Block Shear of Structural Tees in Tension—Alternate Paths

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ABSTRACT

There is little in the way of test results for structural tees used in tension. This is especially true when it comes to results for block shear failures. By varying only the depth of the web, a limited experimental program was designed to produce failures ranging from net section to block shear for tees connected by bolts through their flanges. As the depth increased, block shear failures did result, but not along the paths anticipated. This alternate block shear path is the focus of this paper.

INTRODUCTION

Angles are commonly used as bracing members. As such, they are required to primarily resist tensile forces. As the length of these bracing members increases or as the force they are designed to carry becomes large, pairs of angles may be specified. This helps reduce the slenderness ratios and the inherent eccentricity present when using a single angle.

Figure 1a shows a common arrangement for pairs of angles used for tension. They are connected to opposite sides of a gusset plate. Pairs of angles connected this way are also routinely used for primary tension or compression elements. When there is cross bracing (in the same bay), pairs of angles are usually connected to the same side of the gusset plate, as shown in Figure 1b, in order to avoid interference from the crossing member. If these angles are not stitch bolted together through their outstanding legs, they each carry tensile load exactly as do their counterparts in Figure 1a. When they are stitch bolted, they act much the same as the single structural tee shown in Figure 1c.

Failure of these structural tension elements can occur due to yielding of the gross cross-sectional area, rupture of the effective net cross section (which includes a reduction coefficient U, in LRFD notation), or block shear. The block shear paths usually considered for the double angles of Figure 1b or the tee of Figure 1c are shown in Figure 2.

Through the reduction coefficient (U), the outstanding legs of the double angles and the outstanding leg of the tee have an effect upon the failure of these elements in rupture of the effective net section. This reduction coefficient essentially takes into account the eccentricity inherent in these connections. In a previous study by the author,[1] the outstanding legs of the angles were also shown to have an effect on block shear capacity. This prompted a study to investigate the possibility of similar behavior for structural tees. There is little experimental investigation of structural tees in tension, and the block shear failure for these sections does not appear to have been previously addressed.

The author has now conducted a limited experimental investigation for structural tees in tension, connected only by their flanges (similar to Figure 1c). The original purpose of the specimens fabricated for this experimental program was to investigate the transition from net section to block shear failure and the effect that the outstanding leg might have on these failures. The specimens were all fabricated from the same W section. However, instead of just cutting the W in half lengthwise to produce the usual WT, varying outstanding legs were produced by also cutting the W lengthwise into other proportions.

On the basis of the limited results of the experimental program, it appears that the outstanding leg not only has an effect on the net section rupture but, in a similar manner to angles, may also influence the block shear failures in tees. However, this is not the information that will be presented in this paper as that will wait until additional test results are available. Instead, this paper will focus on a mode of block

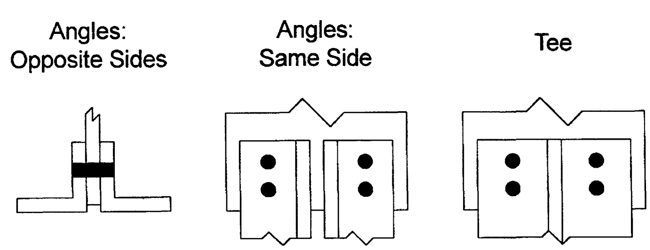

Fig. 1. Tension connections for angles and tees.

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shear failure that was a surprise to this investigator, as it probably will be to the reader.

BLOCK SHEAR

Block shear failure is produced by a combination of tension on a plane perpendicular to the load and shear on a plane parallel to the load. The nature of the failure dictates that when rupture occurs on one plane, the specimen has failed, even though it may only be at yield on the perpendicular plane. The latest LRFD specification (1994, Second Edition[2]) contains two equations governing block shear

$$\phi R_n = \phi[0.6F_y A_{gv} + F_u A_{nt}] \quad (1a)$$

$$\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}] \quad (1b)$$

where:

ϕ = 0.75
A_{nv} = net area subject to shear
A_{nt} = net area subject to tension
A_{gv} = gross area subject to shear
A_{gt} = gross area subject to tension
F_y = material tensile yield strength
F_u = material tensile ultimate strength

