimental program. Other factors of interest included the presence of stagger when two gage lines on the same leg are used. The code treatment for stagger in a block shear path is not exactly defined, but several recent manuals and textbooks[10,11,12] all agree that it seems reasonable to incorporate the long-standing $s^2/4g$ increase to net tensile width.

This paper reports on the results of full-scale testing of double-row, staggered, and unstaggered bolted connections of structural steel angles. The effect of the variation of several parameters are presented. The current code provisions are found to be less conservative for block shear failure than net section tensile failure in angles and a revised treatment is suggested.

EXPERIMENTAL PROGRAM

It was considered desirable to test specimens that could actually be allowed in design. The availability of angle sizes was the first constraint placed upon the selection of connections to be tested. Next, for varying size of the outstanding leg, it was desired to maintain the same angle thickness, if possible, to eliminate thickness as a parameter in the study. Also, there was a requirement of having two gage lines on a leg so that stagger could be studied. It was desired not to have the bolts govern the strength of the connection. At the same time, in order for the connections to be short so that block shear was the governing failure mode, the number of bolts was limited. Finally, testing of a pair of angles was warranted to reduce any eccentricity on the 300 kip capacity testing machine available for the study.

With all the above constraints to be considered, the basic connections to be tested were pairs of angles, 5/16-in. thick, connected by two rows of 3/4-in. diameter bolts in two rows on a 6-in. leg. Angles 6×6×5/16 were first selected for testing various connection configurations by varying the number and possible stagger of the bolts. To investigate the effect of the outstanding leg, the same connections to the 6-in. leg were then used for 6×4×5/16 and 6×3½×5/16 angles.

The effect of staggering the bolts on the connected leg was one of the parameters to be investigated. This requires connections with two gage lines. For larger angles connected by only one leg, two gage lines help to reduce the length of the connection. This has the benefit of reducing the length of the connection, but at the same time has the drawback of emphasizing the shear lag reduction in capacity. When two gage lines are present, the fasteners are often staggered so that the net area is not further reduced. Stagger does increase the length of a connection, but it may be required for certain geometries. For instance, when 3/4-in. or larger diameter bolts are used to connect a 5-in. angle leg, AISC minimum spacing provisions mandate stagger when bolts are used on two gage lines. To further investigate stagger, it was decided to also test specimens having a 5-in. connected leg. To obtain some consistency with the 6-in. angles, 5/16-in. thick angles were also chosen for the 5-in. angles. Therefore, various connections for 5×5×5/16, 5×3½×5/16 and 5×3×5/16 angles were tested.

The connections tested had three to eight bolts in two gage lines, with and without stagger. The bolt configurations were chosen to cover the transition from shorter connections, governed by block shear, to longer connections, governed by net section failure. Since bolt shear was not desired, 3/4-in. A490-X bolts in standard holes were chosen (7/8-in. A325 bolts could also have been used).

Connection geometries for the 6-in. connected leg were chosen to have a minimum of four bolts, as fewer than four bolts would usually lead to bolt capacities governing the allowable load, and this was not desired. Three different four-bolt connections for 6-in. angles are shown in Fig. 1. The first (#1), where the included angle is obtuse at the tension/shear intersection in the block shear path, will be referred to as having positive stagger. Next are shown examples of

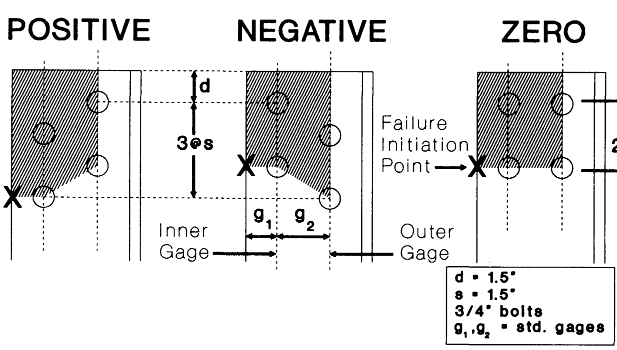

Fig. 1. Sign of stagger.

negative stagger (#2), and a zero stagger (or unstaggered) connection (#3). For all connections, the edge distance was taken to be 1.5 in. and the pitch was the usual 3 in. for this bolt size. Standard gages were used throughout. For a 5-in. leg, these gages are g_1 = 2.0 and g_2 = 1.75 in. For a 6-in. leg, these gages are 2.25 and 2.5 in., respectively.

There were three specimens, each consisting of a pair of angles, tested for each connection. In all, 38 different connections were tested for a total of 114 tests. In order to eliminate material variation for any particular connection, all were fabricated from the same 40 ft length of angle. This limited the length of each specimen (4.5 ft was chosen), but additional tests on longer specimens produced consistent stress distributions and failures. All the specimens were fabricated by The Berlin Steel Construction Company, Berlin, CT. Yield and ultimate strengths were found from coupon tests conducted by The New Haven Testing Laboratory, New Haven, CT.

