Using Moment and Axial Interaction Equations to Account for Moment and Shear Lag Effects in Tension Members

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Shear lag effects and moments reduce the strength of structural steel tension members. This shear lag exists when connections to tension members transmit the load through some, but not all, elements of the member (legs of an angle, web and flanges of a W or tee, etc.) and may result in an eccentrically loaded connection. Recent research on structural tees at the University of Connecticut has shown that the moment produced by eccentric loading depends upon the connection geometry as well as the rotational stiffness of the connection. Once the moments in tension tees and other sections are found, these members may be more correctly and safely designed by accounting for the interaction of bending moment and axial force, instead of the empirical shear lag factors. The rotational stiffness of the connections is shown to be an important design consideration.

INTRODUCTION AND BACKGROUND

Approximately half of the structural members in two-dimensional and three-dimensional (space) trusses are tension members. Structural tension members are also found when assemblies or entire structures are hung. Another significant use is for bracing elements (wind and seismic) of structures and in draglines in structural framing. Structural steel tension members are designed to insure that any possible failure mode, resulting from mandated magnitudes of applied loads, can be safely resisted. For many years, the design of these members was based upon avoiding failure from either the gross cross section yielding or the net section fracturing through any reduced cross section.

The LRFD (AISC, 1999) design strength for yielding of the gross section is given by

$$\phi P_n = \phi_{ty} F_y A_g \tag{1}$$

where P_n is the nominal axial strength, ϕ_{ty} is the resistance factor for yielding equal to 0.90, F_y is the specified minimum yield stress, and A_g is the gross area. Only the LRFD specifications will be referred to in this paper. However, any description or conclusion reached regarding LRFD specifications is equally applicable to the ASD specification treatment (AISC, 1989).

When there is a reduction in cross-sectional area, such as for bolt holes, net section failure strength is given by

$$\phi P_n = \phi_{tf} F_u A_e \tag{2}$$

where ϕ_{tf} is the resistance factor for fracture = 0.75, F_u is the specified minimum ultimate strength, and A_e is the effective net area.

If all the elements on the cross section are connected, the effective net area equals the net area. If some of the elements on the cross section are not connected, the net area, A_n, is reduced to the effective net area, A_e. Over the years, design specifications have treated the reduction in strength differently. For instance, some specifications ignored part of the area of unconnected elements of the cross section. Current specifications use a shear lag reduction factor to compensate for the increase in stress in those elements that are connected. An angle, connected by only one leg, and structural tees with only the flange or web connected, are examples of sections with reduced strength due to shear lag. AISC uses the shear lag factor U that reduces the net area to the effective net area through

$$A_e = UA_n \text{ where } U = 1 - \frac{\bar{x}}{\ell} \leq 0.90 \tag{3}$$

in which $\bar{x}$ is the connection eccentricity and ℓ is the connection length. Alternately, this reduction may be given by specific values, according to the commentary in the LRFD specification.

Birkemoe and Gilmor (1978) observed a possible failure mode for the connection of a coped beam. This failure involved tearing of the base metal along the perimeter of the bolt holes and this new failure mode was termed block shear. They suggested an equation for strength that combined tensile strength on one plane with shear strength on the perpendicular plane. While the treatment of block shear has undergone modest modifications over the years, specification equations for strength remain similar to those orig-

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inally posed. For nominal strength, R_n, the current LRFD specification (AISC, 1999) uses

$$\phi R_n = \phi[0.6 F_y A_{gv} + F_u A_{nt}] \le \phi[0.6 F_u A_{nv} + F_u A_{nt}] \quad \text{when } F_u A_{nt} \ge 0.6 F_u A_{nv} \tag{4a}$$

and

$$\phi R_n = \phi[0.6 F_u A_{nv} + F_y A_{gt}] \le \phi[0.6 F_u A_{nv} + F_u A_{nt}] \quad \text{when } 0.6 F_u A_{nv} > F_u A_{nt} \tag{4b}$$

where $\phi = 0.75$, A_{gv} is the gross area subjected to shear, A_{gt} is the gross area subjected to tension, A_{nv} is the net area subjected to shear, and A_{nt} is the net area subjected to tension. Prior to the 1999 Specification, the block shear strength was based on the fracture of the net tension or shear area together with yield of the gross remaining area (AISC, 1993). Now, the block shear strength cannot exceed that obtained from fracture of both net areas.

Block shear was first documented in coped beam connections. An angle in the debris of the Hartford Civic Center roof collapse led to block shear investigations in tension members. Figure 1 shows a typical block shear failure in (a) a coped beam and (b) an angle in tension. Epstein and Thacker (1991) used non-linear finite element analyses to model an angle with the dimensions of the one that failed in block shear in the collapse of the Hartford Civic Center. The analyses verified block shear as the mode of failure. By varying the stagger paths of the bolt holes, it was concluded that the direction of a staggered path influences the failure load, when shear lag is present.