Equation 1a represents block shear strength determined by rupture of the net tensile area(s) combined with shear yielding of the gross section on the shear area(s). Equation 1b represents block shear strength determined by rupture of the net shear area(s) combined with yielding of the gross tensile area(s). These equations are based on the work of Ricles and Yura[3] as well as that of Hardash and Bjorhovde.[4] Except for slight differences in notation, these equations did not change from the first edition of the LRFD Manual[5] to the second edition. However, the latest code now states in the commentary that "the proper equation to use is the one with the larger rupture term." In other words, when $F_u A_{nt} \geq 0.6F_u A_{nv}$, use Equation 1a and when $F_u A_{nt} < 0.6F_u A_{nv}$, use Equation 1b. Previously, the larger of the two equations was the one used.

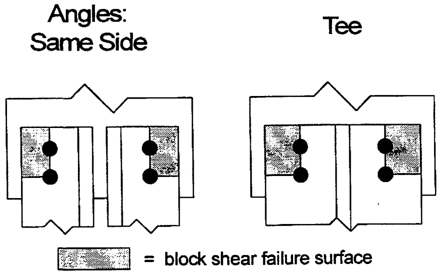

Fig. 2. Block shear failure paths.

This new treatment is more conservative for some connection parameters, as previously examined by the author.[6]

BLOCK SHEAR FOR A STRUCTURAL TEE

Consider a structural tee used in tension, as shown in Figure 3. The tee is assumed to have yield strength F_y and ultimate strength F_u. The WT or ST section (an ST is shown in Figure 3) has a gross area A_g and cross-sectional dimensions given by the standard notations in the LRFD Manual as follows:

b_f = flange width
t_f = flange thickness
d = depth (usually half the depth of the corresponding W or S section)
t_w = web thickness

The WT or ST is connected through the flange with N bolts of diameter D. The bolts are spaced at a gage g, have a pitch p, and longitudinal edge distance e.

In the calculation of the areas needed for rupture failure of the net section

$$A_e = UA_n \quad (2)$$

where A_e is the effective net area,

$$A_n = A_g - 2(D + 1/8)t_f \quad (3)$$

and

$$U = 1 - x/L \leq 0.9 \quad (4)$$

unless a larger value can be justified. In Equation 4, x is the centroidal distance (eccentricity of the connection), shown in Figure 3, and

$$L = p(1/2 N - 1) \quad (5)$$

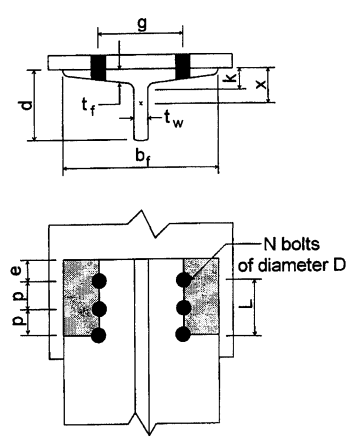

Fig. 3. Geometry of a typical tee connection showing usual block shear paths.

In the above equations, all dimensions are assumed to be in inches, and the holes are punched.

For each of the two identical block shear paths in Figure 3, the areas needed for these calculations are

$$A_{gv} = (p(\tfrac{1}{2}N - 1) + e)t_f \quad (6a)$$

$$A_{gt} = (b_f - g)t_f / 2 \quad (6b)$$

$$A_{nv} = A_{gv} - \tfrac{1}{2}(N - 1)(D + \tfrac{1}{8})t_f \quad (6c)$$

$$A_{nt} = A_{gt} - \tfrac{1}{2}(D + \tfrac{1}{8})t_f \quad (6d)$$

The block shear strength, ϕR_n, obtained from substituting these areas (and the material properties F_y and F_u) into the appropriate block shear equation (either 1a or 1b), must be compared to the strengths predicted from yielding of the gross cross section

$$\phi_t P_n = 0.90 F_y A_g \quad (7)$$

or rupture of the effective net section

$$\phi_t P_n = 0.75 F_u A_e \quad (8)$$

with the lowest of the three values determining the capacity of the WT.