All the connection geometries are listed in Table 1 along with their yield and ultimate strengths. A summary of all connection patterns is shown in Fig. 2. For reference, connections are specified by the number of bolts on the outer and inner gage lines and by the sign of the stagger. For instance, connections #1 to #3 are designated as $2/2,^{+}$ $2/2,^{-}$ and $2/2,^{0}$ respectively. Connections #4 to #6 are the five-bolt patterns $2/3,^{-}$ $3/2,^{+}$ and $2/3,^{0}$ respectively. Connections similar to #6

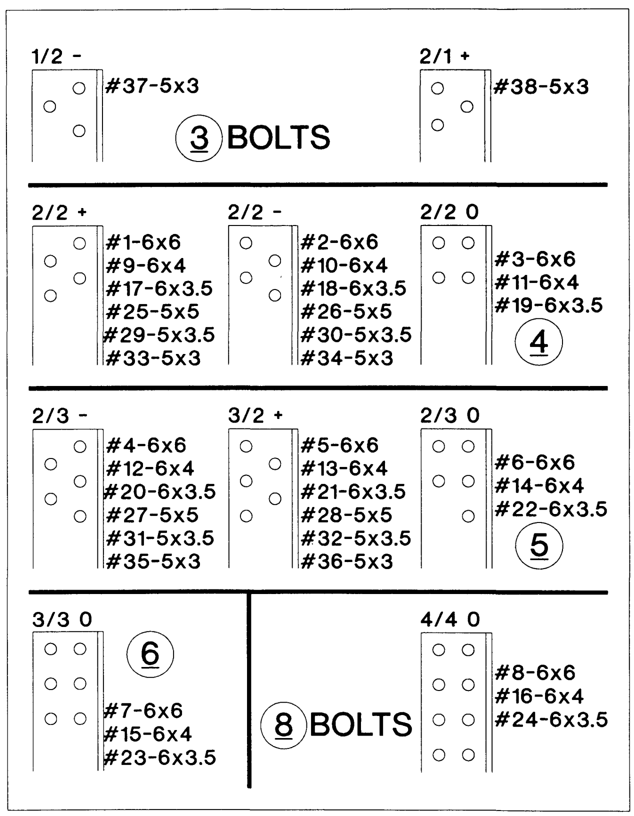

Fig. 2. Connections tested.

are often specified for an odd number of bolts. The thinking behind this is that the net section, which resists the full load, only has one hole deducted from the gross area. If all bolts are assumed to be resisting an equal load, only a fraction of the load (in this case 4/5) is taken by the cross section having two holes deducted from the gross area. The lead, sometimes called "poisoned," bolt has traditionally been placed on the inner gage line in order to minimize the loading eccentricity. Completing the 6×6 angles tested are connections #7 and #8 which are unstaggered six- and eight-bolt patterns, respectively. Since the test program was primarily designed to test block shear, other six- and eight-bolt geometries and all seven-bolt geometries were not investigated because their failures should be predominantly net section. The only missing five bolt pattern ($3/2^0$) was not investigated because it is not typically fabricated.

These same eight connection geometries are repeated for 6×4×5⁄16 angles in connections #17 to #24. No other angles having a 6-in. leg are available in a 5⁄16-in. thickness. Therefore, for this constant thickness, these 24 connections represent all the reasonable connection geometries. Thicker angles (3⁄8-in., for instance) were not chosen for a number of reasons including the diminished number of block shear failures and the limitation of a 300 kip capacity testing machine.

Fortunately, the 5⁄16-in. thick angles are also available in three different 5-in. connected legs (5×3½ and 5×3). Since unstaggered patterns are not permissible, the only four and five-bolt patterns tested were connections #25 to #28 for 5×5 angles. These four patterns are repeated for 5×3½ angles in connections #29 to #32 and for 5×3 angles in connections #33 to #36. In addition, two three-bolt patterns are possible (without bolt shear governing) and were tested as connections #37 and #38. These two patterns are not usually fabricated, but theoretically can be. Longer connections for the 5-in. angles are possible, but the already completed 6-in. connections had demonstrated the transition from block to net section failure.

TEST RESULTS

The last two columns of Table 1 show the average failure load of the three specimens for each connection tested as well as the type of failure. In general, the variation of the test loads fell within a few percent of the average. The maximum and minimum failure loads fell within 10 percent of the average value for 34 of the 38 connections tested. The failures were classified into five different types:

(A) block shear,
(B) predominantly block shear with some net section,
(C) predominantly net section with some block shear,
(D) net section, and
(E) bolt shear plus block shear.

Examples of different failure types are shown in Fig. 3a–3e.