Madugula and Mohan (1988) reviewed test results of angles in eccentric tension. They documented 13 block shear failures out of 61 angles tested. They concluded that the block shear failure mode might be critical for angles in eccentric tension.

In the years 1990-92, results were presented from the full-scale testing of double-row, staggered and unstaggered, bolted tension connections of structural angles (Adidam, 1990; Epstein and Adidam, 1991; Epstein, 1992). It was concluded that the factors of safety in AISC s ASD and LRFD appeared to be inadequate for block shear failures of angles. Also, as the area of the outstanding leg increases, the amount of eccentricity increases, reducing the strength of the member. To account for this, it was recommended that the shear lag reduction factor, U, should be included in the tension terms of Equations 4(a) and 4(b).

A limited series of tension tests on structural tees, with unconnected webs, was then performed (Twilley, 1996; Epstein, 1996). The testing program revealed a previously undocumented block shear failure path. Figure 2 shows the failure paths for (b) the expected failure path and (c) the previously undocumented (alternate) failure path. This alternate block shear failure path involved tension on transverse sections in the flange of the tee and shear on a longitudinal section in the web of the tee.

EFFFECT OF MOMENTS IN TEES USED IN TENSION

Several of the aforementioned studies have shown that the eccentricities, commonly found in block shear failures in tension connections, are important in determining failure modes and loads. Eccentricity of the load causes moments along the length of a member and produces non-uniform stresses. Therefore, the stresses in an eccentric tension member, in general, and in block shear failure paths, in particular, are not simply uniaxial, but also include bending stresses. As a first step in an ensuing investigation, McGinnis (1998) used finite element analyses to predict the failure modes of tees in tension. He compared the results of the finite element models with the previous test failures (Epstein, 1996). The models accurately predicted the test failure modes, including the previously undocumented alternate block shear failure path.

One interesting finding in McGinnis analyses of models with large eccentricities was the presence of a compressive

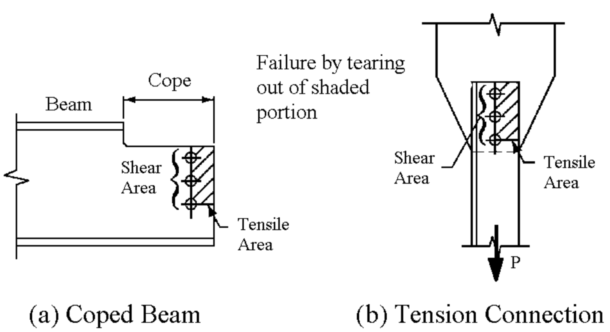


Fig. 1. Block shear failure paths.

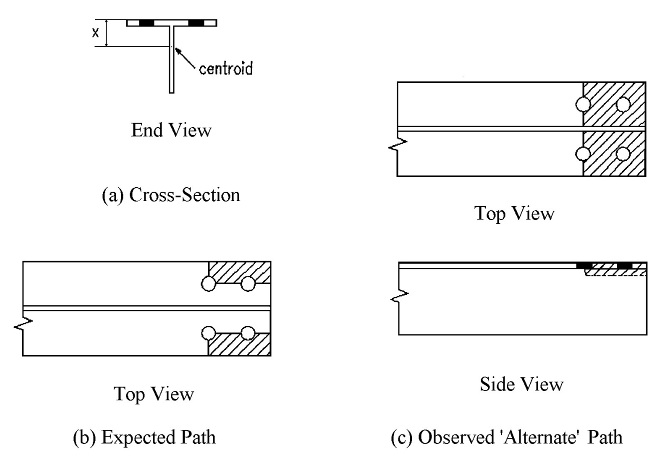


Fig. 2. Block shear paths in structural tees.

zone in the web of tees, at the lead bolt holes. The moments became so large that the resulting maximum compressive stresses, at the unloaded edge of the web, more than offset the uniform tensile stresses. Recent tests conducted at the University of Connecticut have clearly shown the presence of compressive stresses. In fact, local buckling of the web has been observed prior to the fracture of many of the specimens tested. It was also demonstrated that the moments are not just the load multiplied by the eccentricity (Epstein and McGinnis, 2000).