A STRUCTURAL TEE EXAMPLE

As an example, consider a WT6×9.5 of A36 steel used in tension and connected as shown in Figure 3. The gross area $A_g = 2.79$ in.^{2} Therefore, the strength predicted from Equation 7 is

$$\phi_t P_n = 0.90(36)2.79 = 90.4 \text{ kips.}$$

If ¾-in. ϕ bolts (D) are to be used, since the flange thickness, t_f, is 0.350 in., the net section from Equation 3 is

$$A_n = 2.79 - 2(\tfrac{3}{4} + \tfrac{1}{8})0.350 = 2.18 \text{ in.}^2$$

In order to determine the effective net area, the geometry of the connection must be known. Assuming that six bolts are required for this connection and using a pitch (p) equal to three bolt diameters (2.25 inches), Equation 5 gives $L = 4.5$ inches. Since $x = 1.65$ inches for this WT, it follows from Equation 4 that

$$U = 1 - (1.65 / 4.5) = 0.64 < 0.9$$

Here is an area of concern for a structural engineer. Small values of U are not only possible, but very likely for many structural tees of modest size. In fact, for the connection in this example only four bolts could be required (for instance, A490-X @ 24.9 kips factored load per bolt[7]) for which $L = 2.25$ inches which gives $U = 0.27$! This value for U would produce an effective net area that is considerably less than the flange area. The current LRFD Specification[2] adopted $U = 1 - x / L$ in the body of the specification (instead of where it formerly appeared, the commentary) states that, "Larger values of U are permitted to be used when justified by tests or other rational criteria." The current LRFD commentary partially addresses this issue by stating that for tees, acceptable values are retained from previous issues of the Specification:

(a) W, M, or S shapes with flange widths not less than two-thirds the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress, $U = 0.90$.

(b) W, M, or S shapes not meeting the condition of subparagraph a, structural tees cut from these shapes, and all other shapes including built-up cross sections, provided the connection has no fewer than three fasteners per line in the direction of stress, $U = 0.85$.

(c) All members having only two fasteners per line in the direction of stress, $U = 0.75$.

Therefore, for the connection in this example that uses six bolts, it is perfectly reasonable and acceptable to take $U = 0.85$. It then follows that the effective net area is $0.85 \times 2.18 = 1.85$ in.^{2} Then, the strength predicted from Equation 8 is

$$\phi_t P_n = 0.75(58)1.85 = 80.5 \text{ kips.}$$

The calculation of the block shear capacity requires several additional geometric parameters. The flange width (b_f) for this section is 4.005 inches, the gage (g) for this section (taken from "dimensions for detailing" from previous AISC Specifications) is 2.25 inches and the edge distance (e) is the usual 1.25 inches for these ¾-in. bolts. Then, substituting into Equations 6a-d gives

$$A_{gv} = [2.25((6 / 2) - 1) + 1.25]\ 0.350 = 2.013 \text{ in.}^2$$

$$A_{gt} = \tfrac{1}{2}\ (4.005 - 2.25)\ 0.350 = 0.307 \text{ in.}^2$$

$$A_{nv} = 2.013 - \tfrac{1}{2}\ (6 - 1)\ (\tfrac{3}{4} + \tfrac{1}{8})\ 0.350 = 1.247 \text{ in.}^2$$

$$A_{nt} = 0.307 - \tfrac{1}{2}\ (\tfrac{3}{4} + \tfrac{1}{8})\ 0.350 = 0.154 \text{ in.}^2$$

Since $A_{nt} = 0.154 < 0.6\,A_{nv} = 0.748$, the use of Equation 1b is currently specified. This produces a block shear strength

$$\phi R_n = 0.75[0.6(58)1.247 + 36(0.307)] = 81.7 \text{ kips.}$$

Therefore, it would appear that block shear does not govern and the capacity of this WT is 80.5 kips, as calculated from net section rupture.

EXPERIMENTS SHOW AN ALTERNATE BLOCK SHEAR PATH

As stated earlier, a limited experimental program was conducted to investigate failures of structural tees used in tension. The designing of the specimens to be tested anticipated the possibility of block shear failures and used calculations similar to those of the preceding section. The experimental program had the luxury of using non-standard WT sections made by fabricating the specimens with depths that were less than, equal to, and greater than the standard half W section.