As expected, the shorter connections failed in true block shear, while longer connections failed through the net section. Other results concerning the failures are:

- Failures for the shortest connections of equal leg angles (all #1 and #25 specimens) were all classic block shear failures.
- As the length of the connection increased, failures generally went from types (A) to (B) to (C) to (D), as described above.
- For the same bolt pattern, as the length of the outstanding leg decreased, failures increasingly became net section. For instance, the three specimens (six angles in all) comprising connection #4 produced predominantly block shear failures. Block shear and net section failure occurred equally for the six angles of connection #12. Net section failure predominated in connection #20.
- Failures for the longest connections of 6×6 angles (#8, #16, and #24) were all through the net section.
- Initial yielding was usually observed on the connected leg near the bolt closest to the center of the specimen on the outer gage line (as indicated in Fig. 1).
- For the five-bolt, unstaggered connection geometry, all the specimens (connections #6, #14, and #22) had the lead bolt shear while the remaining four-hole pattern failed in block shear (type E).
- Two of the three eight-bolt specimens for connection #8 ($4/4^0$) exceed the 300 kip capacity of the testing machine, but there was significant yielding in evidence at that load. Judging from the failures observed in other tests, these specimens had almost reached their failure loads.
- The time required to complete each test did not appear to be a factor. One of the three specimens was tested to failure in a few minutes, approximately the same time required for coupon tests. There were no significant differences obtained when these tests were compared to the specimens which required longer to accomplish.

ALLOWABLE OR DESIGN LOADS

For bolted tension connections, ASD allowable capacities or LRFD design strengths must consider several modes of failure. The tension member itself must be designed against yielding of the gross area and rupture of the effective net area. Where there is stagger, all possible failure paths must be considered. All possible block shear paths must also be investigated. This includes paths that require bolts to fail in addition to the angle failing along a block shear path.

All connections tested used ¾-in. bolts in double shear. Bearing areas and edge distances were such that they never governed allowable or design loads. The connections were considered to be bearing (as they most certainly were at failure) and the threads were excluded from the shear planes. The resulting X designation gave 35.3 kips allowable ASD load per bolt (ASD/J3.2) and 51.7 kips LRFD design strength per bold (LRFD/C-J3).

Table 1.
Connections Tested

Connection #	Angle Size (in.)	Connection Geometry	Yield Strength F_y (ksi)	Ultimate Strength F_u (ksi)	Average Failure Load (kips)	Failure Type
1	6×6×5⁄16	$2/2^+$	51.9	73.9	182.5	A
2		$2/2^-$	51.4	77.0	204.2	A-B
3		$2/2^\circ$	51.0	75.5	188.7	A-B
4		$2/3^-$	53.0	77.2	242.7	B
5		$3/2^+$	49.3	73.6	204.9	C
6		$2/3^\circ$	51.4	75.0	259.7	E
7		$3/3^\circ$	51.6	74.8	237.1	B-C
8		$4/4^\circ$	52.0	74.6	>297.7	D
9	6×4×5⁄16	$2/2^+$	51.0	72.4	202.7	A
10		$2/2^-$	46.8	68.2	203.9	B
11		$2/2^\circ$	50.3	71.0	194.2	A
12		$2/3^-$	55.5	80.0	247.1	B-C
13		$3/2^+$	50.5	70.2	189.1	C
14		$2/3^\circ$	49.4	68.9	219.8	E
15		$3/3^\circ$	46.5	64.9	218.6	C
16		$4/4^\circ$	48.1	65.7	243.5	D
17	6×3.5×5⁄16	$2/2^+$	48.3	74.5	198.2	B
18		$2/2^-$	52.5	76.6	198.8	B
19		$2/2^\circ$	52.1	78.2	199.3	B
20		$2/3^-$	50.3	68.5	238.5	C
21		$3/2^+$	49.5	69.4	216.1	B
22		$2/3^\circ$	48.0	69.1	250.6	E
23		$3/3^\circ$	45.6	69.3	236.5	C-D
24		$4/4^\circ$	46.8	69.7	255.2	D
25	5×5×5⁄16	$2/2^+$	44.3	62.0	154.1	A
26		$2/2^-$	44.6	61.5	155.8	B
27		$2/3^-$	45.1	63.2	194.9	B
28		$3/2^+$	50.4	70.1	169.6	C
29	5×3.5×5⁄16	$2/2^+$	47.9	71.6	174.1	B
30		$2/2^-$	45.0	67.8	171.8	B
31		$2/3^-$	45.2	68.2	208.8	C
32		$3/2^+$	48.8	72.6	189.9	B
33	5×3×5⁄16	$2/2^+$	42.5	59.4	149.4	B
34		$2/2^-$	43.1	61.0	161.5	B
35		$2/3^-$	42.5	62.6	187.2	B
36		$3/2^+$	42.2	61.1	163.0	C
37		$1/2^-$	46.1	65.4	173.3	A
38		$2/1^+$	44.1	61.8	126.8	B

The allowable ASD load for rupture of the net section is given by $0.5F_u UA_n$ (ASD/D1), and the LRFD design strength is 50 percent greater than the ASD allowable (LRFD/D1). Due to the spacing used, failure through one hole never governed for any connection when compared to the two-hole stagger path. The shear lag reduction coefficients U used in these calculations were obtained from the current AISC codes and not the $1 - x/L$ contained in the original research.[13] U was therefore set equal to 0.75 for all connections except those with the $3/3^0$ and $4/4^0$ patterns for which U was 0.85.

The allowable ASD load for yielding of the gross cross-section is given by $0.6F_y A_g$, where A_g is the gross area. The LRFD design strength is 50 percent greater than this ASD allowable. Neither yielding provision came close to governing the strength of any connection tested.

