DISCUSSION

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

Paper by HOWARD L. EPSTEIN and CHRISTOPHER L. D'AIUTO
(Second Quarter, 2002)

Discussion by GILBERT Y. GRONDIN

The discusser read with interest the paper by Epstein and D'Aiuto and wishes to comment on some of the aspects of block shear, shear lag, load eccentricity, and load and resistance factor design raised in this paper.

The authors propose an approach that attempts to account for shear lag in structural tees and angles in tension by using an interaction equation that is intended to account for the combined action of tension plus bending stresses. Although the paper is intended to discuss the effect of shear lag, there seems to be some confusion between shear lag and block shear. The authors advocate the use of a shear lag factor when calculating the block shear resistance (page 96 of the paper) and the first author and his co-workers have, over the past decade, advocated the use of a shear lag factor to reduce the predicted block shear capacity (Epstein, 1992, 1996; Epstein and McGinnis, 2000). In the view of the writer, Epstein and co-workers seem to have overlooked some fundamental differences between the two failure modes.

Shear lag occurs in the connection area of a member loaded in tension when only part of the cross-section is connected. To use the example provided in the paper, when a structural tee is connected only by the flange, the stress carried by the stem of the tee at some distance from the connection has to make its way into the flange as one moves notionally towards the connection. This stress transfer from the stem towards the connected flange is done through longitudinal shear transfer, and this is termed shear lag effect. The result is that a portion of the cross-section (the stem, in this case) is only lightly stressed at the connection and, therefore, part of the cross-section is ineffective at that location. This has the effect of reducing the net area at the connection to a "net effective" area. The manifestation of the shear lag effect is premature rupture of the cross-section and it is important to note that rupture is through the entire cross-section. The situation indicated by the authors where the stem only would be connected is not a sound design since only a small portion of the cross-section is connected and significant eccentricity is created. It is believed that this case is rather uncommon in practice.

The writer questions how the authors would deal with the common case of wide flange sections either connected by both flanges or by the web only. There is no eccentricity of load in these situations, and, in the absence of eccentricity, the authors seem to indicate that shear lag would not be a problem. It is common knowledge; however, that shear lag indeed exists in the cases just described.

Block shear, on the other hand, is failure by shear and tension of the part of a cross-section that is fully connected. In the case of bolted connections, block shear failure involves the tearing of a block of material defining a failure path that generally joins the perimeter of bolt holes. Figure 1 depicts the connection area in a wide flange section at two different stages of failure, namely, immediately after fracture of the tension face (Figure 1a) and at a later stage-after the shear surfaces have failed (Figure 1b). Typically, the peak capacity of the connection is reached just prior to tension fracture. If a free body diagram of the block of material is drawn just prior to tension fracture (Figure 2), it can be seen that both normal stresses and shear stresses act on the block of material. It is reasonable to assume that the tension stress will reach a value equal to the ultimate tensile strength just prior to or at the time of fracture. Because of symmetry in the case illustrated, the stress on the tension face is uniform and equal to the tensile strength of the material. The stresses on the shear areas and the size of the shear

Gilbert Y. Grondin, department of civil and environmental engineering, University of Alberta, Edmonton, Alberta, Canada.

areas, however, are not well defined. Experimental work from different sources indicates that, although the shear area involved is not well defined, good predictions of the ultimate capacity of such connections can be made (Kulak and Grondin, 2001). It is important to note that the normal and shear stresses associated with block shear are not affected by the non-uniform normal stresses that will exist if shear lag is also present. Hence, it is inappropriate to multiply the block shear capacity, however calculated, by a shear lag factor. The shear lag factor should be used where it was always intended to be used, namely, when calculating the capacity of a tension member when failure occurs at the net section and where the fracture surface goes through the full cross-section. Shear lag does not manifest itself in block shear failure.

An aspect of block shear failure that is rightfully brought up in the paper is that of eccentric loading. However, one must distinguish between in-plane and out-of-plane eccentricity, and this is not done in the paper. In-plane eccentricity is clearly manifested in block shear failure of coped beams. As shown in Figure 3, the resultant resisting force and the resultant applied force are not co-linear. The lack of