For small depths (d), net section rupture was anticipated to be the mode of failure, and as the depth increased there would be a transition to the block shear failure shown in Figure 3. Well, there was such a transition, but the block shear path was not the one shown in Figure 3. Figure 4a is a picture showing the block shear failure that resulted. It has a tension path that encompasses the entire flange area. There is only one shear path (not two), and that path lies in the plane of the outstanding leg (web). Figure 4b shows the failures of a family of four similarly connected structural tees, with the only variable being the depth (i.e., the length of the outstanding leg). The second number in the specimen designation is the depth, d (in inches). The first three specimens (2-3, 2-4A and 2-5) failed in net section. The fourth (26-B) is the block shear failure that was also shown in Figure 4a. Upon reflection, and subsequent calculations, the specimens outsmarted the experimenter, and failed exactly as they should have.

For calculating the block shear failure load for the path in Figure 4a, refer to Figure 5. The minimum area for the tension plane encompasses the entire flange and extends to the web toe of the fillet between the flange and web. The distance from the outer face of the flange to this point is designated in AISC publications by k (see Figure 5). As can be seen in this figure, the areas necessary for the calculation of this block shear failure load are

$$A_{gv} = A_{nv} = (L + e)t_w \tag{9a}$$

$$A_{gt} = A_g - (d - k)t_w \tag{9b}$$

$$A_{nt} = A_{gt} - 2(D + 1/8)t_f \tag{9c}$$

Before applying these equations to the WT6×9.5 example of the previous section, it must be pointed out that the experimental results, partially shown in Figure 4, were not obtained for the connection used in the example. In fact, the alternate path block shear failure that was produced is not predictable using current AISC codes (possibly implying that the current codes may needed amending once further experimental evidence is obtained). Therefore, please disregard the actual failure loads shown in Figure 4. Keep in mind that the experimental results showed the existence of this alternate block shear path and the continuation of the example pre-

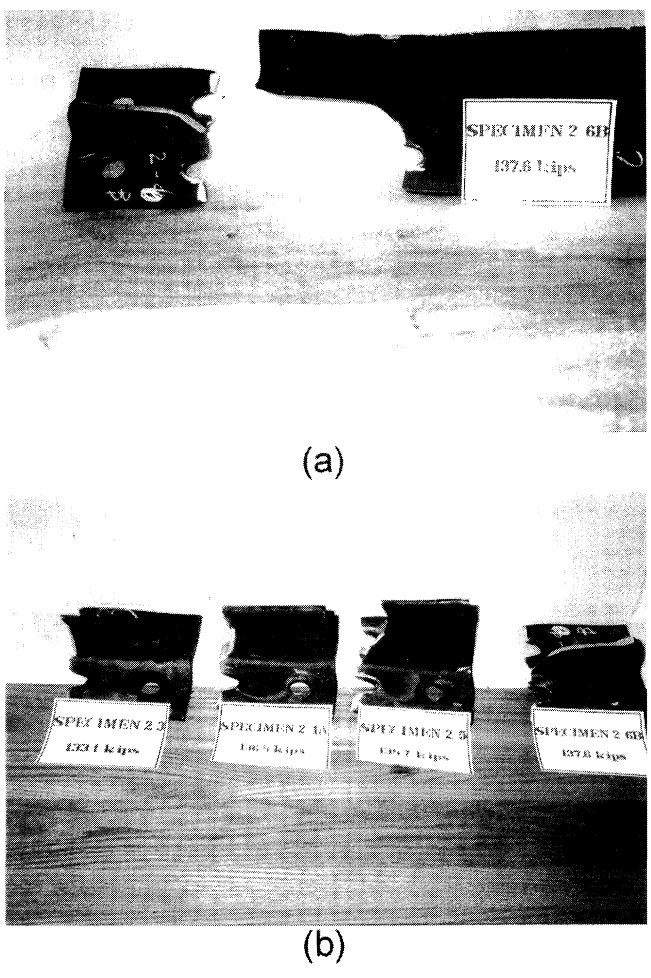

Fig. 4. Experimental failure modes: (a) specimen exhibiting block shear failure with a single shear plane through the web; (b) failures of a series of specimens which had varying depths. These tested connections are not the example presented in this paper (disregard the loads shown).