Finite element analyses have been shown to be an excellent method for investigating tension failure phenomena (Ricles and Yura, 1983; Epstein and Chamarajanagar, 1996; Epstein and McGinnis, 2000). Finite element analyses have accurately predicted failure patterns and relative failure loads. D Aiuto (1999) used finite element analyses together with conventional structural theory to find actual moments present in an eccentrically loaded structural tee. If a member is acted upon by a force, P, that is applied with an eccentricity, e, equilibrium dictates that the moment along a simply supported member is equal to P multiplied by e, as shown in Figure 3. For a connection of length that is fixed against rotation, Figure 4 shows that the resulting moment is not merely Pe, but may be approximated by

$$M = Pe - R\ell \tag{5}$$

where R is the reaction at the inner and outer bolt holes. The moment, therefore, requires knowledge of this reaction.

In order to determine the reactions, R, as a first approximation consider the elastic displacements shown in Figure 5. From elementary mechanics of materials theory

$$\delta_M = \frac{M_a \ell}{2EI}(L - \ell) \tag{6a}$$

$$\delta_R = \frac{R\ell^2}{6EI}(3L - 4\ell) \tag{6b}$$

where M_a is the applied moment equal to Pe (as shown in Figure 3), E is the modulus of elasticity, I is the moment of inertia about the axis of bending, ℓ is the connection length, and L is the overall length of the member.

If the connection does not rotate, the two displacements in Equation 6 are equal. Thus,

$$\delta_M = \delta_R \tag{7}$$

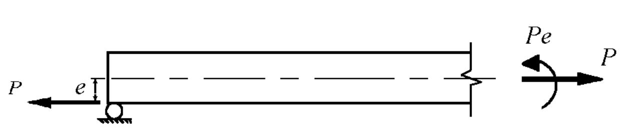


Fig. 3. Moments in an eccentrically loaded tension member.

and results in

$$R = \frac{\left(\dfrac{Pe\ell}{2EI}(L - \ell)\right)}{\left(\dfrac{\ell^2}{6EI}(3L - 4\ell)\right)} \tag{8}$$

The reaction in Equation 8 did not agree with finite element results for structural tees connected by their flanges (D Aiuto, 1999). It was found necessary to account for shear deformations in the vicinity of the connection. Further, connections may not be either fully restrained or simply supported, but may have a rotational stiffness. When these two factors (shear deformation and rotational stiffness) are included, Equation 8 becomes

$$R = \frac{\dfrac{Pe\ell}{2EI}\left(L - \dfrac{3}{2}\ell\right)}{\left(\left(\dfrac{\ell^2}{6EI}(3L - 4\ell)\right) + \left(\dfrac{\ell}{\lambda G t_w d}\right) + \left(\dfrac{\ell^2}{K_\theta}\right)\right)} \tag{9}$$

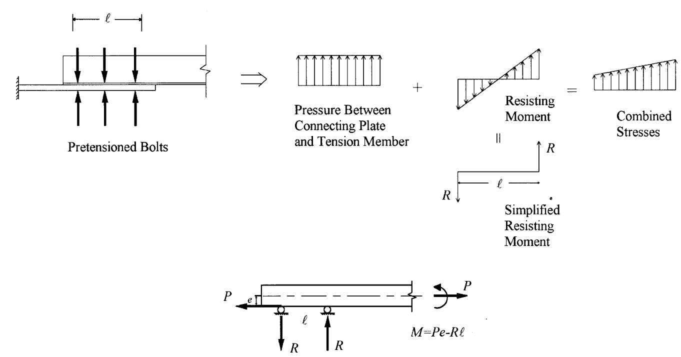


Fig. 4. Simplified resisting moment.

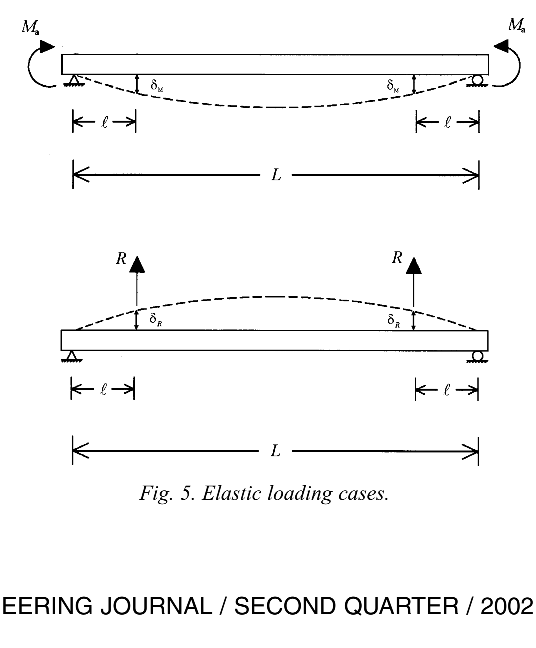


Fig. 5. Elastic loading cases.