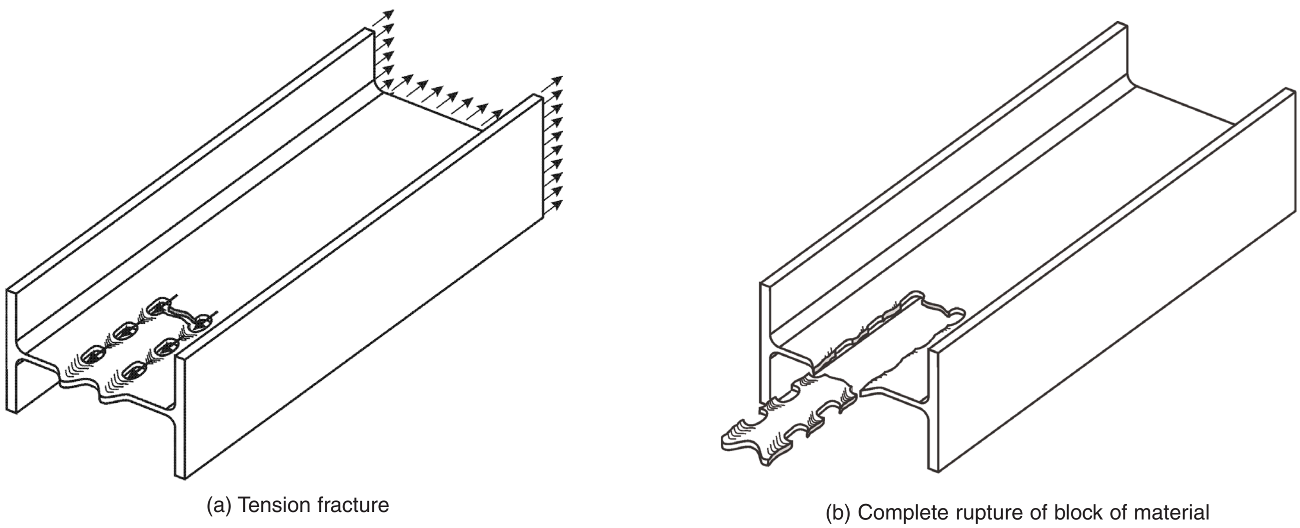

Fig. 1. Stages of block shear failure.

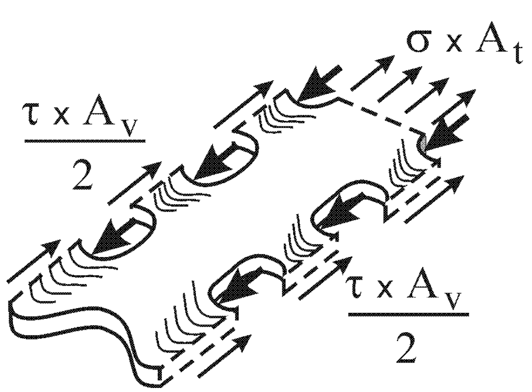

Fig. 2. Free body diagram of material block just prior to tension fracture.

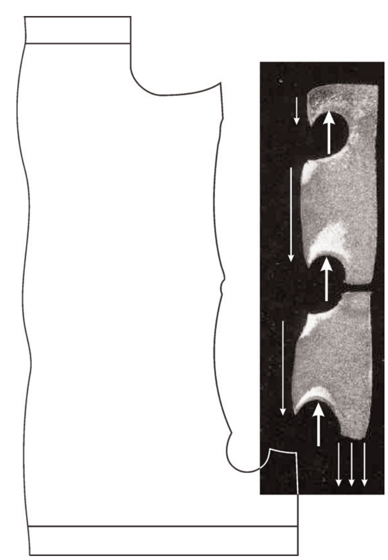

Fig. 3. Block Shear Failure of Coped Beam.

symmetry of the block shear failure surface gives rise to eccentricity in the plane of the block of material that is involved in the block shear failure. This, in turn, results in a non-uniform tensile stress on the tension portion of the block shear failure surface. The non-uniform stress on that surface can be accounted for by adding a reduction factor on the tension contribution (Ricles and Yura, 1983; Kulak and Grondin, 2001). This approach provides a practical and simple design approach that provides an adequate level of safety (Franchuk, Driver, and Grondin, 2002; Driver, Grondin, and Kulak, 2004).

In the case of structural tees connected at the flange, the eccentricity lies out of the plane of the block shear failure plane. This has the effect of increasing the tensile strain in the flange of the tee, which, in turn, may affect the block shear capacity. This is a normal stress effect from the secondary moment created in the member, not a shear lag effect. The eccentricity of the applied load also affects the stress distribution in the cross-section, which could also influence the shear lag effect. Although the problem of shear lag was recognized decades ago by Munse and Chesson (1963), the equation that the authors proposed to account for shear lag is not universally applicable since it was derived based on test results. The authors have pointed the way to what may become a better approach to assess shear lag effects in some cross-sections. Some aspects of the approach proposed by the authors are discussed below.

The authors mention that the finite element method presented by Epstein and Thacker (1991), Epstein and Chamarajanagar (1996), and Epstein and McGinnis (2000) was able to predict test results accurately. It should be noted that all these analyses suffer from the same problems: insufficiently fine mesh to reach convergence of strains, and an incorrect failure criterion.