For each connection tested, the ASD allowable load P_A and the LRFD design load ϕP_n were calculated on the basis of the actual yield and ultimate strengths given in Table 1. The ASD and LRFD governing loads are presented in Table 2 along with the code equation numbers which produce them. Most of the connections are governed by the block shear loads calculated from Eqs. 1 through 3. Connections #6, #14, and #22 were governed by the combination of lead bolt shearing and block shear through the remaining bolt pattern (the type E failure discussed previously). Some of the longer connections for the smaller angles were governed by net section failure.

The ratios of the failure loads to allowable loads are also shown in Table 2. The ratio of the failure load to the ASD allowable load and LRFD unfactored nominal resistance are given by R_A and R_L, respectively. The adequacies of these ratios will be discussed in a subsequent section.

RESULTS

The 38 connections tested were chosen so that many of the variables considered to be important to the block shear phenomenon could be isolated. Since the material properties varied from one connection to another, these factors were eliminated by calculating code loads and all appropriate ratios based on actual material strengths. The effect of several other parameters are now examined.

The Outstanding Leg—To demonstrate how the outstanding leg influences the failure load, nondimensional average failure loads are plotted versus the length of the outstanding leg for constant connection geometries. The failure load is nondimensionalized by dividing by $F_u A^*$, the product of the ultimate strength and the gross area of the 6×6 or 5×5, whichever is appropriate. The results are presented in Fig. 4. Code equations predict that as the outstanding leg increases, the failure load increases since the gross area increases. Eventually, block shear will govern and the failure load will then remain constant since the code equations do not contain any outstanding leg effects.

In all, twelve different connection geometries were each tested for three different outstanding legs. For instance, connections #24 to #16 to #8 are plotted as the 6-in. $4/4^0$ line in Fig. 4. Ten of the twelve connection geometries actually show a decrease in failure load as the outstanding leg increases. Only two connections produced a monotonic increase (the 6-in. $4/4^0$ and the 5-in. $2/3^-$ patterns). The $4/4^0$ connection exhibits no block shear, and the increase shown in Fig. 4 is as expected for net section failure. The 5-in. $2/3^-$ patterns are the longest 5-in. connections and failures and code predictions are partially net section and block shear.

The decrease in failure load with increasing outstanding leg size and, therefore, gross area, required further investigation. A significant clue in explaining this behavior was clearly in evidence after observing all 114 failures. Almost every failure was preceded by necking down and eventual failure initiation at the points indicated in Fig. 1. Clearly, this indi-

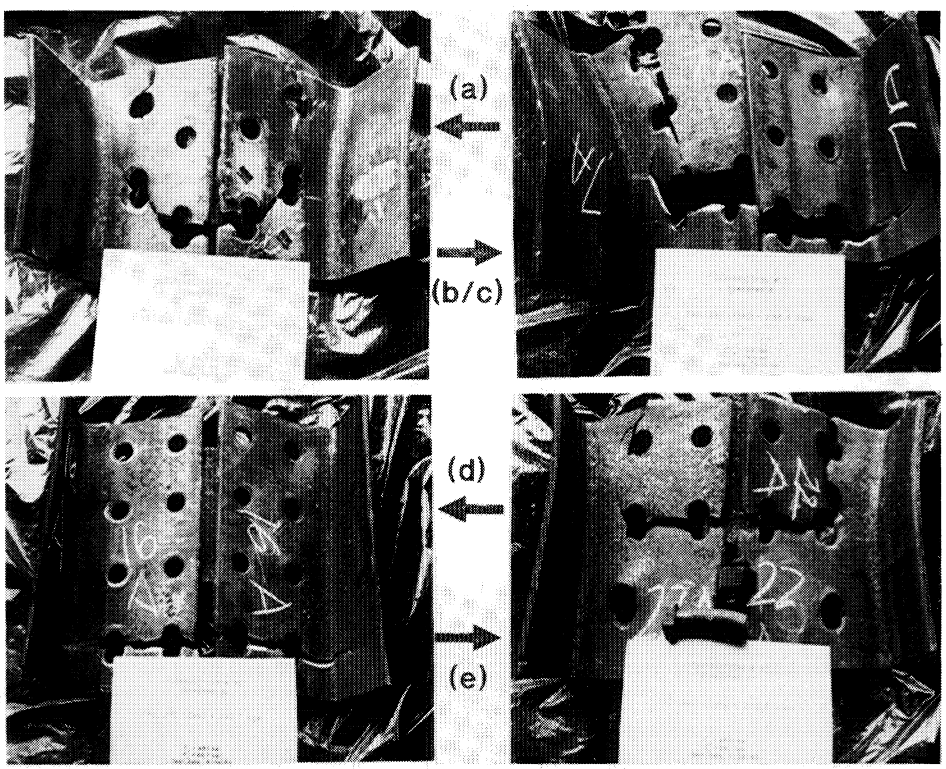

Fig. 3. Examples of various failure types.