where in addition to the parameters previously defined, G is the shear modulus, d is the depth of the tee, t_w is the thickness of the web, and K_θ is the rotational stiffness of the connection. The parameter λ accounts for the shear deformation associated with deep beam behavior. If the shearing stress on the web is constant between the reactions, $\lambda = 1$. Figure 6(a), however, shows that the shearing stresses tend to distribute in an arching pattern between the reactions. Assuming the 45 distributions in Figures 6(b) or 6(c) produces

$$\lambda = \frac{(\ell - d)}{\ell}, \text{ for } \ell \geq 2d \tag{10a}$$

$$\lambda = \frac{\ell}{4d}, \text{ for } \ell < 2d \tag{10b}$$

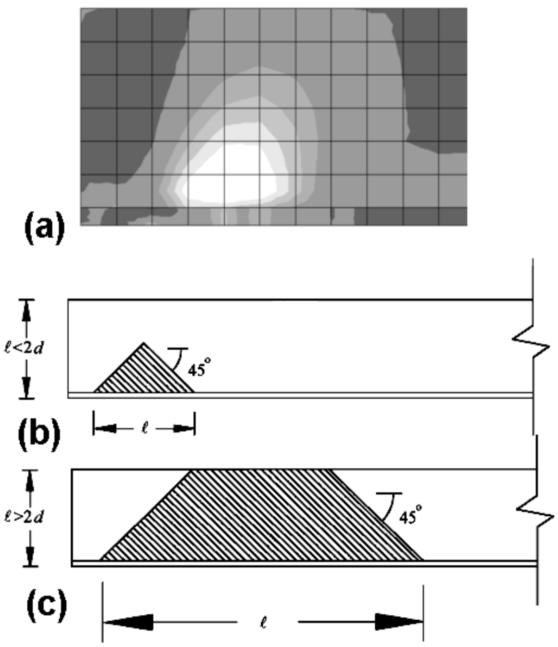


Fig. 6. Shear stress distribution at the connections and approximate arching patterns.

Figure 7 shows the comparison of the reactions for the theoretical and finite element results for connections fully fixed against rotation (K_θ becomes infinite). The nondimensional reaction is plotted versus nondimensional connection length. As can be seen, Equation 8 did not give satisfactory results for short connections, but adding uniform shear deformation over the connection length led to improvements. Then, using the more realistic estimate of shear deformation, a very satisfactory comparison was produced. The single Mesh 5 point in the figure resulted from a much finer mesh in the vicinity of the connection. This gave further confirmation to the appropriateness of the theory. Also, very small values of ℓ/L do not have practical applications. Thus, Equation 9 was shown to represent reactions very well.

When R, in Equation 9, is substituted into the moment equation, Equation 5, the result is

$$M = \left(1 - \frac{\frac{\ell^2}{2EI}\left(L - \frac{3}{2}\ell\right)}{\left(\left(\frac{\ell^2}{6EI}(3L - 4\ell)\right) + \left(\frac{\ell}{\lambda G t_w d}\right) + \left(\frac{\ell^2}{K_\theta}\right)\right)} \right) Pe \tag{11}$$

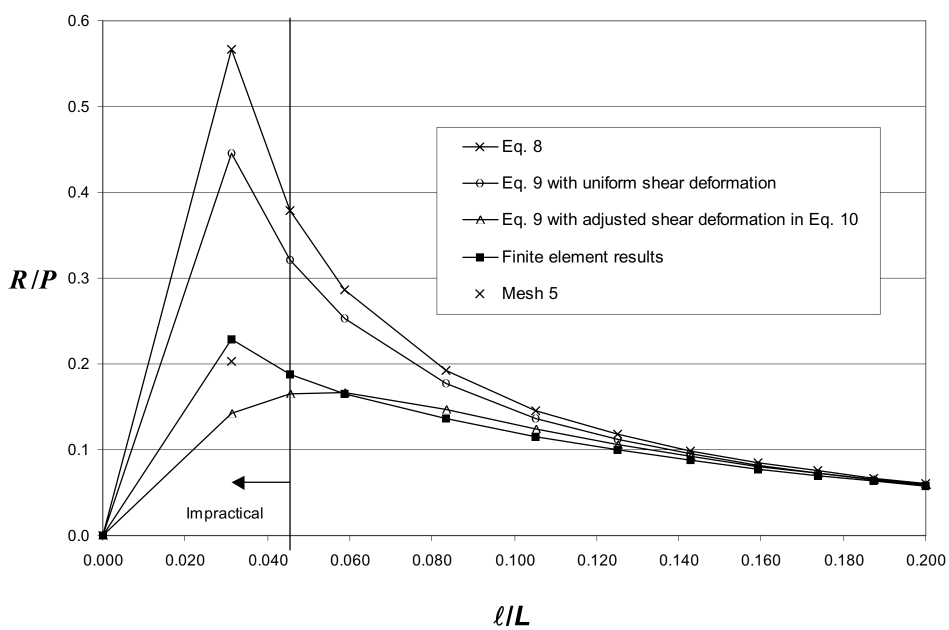


Fig. 7. Comparison of analytic and finite element nondimensional reactions versus nondimensional connection lengths.

or,

$$M = \beta P e \tag{12}$$

where β is the term in the large brackets in Equation 11.