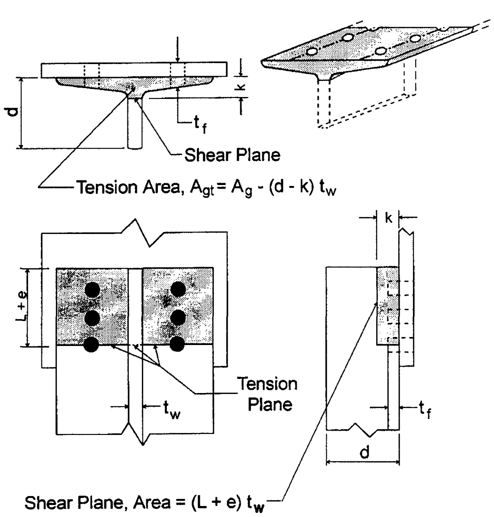

Fig. 5. Geometry of the block shear path having a single shear plane through the web.

sented below will show that this alternate path will govern the design of some connections even when current code provisions are applied.

THE ALTERNATE BLOCK SHEAR FAILURE PATH APPLIED TO THE EXAMPLE

For the WT6×9.5 example, the additional dimensions needed for substituting into Equations 9a–c are the web thickness (t_w) = 0.235 inches, the overall depth (d) = 6.080 inches and the distance from the outer face of the flange to the toe of the web fillet (k) = 13⁄16-in. Along with the dimensions and areas previously given, substitution into Equations 9a–c yields

$$A_{gv} = A_{nv} = (4.5 + 1.25)0.235 = 1.351 \text{ in.}^2$$

$$A_{gt} = 2.79 - (6.080 - 0.813)0.235 = 1.552 \text{ in.}^2$$

$$A_{nt} = 1.552 - 2(3/4 + 1/8)0.350 = 0.949 \text{ in.}^2$$

and since $A_{nt} = 0.949 > 0.6\,A_{nv} = 0.11$, the use of Equation 1a is required. This produces a block shear strength

$$\phi R_n = 0.75[0.6(36)1.351 + 58(0.940)] = 62.8 \text{ kips}$$

which is considerably less (23 percent) than the previous block shear path. Since this 62.8 kip value is also less than the previously governing effective net section rupture, this block shear failure becomes the design strength for this example. Many other WT and ST sections having appropriate connections would similarly have this alternate block shear path produce lower design loads than the path shown in Figure 2. Further, when connections are relatively short, this new alternative path will frequently govern the overall design strength.

Is this block shear path a surprise? For an angle welded to a gusset plate, Smith[8] in his textbook shows a potential block shear path that has a tension plane that encompasses the entire connected leg and extends into the outstanding leg. The shear plane is completely in the outstanding leg. This is certainly similar to the governing block shear path for the WT of this example.

It can be argued that a prudent engineer should recognize this potential path. The phenomenon of block shear was first investigated for coped beam connections. During the 1970s and early 1980s, few designers recognized the applicability of block shear failure to tension members. In fact, textbooks at the time showed many examples of tension connections where block shear should have been checked for the structural member, but wasn't. It wasn't until the first edition of the LRFD Specification in 1986 that figures for tension member block shear paths (similar to those in Fig. 2) first appeared in AISC literature. This brings back the question of the consideration of the alternate block shear path presented in this paper. The author talked with many structural engineers concerning the way they would check the design of these WT or ST bolted connections. None of them considered this alternate path.

BLOCK SHEAR FAILURE IN TEES— QUESTIONS REMAIN

This brief paper has shown that the observed alternate failure path for block shear in tees may govern the capacity of some connections, according to current codes. In recent tests on A36[1] and high strength angles,[9] experimenters are finding that current block shear equations may not adequately predict block shear failures. It is not clear whether current code equations will adequately predict block shear failure loads for tees (with either path).

The limited experimental results for the block shear failure of tees, as described in this paper, seem to show that increasing the eccentricity of the connection will probably reduce the block shear failure load. If further testing proves this to be the case, the current block shear equations may need modification for angles as well as tees. The resulting reduction in capacity will bring many more structural tee connections into the domain in which block shear failure (by either mode of block shear) will govern.

The author hopes to be conducting additional tests on structural tee connections in the near future. In the meantime, this paper was presented primarily to help the design profession become aware of the possible alternative block shear paths for this type of structural tee tension connection.

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