Table 2.
Comparison of Test Loads to AISC Codes

Conn. #	Average Failure Load (kips)	ASD			LRFD		
		P_A (kips)	Equation No.	R_A	ϕP_N (kips)	Equation No.	R_L
1	182.5	105.7	J4-1,2	1.727	157.9	C-J4-1	0.867
2	204.2	131.7	J4-1,2	1.550	191.9	C-J4-2	0.798
3	188.7	102.6	J4-1,2	1.839	157.3	C-J4-2	0.900
4	242.7	141.1	J4-1,2	1.720	208.5	C-J4-2	0.873
5	204.9	125.9	J4-1,2	1.627	183.7	C-J4-2	0.837
6	259.7	137.2	J4,3-2	1.893	209.3	J3+CJ4	0.931
7	237.1	131.5	J4-1,2	1.803	202.5	C-J4-2	0.878
8	>297.7	160.9	J4-1,2	1.850	247.4	C-J4-2	0.902
9	202.7	103.5	J4-1,2	1.958	154.9	C-J4-1	0.981
10	203.9	116.7	J4-1,2	1.747	172.2	C-J4-2	0.888
11	194.2	96.5	J4-1,2	2.012	152.1	C-J4-2	0.958
12	247.1	146.3	J4-1,2	1.689	217.1	C-J4-2	0.854
13	189.1	120.1	J4-1,2	1.575	181.3	C-J4-2	0.782
14	219.8	128.9	J4,3-2	1.705	200.3	J3+CJ4	0.823
15	218.6	114.1	J4-1,2	1.916	178.7	C-J4-2	0.917
16	243.5	138.7	D1	1.756	208.0	D1-2	0.878
17	198.2	106.5	J4-1,2	1.861	154.1	C-J4-1	0.965
18	198.8	131.1	J4-1,2	1.516	193.3	C-J4-2	0.771
19	199.3	106.3	J4-1,2	1.875	161.7	C-J4-2	0.924
20	238.5	125.2	J4-1,2	1.905	191.6	C-J4-1	0.934
21	216.1	118.7	J4-1,2	1.821	178.5	C-J4-2	0.908
22	250.6	129.2	J4,3-2	1.940	198.0	J3+CJ4	0.949
23	236.5	121.8	J4-1,2	1.942	183.7	C-J4-2	0.966
24	255.2	137.6	D1	1.855	206.4	D1-2	0.927
25	154.1	76.0	J4-1,2	2.028	117.9	C-J4-2	0.980
26	155.8	92.7	J4-1,2	1.681	143.8	C-J4-2	0.813
27	194.9	102.6	J4-1,2	1.900	157.9	C-J4-2	0.926
28	169.6	105.6	J4-1,2	1.606	163.3	C-J4-2	0.779
29	174.1	87.7	J4-1,2	1.985	131.6	C-J4-2	0.992
30	171.8	102.2	J4-1,2	1.681	152.7	C-J4-2	0.844
31	208.8	110.7	J4-1,2	1.886	165.5	C-J4-2	0.946
32	189.9	109.4	J4-1,2	1.736	164.3	C-J4-2	0.867
33	149.4	72.8	J4-1,2	2.052	113.0	C-J4-2	0.992
34	161.5	89.4	D1	1.806	134.1	D1-21	0.903
35	187.2	97.8	D1	1.914	146.8	D1-2	0.956
36	163.0	92.1	J4-1,2	1.770	139.9	C-J4-2	0.874
37	173.3	80.1	J4-1,2	2.164	123.5	C-J4-2	1.052
38	126.8	68.5	J4-1,2	1.851	106.6	C-J4-2	0.892
			average = 1.820			average = 0.901	

cated the presence of bending for this tension loading. In fact, the eccentricity associated with the loading was shown to account for the observed test behavior. Eccentricity turns out to be the key to explaining the behavior shown in Fig. 4. Adidam,[14,15] demonstrated this by analytically varying the eccentricity. Chamarajanagar[16] obtained the same conclusions using finite element studies.

Stagger—Connection #3 has no stagger $(2/2^0)$. Connection #1 has positive stagger $(2/2^+)$. Both connections are $6 \times 6 \times 5/16$ angles and have the same shear area. When the test results are nondimensionalized for material properties, the effect of stagger can be isolated. Code equations predict an increase in loads, as the result of the addition of the $s^2/4g$ factor to the width. There are two other sets of connections which differ only in that one has zero and the other has positive stagger. The ASD code predicted increase for these geometries is 5.2 percent (see Ref. 14 for the calculation details). This compares with a test average of a 1.7 percent increase. Table 3 shows this result.

Other connections which differ only in stagger are also shown in Table 3. There are seven sets of connections that only differ in having negative versus positive stagger. The codes do not recognize any difference in the sign of the stagger and, hence, predict the same failure load. The average of all the tests showed an increase of 1.4 percent. There are three sets of connections that only differ in having zero versus negative stagger. The codes predict an increase of 4 percent while the tests averaged a decrease of 2.4 percent.

In an analytical investigation, using finite elements, Thacker and Epstein[17] demonstrated that the stagger should probably have a sign associated with it. That is, positive stagger, as defined by this study, should have an increase in failure load, and negative stagger a decrease. These test results are hardly conclusive, but do reinforce the analytical work. The results presented here are for the ASD code treatment. LRFD comparisons are more difficult due to the inclusion of both F_y and F_u, and two spearate equations for block shear. When attempted, however, LRFD comparisons show results similar to those presented here for ASD.