Once the moment is known, the capacity of an eccentrically loaded tension member does not have to be found with the empirical shear lag reduction factor. Instead, existing specification treatments for the interaction of flexure and axial force (tension in this case) can be used. The LRFD specification uses, for doubly and singly symmetric shapes,

$$\frac{P_u}{\phi_t P_n} + \frac{8}{9}\left(\frac{M_u}{\phi_b M_n}\right) \le 1.0 \text{ when } \frac{P_u}{\phi_t P_n} \ge 0.2 \tag{13}$$

where P_u is the tensile strength, M_u is the flexural strength, ϕ_b is the flexural resistance factor equal to 0.90, and since this is a tension connection and there is no lateral instability, M_n (for compact sections) is the flexural design strength $= M_p = F_y Z$, in which M_p is the plastic moment and Z is the plastic section modulus. The other LRFD interaction equation (for $P_u/(\phi_t P_n) < 0.2$) is not appropriate for this application. If there is no moment present, since

$$P_u \le \phi P_n = \phi_t F_u A_e = \phi_t F_u U A_n \tag{14}$$

Equation 13 would produce

$$\frac{P_u}{\phi_t P_n'} \le U \tag{15}$$

where $P_n' = F_u A_n$. In effect, U is the efficiency of the connection due to the presence of shear lag. To see how the efficiency changes as a result of considering the moment given by Equation 12, simply substitute Equation 14 into Equation 13 and redefine efficiency with the symbol U_L. The result is

$$\frac{P_u}{\phi_t P_n'} \le \left(\frac{1}{1+\left(\frac{8}{9}\right)\left(\frac{\phi_t}{\phi_b}\right)\left(\frac{F_u}{F_y}\right)\left(\frac{eA_n}{Z}\right)\beta} \right) = U \equiv U_L \tag{16}$$

For ASD, the efficiency of a connection with shear lag can be similarly redefined by U_A. Considering the moment, given in Equation 12, and the ASD interaction equation, the result is (D Aiuto, 1999)

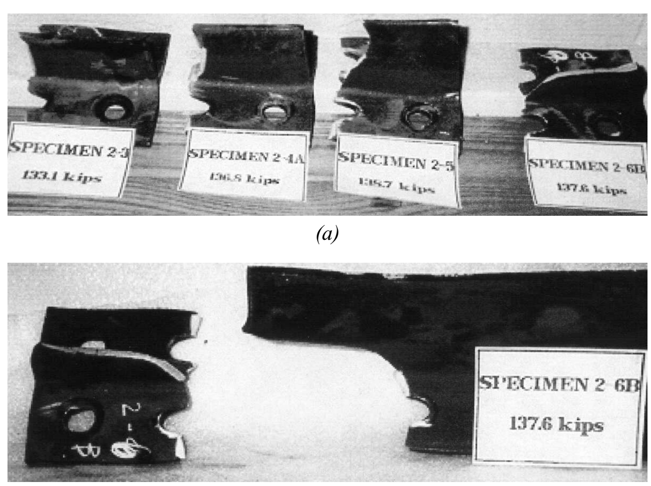
$$\frac{f_a}{0.50 F_u} \le \left(\frac{1}{1+\left(\frac{0.50}{0.66}\right)\left(\frac{F_u}{F_y}\right)\left(\frac{eA_n}{S}\right)\beta} \right) = U \equiv U_A \tag{17}$$

where f_a is allowable stress and S is the section modulus. For this application, $S = I/e$, since the eccentricity to the tension side is where the interaction needs to be considered.

TEST RESULTS AND SPECIFICATION TREATMENT

Equation 16 represents a new reduction factor that is based on the moments created by the eccentricity of the load and the bending and tensile capacities of the section. In other words, U_L serves a similar function to the empirical shear lag factor, U, currently used for eccentric tension connections. The shear lag factor was originally introduced to account for cross sections that had insufficient shear stiffness to develop a condition of ultimate stress over the entire net section area. Since shear is rarely present without bending, one can logically suggest that shear lag is also related to bending. In fact, both shear and bending are recognized in the empirical $[1 - x/\ell]$ because x is a measure of bending and both x and ℓ are related to shear stiffness. Logically, the shear area (depth and the width of the web) should also be important factors in the amount of shear lag present. Equation 11 does, indeed, incorporate actual shear stiffness.

Recently conducted tests (D Aiuto, 1999; Stamberg, 2000) were designed to have failures transition from net section to block shear as the parameters of connection length and length of the unconnected leg (and, therefore, connection eccentricity) were varied. Figure 8(a) shows a typical series of the failed ends of specimens depicting (left to right) failures that transitioned from net section to block shear (alternate) as the depth of the unconnected web increased. Both parts of the specimen in Figure 8(a) that failed in alternate block shear are shown in Figure 8(b).



(a)

(b)

Fig. 8. Failures from net section to alternate block shear.

Block Shear

Because there is not a uniform tension field, the eccentricity of the load appears to cause premature failures of the tension plane, when compared to coped beams. Previous investigations of block shear failure in angles (Epstein, 1992; Gross, Orbison, and Ziemian, 1995; Cunningham, Orbison, and Ziemian, 1995) have demonstrated that the use of Equations 4(a) and 4(b), without the recently incorporated (AISC, 1999) additional restrictions, was not appropriate. It was shown (Epstein, 1992) that including the shear lag reduction factor, U, in the tension terms of block shear led to significant improvements. This was done before the 1999 Specification was introduced. The use of the 1999 Specification helps for some connections, but perhaps the incorporation of eccentricity effects, such as contained in U_L, may be more appropriate.

More than Block Shear

When D Aiuto and Stamberg (D Aiuto, 1999; Stamberg, 2000) examined specimens that only failed in the net section, it became apparent that the use of U_L produced more appropriate safety factors over wider ranges of connection geometries for these net section failures. In all there were 50 tests conducted. The 36 tests that had net section failures are shown in Figure 9. All tests had ends restrained against rotation. In this figure, *PF* is the so-called professional factor. *PF* values are obtained by calculating the test failure load divided by the nominal specification strength and they are shown as crosses in this figure. The circles in this figure are for the same tests, same values of $[1 - x/\ell]$, but use the newly derived factor U_L in place of U.

Numerical Example

One of the tests was for a standard WT5×6. The specimen failed at a load of 83.0 kips. The material properties were F_y = 58.3 ksi and F_u = 77.5 ksi. A WT5 × 6 has the following properties: A_g = 1.77 in.^{2}, d = 4.935 in., t_w = 0.19 in., b_f = 3.96 in., t_f = 0.21 in., $e = y$ = 1.36 in., I = 4.35 in.^{4}, and Z = 2.50 in.^{3} The connection length, ℓ, was 3 in. and the overall length, L, was 50 in. This gives $U = 1 \quad 1.36/3 = 0.547$.

The connection had four ¾-in. bolts connected to the flange so that two full holes were taken from the cross section, similar to the connection labeled 2-3 in Figure 8(a). Thus, the net area is 1.40 in.^{2} In finding the strength of this connection, based on Equation 2, the effective area needs to include the shear lag factor, U. Values for the calculated U can get unrealistically small and designers would likely opt for the values given in the commentary. That approach (U = the larger of $[1 - x/\ell]$ or the appropriate commentary value of 0.75 or 0.85) was done for all of the results reported in this paper. In this case, U was taken as 0.75. Substituting into Equation 2 therefore gives ϕP_n = 61.1 kips. The strengths predicted from other failure modes are all higher and the numbers are not reproduced here. Thus, PF = $\phi P_{test}/\phi P_n$ = 0.75(83.0)/61.1 = 1.018. The corresponding data point (**x**) is shown in Figure 9 {0.547, 1.02}.

To see the effect that the use of the U_L factor has on this data point, β must first be found using Equations 11 and 12. Aside from the values already given, the usual values for E