Shear Length—Table 4 presents the comparisons for ASD code and test results for those connections that only differ in that parameter. In some cases, however, there was also a change in the sign of the stagger (which makes no difference in the code results). The results seem to indicate that, on the average, the code is reasonably taking this parameter into account. When the sign of the stagger is incorporated into the code equations, the comparisons, on the average, are even better.

A question which naturally arises is which connection geometry to choose. For instance, if the load to be transmitted requires four bolts, should the specified connection be a $2/2,^+$ a $2/2,^-$ or an unstaggered $2/2^0$ connection, if that is appropriate. There are opposing factors at work in making this decision. The first is the increase in block shear strength associated with an increase in shear length. The second is the decrease probably associated with negative stagger.

The test results indicate that the increase in shear length, associated with the negative stagger patterns, more than offsets the decrease due to the stagger. Therefore, if block shear is the mode of failure and stagger is either desirable or required, the negative stagger pattern should be specified. However, when the connections become long enough so that net section is the failure mode, the positive stagger pattern should probably be specified.

ADEQUACY OF THE AISC CODES FOR BLOCK SHEAR

For ASD, the assumed factor of safety in connection designs is 2.0. As seen in Table 2, the tests averaged nine percent less than their desired strength. For LRFD, the nominal code capacity should equal the failure loads of tests. On this basis, the tests averaged 9.9 percent less than their desired strength. It also takes into account the scatter of the test data by finding the coefficient of variation of the test resistance.

Analysis of the data from the 38 connections tested on the basis of a statistical approach is inappropriate for this study. If the tests were only for block shear, this would make sense. However, the tests spanned the range from true block shear failure through net section failures. Therefore, while on the surface the average results indicate that the codes reasonably predict failure loads, a delineation from block shear to net section failure must be accomplished.

When the results are grouped, consistent trends appear. For instance, the ratios R_A or R_L are seen to increase, on the average, as connections become longer. For the same length of connections, these ratios also increase as the outstanding

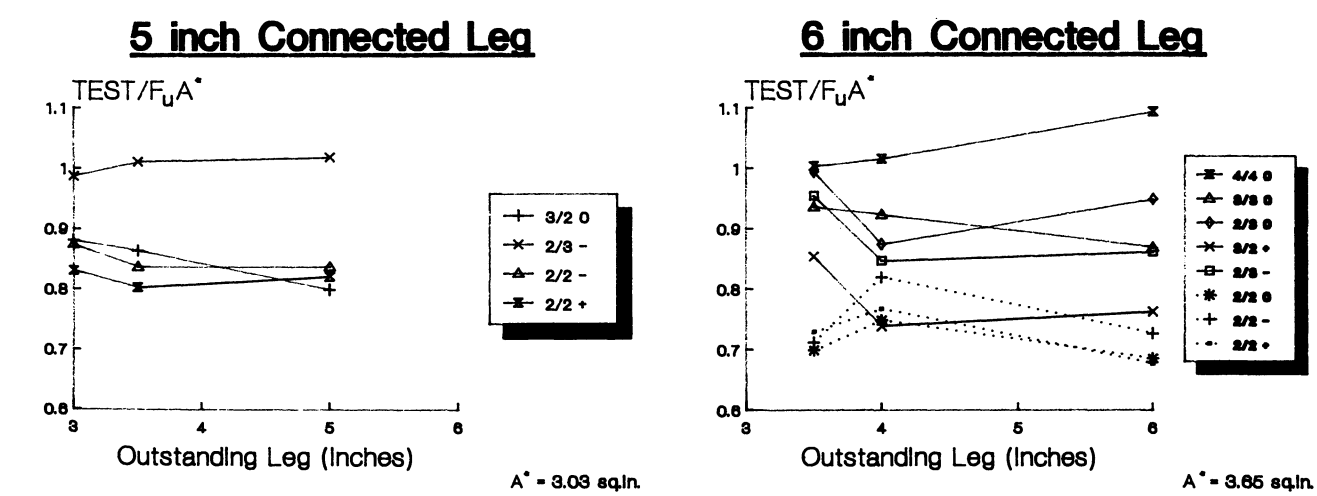

Fig. 4. Effect of the outstanding leg.

Table 3.
Stagger Effects

Conn. Leg	Connections	% increase $s^2/4g$	% increase Test
6"	#3, #11, #19 → #1, #9, #17; 4.5" (shear length); 0 → +	+5.2%	+1.7% (average of 9 specimens)
6"	#2, #11, #18 → #5, #13, #21; 6"		
5"	#37 → #33; 4.5"	zero	+1.4% (average of 21 specimens)
5"	#26, #30, #34 → #28, #32, #36; 6"; − → +		
6"	#7, #15, #23 → #4, #12, #20; 7.5"; 0 → −	+4.0%	−2.4% (average of 9 specimens)