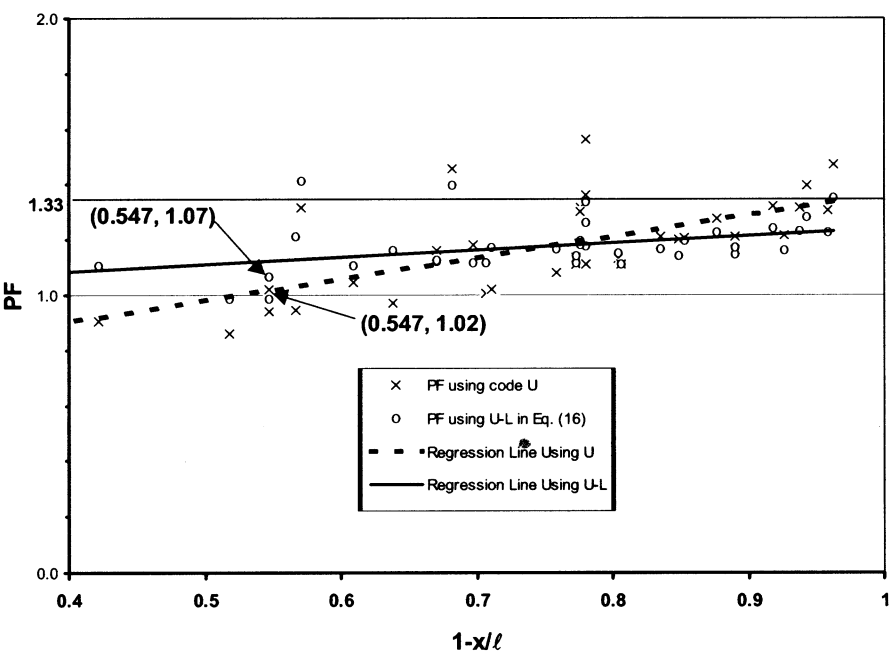


Fig. 9. The professional factor, PF, *using* U_L *versus using* U.

and G are used, 29,000 and 11,200 ksi, respectively, and $\lambda = 0.152$ is calculated from, in this case, Equation 10(b). This gives $\beta = 0.539$. Then, substituting into Equation 16 gives $U_L = 0.7118$. Therefore, PF, if U_L is substituted for the U of 0.75, is

$$PF = 1.018\ (0.75) / 0.7118 = 1.073$$

The corresponding data point (o) is shown in Figure 9 {0.547, 1.07}.

Results

The least-square linear regression line for the U data points (x) is shown in Figure 9 as a dashed line. As can be seen, many tests produced PF values less than 1.0, and when this occurs design specifications need to be seriously questioned. PF should ideally remain fairly constant at 1.33 (which is 1.0 divided by the ϕ factor for connections, 0.75). The trend in the test results clearly shows that as $[1 - x/\ell]$ decreases, so does the professional factor, PF. So, with decreasing connection length or with increasing eccentricity and, therefore, moment, the present shear lag reduction in nominal specification strength appears to be insufficient. It is exactly for the conditions of short connection length or large eccentricity, however, which the shear deformations over the length of the connection become increasingly significant, and these are included in Equation 16.

The solid regression line in Figure 9 is the least-square fit of the U_L data points (o). As can be seen, the slope of the regression line is more appropriate and the magnitude of the U_L line is more appropriate for the lower values of $[1 - x/\ell]$.

With high strength bolts, connections have become shorter. Many actual connections can produce values for $[1 - x/\ell]$ less than the 0.75 (or 0.85) minimum values permitted. The connections in Figure 9 that had PF less than one when using $[1 - x/\ell]$ for U all had less than 0.75. Therefore, at least a rethinking of these minimum specification values for U appears to be warranted.

Structural Rotational Restraint at the Connection

If a member is attached in a way that develops significant moments at the connection, the structural restraint is high. If the moment that develops at the net section is simply that caused by the eccentricity of the axial load, the structural restraint is zero. As the connection becomes less stiff, the moment resistance in Equation 5, $R\ell$, decreases, causing the moments to increase. If there is symmetry due to, for instance, back-to-back members, the rotational stiffness is infinite. Actual unsymmetrical connections will usually have rotational stiffness that greatly increase the moments when compared to the same connection where rotation is completely restrained. Figure 10 shows curves of β (recall that $M = \beta Pe$) versus non-dimensional connection length, ℓ/L, for varying connection rotational stiffnesses.

To get an idea about how important the end restraint could be, suppose that there is a 6-in. connection length for a 10-ft long member, thus giving $\ell/L = 0.05$. Figure 10 shows that the moment in that member would approximately double for a simple versus a fixed ended connection.