Table 4.
Shear Length Effects

Conn. Leg	Connections	% increase Code	% increase Test
6"	4.5" → 7.5"; #3, #11, #19 → #7, #15, #21	+29.3%	+28.0% (average of 9 specimens)
5"	4.5" → 7.5"; #37 → #35	+32.5% (+38.9%) (average of 3 specimens)	+12.9%
5"	3.0" → 6.0"; #38 → #36	+36.0%	+30.0% (average of 3 specimens)
6"	#1, #9, #17 → #4, #12, #20; 4.5" → 7.5"	+27.9% (+18.0%)	+22.8% (avg. of 9 specs.)
5"	#25, #29, #33 → #27, #31, #36; 4.5" → 7.5"	+32.5% (+16.6%)	+23.0% (avg. of 9 specs.)
	Average of all the above	+30.7% (+24.2%)	+24.0%

( ) = % using $-s^2/4g$ for negative stagger

legs become shorter. Putting this another way, the ratios are closer to their desired values the more that net section is the mode of failure and for true block shear failures, the equations are not as conservative.

There are many ways of representing the results to demonstrate the observed trends. The parameter that best represents the transition from block shear to net section failures is the same used in the study of the shear lag effects,[13] $1 - x/L$. As the connection length increases, this coefficient increases and approaches one for long connections (net section). Also, as the outstanding leg decreases, the centroidal distance x decreases and the coefficient again increases and approaches one.

Figure 5 plots the ratio R_A and R_L as functions of $1 - x/L$. The 38 data points for each represent the ratios given in Table 2 and the corresponding $1 - x/L$ calculated from the centroidal distance and the length of each connection.[14,15] The trends in the data become evident when a regression analysis is accomplished. Least square straight line fits of the data are shown in the figure. It appears that if connections were considered that only exhibited block shear failure, the code equations would be significantly deficient.

NEEDED AISC CODE MODIFICATIONS

There are many factors that point to the modifications needed in the present code equations. First of all, it was demonstrated that eccentricity is present in these connections and accounts for the trends observed. Then, it was seen that the code treatment for the shear contribution to the block shear equations is adequate. If these conclusions are accurate, the implication is that the tension contribution is not being adequately addressed. When one considers that shear lag (eccentricity) for net section tension failure incorporates a reduction coefficient, it then appears obvious that this coefficient is required for block shear as well.

Figure 6 shows the results for R_A and R_L when the code U is incorporated into the AISC code equations as follows:

ASD (J4-1,2):

$$P = 0.3A_{nv}F_u + 0.5UA_{nt} \tag{4}$$

LRFD, the larger of (C-J4-1):

$$\phi P_n = 0.75(0.6F_y A_{vg} + F_u UA_{nt}) \tag{5}$$

or (C-J4-1):

$$\phi P_n = 0.75(0.6F_u A_{ns} + F_y UA_{tg}) \tag{6}$$

The average value of R_A, when U is included as in Eq. 4, is now 2.038. Not only that, but the results of the regression analysis on these new values of R_A, as shown in Fig. 6, produce a conservative line with little slope. Similar behavior is obtained for the LRFD ratio R_L for which the average value of R_L is now 1.008 when the U factor is included, as in Eqs. 5 and 6. The regression analysis for the new values of R_L, as shown in Fig. 6, also appears appropriate. Both lines are in excellent agreement with their ideal values. It is therefore strongly recommended that Eqs. 4 through 6 be used for block shear.

CONCLUSIONS

This study was conducted to see if the block shear failure of angles in tension is substantially different from that of beam web-to-column connections. The results of this study have demonstrated this as well as showed some shortcomings in the AISC code equations that were based on the beam web studies. The effect of several of the parameters in the connection geometry was investigated. The eccentricity inherent in the loading of these angles was shown to be a significant factor in their failure. Eccentricity accounted for the role that the length of the unconnected, outstanding leg played in the failures.

The primary conclusion of this study is the need for modifications in the AISC code treatment of block shear. The proposed inclusion of the shear lag reduction coefficient U for the tension area appears to produce appropriate results. The extension to structural shapes other than angles, structural tees, for instance, is a subject that will require further investigation. However, based on the inherent bending associated with any tension member having $U < 1$, it seems reasonable that the proposed code treatment should be appropriate and conservative for other shapes as well.

ACKNOWLEDGMENTS

This study was made possible by grants from the American Institute of Steel Construction, The National Science Foundation, and the University of Connecticut Research Foundation. Special thanks are due to the Berlin Steel Construction Company, Berlin, CT, for donating the test specimens. David Ricker, of Berlin Steel, generously gave his time and effort to this project.

Three Connecticutt firms, The New Haven Testing Company (New Haven), Raymond Engineering (Middletown), and George Torello, Jr., Consulting Engineers (Old Lyme), all donated to this project. Their contributions are gratefully acknowledged. Naren Adidam, a very gifted graduate student who contributed immensely to this project, tragically died in an automobile accident near the conclusion of this study. This work is dedicated to his memory.