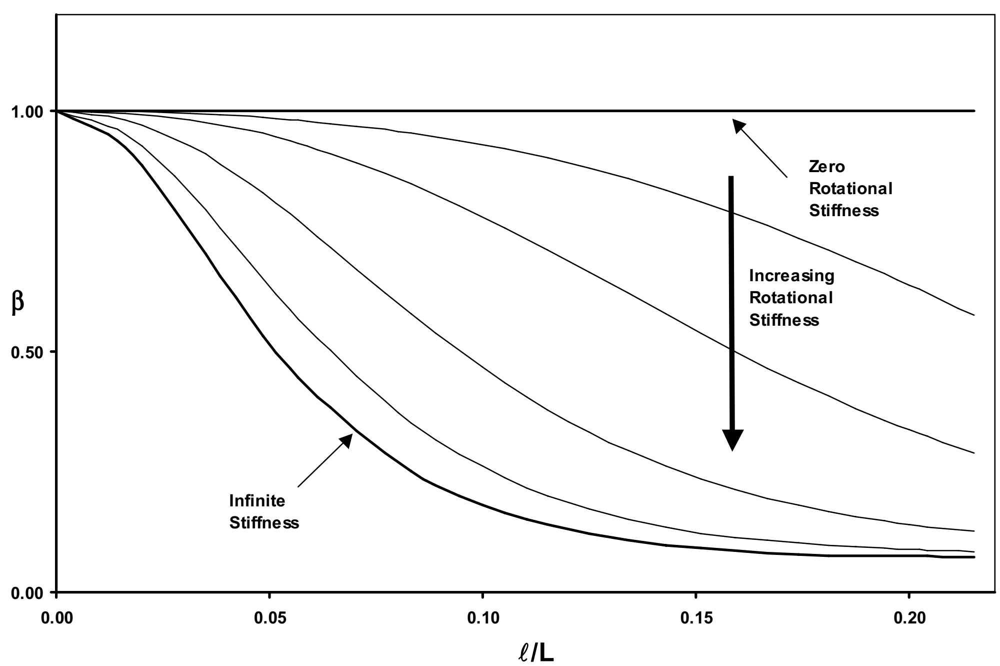


Fig. 10. β versus ℓ/L with variable rotational stiffness.

Assume, for example, that a connection with $\ell/L = 0.05$ produced unity, for a fixed ended connection, using the moment-axial interaction from Equation 13. Further assume that the moment only accounted for 25 percent of the interaction, a reasonable percentage. For the same connection that is free to rotate, the interaction equation would produce $0.75 + 0.25 \times 2 = 1.25$, a 25 percent overstress.

The variation of moments with connection rotational stiffness is an important factor that is not accounted for in current specification treatments. The present shear lag reduction factor, U, is only a function of the eccentricity, e, and the connection length, ℓ. Much of the work that led to the current empirical shear lag reduction factor is based on tests at the University of Illinois during the late 1950s and early 1960s (e.g., Munse and Chesson, 1963). The reported tests appear to have been for symmetrically placed members and, therefore, have connections that are infinitely stiff for rotation. Partially due to not having high-strength bolts at the time, the connections tested were longer than they probably would be today. The appropriateness of the empirical equation that resulted is, therefore, in question.

The tests conducted at the University of Connecticut were intended to limit end rotations. For these tests, steel cantilevered pallets that were 2-in. deep by 8-in. wide (4 in. at the grip of the testing machine) restrained the ends. One of the early tests actually broke the pallet before the specimen failed due to the moment caused by the eccentricity of the load. The connections are subsequently braced to minimize the end rotation. The authors strongly recommend that future testing of eccentric tension connections should incorporate the effects of end rotational stiffness. Until then, unless connections are symmetric or are essentially restrained against rotation, thus eliminating end rotation, a conservative approach would be to consider the moment in the member as just the load times the eccentricity ($\beta = 1.0$) and use the interaction equations already in the specifications.

CONCLUSIONS

For as long as there have been codes and specifications to govern the design of structural steel, the need to reduce the design strength of a tension member has been recognized when some, but not all, of the cross-sectional elements of a member are included in its connection. The reduction has been attributed to shear lag. Over the years, hundreds of tests at the University of Connecticut have shown that the eccentricity of the load, with the resulting moment, appears to account for the diminished strength.

Present shear lag factors are based on tests that were performed forty years ago. These tests were for connections that, in essence, did not rotate and were relatively long. Many of the tests of the tees, reported in this paper, have produced inappropriate professional factors. Further, the trend in the data appears to show that, especially for shorter connections or connections with larger eccentricities or both, the presently permitted minimum shear lag reduction factor is not sufficient. Additional analytic work as well as tests (especially varying stiffness, other sections and shorter connections) will probably lead to a more appropriate design treatment for tension members with eccentricity. At present, it appears that any such treatment should include the actual moments present.

This paper has presented a new approach to the design of tension members that takes the moment resulting from loading eccentricity as well as structural rotational restraint into account. The moments were found for structural tees. These moments can be used with existing interaction equations in the specifications.

It is far from the intent of this paper to suggest the incorporation of the presented equations for moment into any specification. First, these equations have only been shown for tees and suggesting a design of a particular type of tension member would not be appropriate. Second, without extensive design aids, the equations are too involved. The intent of presenting this information is to show a new approach to possible future tests and recommendations concerning connection efficiency.

For the present, however, it is reasonable to recommend that at least a reduction be made for the minimum $[1 - x/\ell]$ values currently permitted. Additional investigation of eccentric tension members is warranted. Further, future investigators of tension connections should undertake tests and analytic work that incorporate end rotational effects.

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