REFERENCES

1. Birkemoe, P. C., and M. I. Gilmor, "Behavior of Bearing Critical Double-Angle Beam Connections," AISC *Engineering Journal,* Vol. 15, 4th Quarter, 1978, pp. 109–115.
2. Yura, J. A., P. C. Birkemoe, and J. R. Ricles, "BeamWeb Shear Connections: An Experimental Study," ASCE *Journal of the Structural Division,* Vol 108, No. ST2, Feb. 1982, pp. 311–325.
3. Ricles, J. R., and J. A. Yura, "Strength of Double-Row Bolted Web Connections," ASCE *Journal of Structural Engineering,* Vol. 109, No. ST1, Jan. 1983, pp. 126–142.
4. Madugula, M. K. S., and S. Mohan, "Angles in Eccentric Tension," ASCE *Journal of Structural Engineering,* Vol. 114, No. ST10, Nov. 1988, pp. 2387–2396.

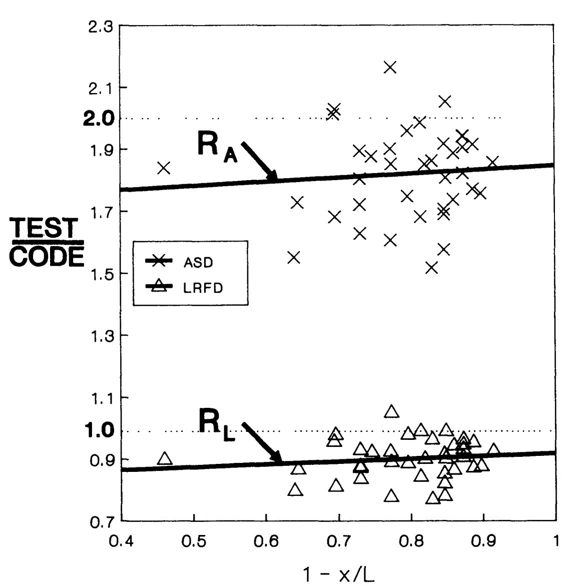

Fig. 5. R versus 1 – x / L.

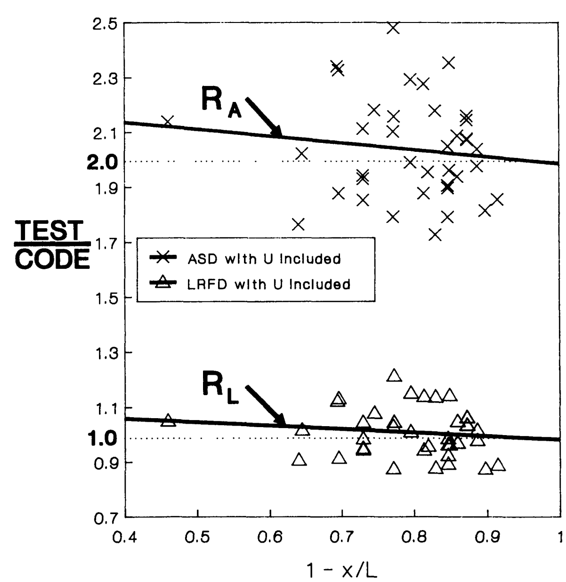

Fig. 6. R versus 1 – x / L with the proposed code modification included.

5. American Institute of Steel Construction, *Manual of Steel Construction, Allowable Stress Design,* 9th ed., Chicago: AISC, 1989.
6. American Institute of Steel Construction, *Manual of Steel Construction, Load and Resistance Factor Design,* 1st ed., Chicago: AISC, 1986.
7. Hardash, S., and R. Bjorhovde, "New Design Criteria for Gusset Plates in Tension," AISC *Engineering Journal,* Vol. 22, 2nd Quarter, 1985, pp. 77–94.
8. Thacker, B. H., "An Analytical Investigation of Block Shear Failure in Structural Tension Members," M.S. Thesis, University of Connecticut, 1987.
9. Gulia, F. S., "Analysis of the Effects of Stagger in Structural Tension Members," M.S. Thesis, University of Connecticut, 1988.
10. Munse, W. H., and W. H. Chesson, Jr., "Riveted and Bolted Joints: Net Section Design," ASCE *Journal of the Structural Division,* Vol. 89, No. ST1, Feb. 1963, pp. 107–126.
11. Yura, J. A., "Elements for Teaching Load and Resistance Factor Design," American Institute of Steel Construction, 1988.
12. Smith, J. C., *Structural Steel Design,* New York: John Wiley & Sons, 1988.
13. McCormac, J. C., *Structural Steel Design: LRFD Method,* New York: Harper & Row, 1989.
14. Adidam, N. S., "Analysisof Block Shear Experiments for Structural Steel Angles in Tension," M.S. Thesis, University of Connecticut, 1990.
15. Epstein, H. I., and N. S. Adidam, "The Effect of Eccentric Tension on the Block Shear Failure of Angles," *Advances in Structural Testing, Analysis and Design,* ICSTAD Proceedings, Bangalore, India, 1990, Vol. II, pp. 758–763.
16. Chamarajanagar, R., "Finite Element Studies for the Correlation of Stresses and Failures of Block Shear Tests," M.S. Thesis, University of Connecticut, 1990.
17. Epstein, H. I., and B. H. Thacker, "The Effect of Bolt Stagger for Block Shear Tension Failures in Angles," *Computers and Structures,* Vol. 39, No. 5, pp. 571–576.