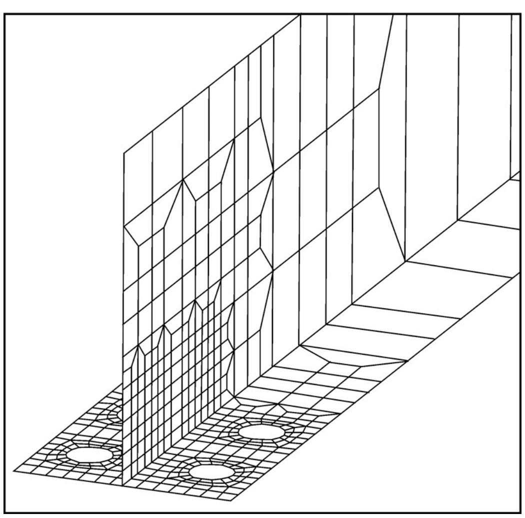

Fig. 4. Finite element mesh by Epstein and McGinnis (2000).

In the finite element work on angles in tension by Epstein and Thacker (1991), it is the writer's opinion that the models investigated were too short to capture the true behavior of bolted angles in tension—the authors of the work on angles reported that in order to predict the test capacity accurately only the connected leg had to be loaded. This clearly indicates that the models were not able to capture the shear lag behavior correctly.

In the finite element analyses conducted by Epstein and co-workers, the finite element mesh seems to be too coarse to have reached convergence of strains. It is indicated by Epstein and co-workers (Epstein and Chamarajanagar, 1996; Epstein and Thacker, 1991; Epstein and McGinnis, 2000) that convergence was sought on the stresses. Recent work by Huns, Grondin, and Driver (2002) has indicated that a mesh much finer than the ones used by Epstein and co-workers was needed to get convergence of strains in the inelastic range of material response. It should be noted that when an inelastic analysis is performed, convergence should be sought for strains, not for stresses. This is particularly important when strain is used as a failure criterion, as was the case in the work conducted by Epstein and co-workers. Figure 4 shows the finite element mesh used by Epstein and McGinnis (2000) and Figure 5 shows a converged mesh used by Huns and others (2002). In this latter work, it was found that convergence of stresses was reached with a fairly coarse mesh whereas strain convergence required a much finer mesh.

A strain of up to five times the yield strain was used as a failure criterion in these studies (Epstein and Chamarajanagar, 1996). For a Grade 50 steel, this corresponds to about 0.9 percent. This is an unrealistically small rupture strain for structural steel. The engineering strain at rupture measured over a 2 in. gauge length coupon is at least 20 to 25 percent for structural steels. To use strain as a failure criterion in a non-uniform strain field (in other words, the region where strain concentration occurs) the strain measure that should be used is the true strain. The true strain at rupture (which is the localized strain in the necking region of a tension coupon) can easily be estimated from a tension coupon test by the following expression (Dowling, 1999):

$$\varepsilon = \ln\left(\frac{A_o}{A}\right) \tag{1}$$

where A_o is the initial cross-sectional area and A is the final cross-sectional area measured at the fracture location. Using Equation 1 above, a reasonable estimate of true strain at rupture is at least 100 percent, which is about 110 times the value used in the work of Epstein and co-workers. Khoo, Cheng, and Hrudey (2001) have proposed values from 90 to 120 percent for steels of structural and pressure vessel grades.

It is believed that the finite element analyses of Epstein and co-workers were able to predict the test results with the high level of accuracy claimed by the authors simply because the combination of a coarse mesh with a much lower rupture strain criterion would result in a situation where one effect offsets the other. The coarse mesh adopted for the investigation of angles and tees would predict a much smaller strain than the exact strain. The critical strain used in the authors' analysis was determined so that the analysis would predict the test results well. In the view of the writer, the models are therefore not reliable and the results of analysis using such models cannot be used with confidence to expand the range of the tested parameters. The writer agrees, however, with the statement of the authors that the finite element method can be used to predict accurately the strength and behavior of tension members in the connection area. However, a mesh that has reached convergence and a valid failure criterion must be used.

The entire derivations that lead to the authors' Equation 11 are based on several simplifying assumptions, including elastic behavior of the tension member, the point of application of the axial load on the cross-section, and the resisting moment at the connection. The assumption of the elastic behavior affects directly the shear deformation and rotational stiffness of the connections. The point of application of the axial force on the cross-section away from the end connection is a function of the member length. In short members the tension force would be transferred mainly by the flange of the tee section and the stress in the stem would be small, thus shifting the resultant force P towards the flange. If the member is long, the stress in the cross-section half way between the end connections would be uniform and the resultant force would be at the geometric centroid of the member. The authors should comment on the implication of the assumptions made in the proposed model. Without a critical assessment of these assumptions, the proposed equation is of little use.

Equation 13 represents one segment of a two straight-line interaction curve. The authors comment that the second segment of the interaction curve (for $P_u / \phi P_n < 2.0$) is not appropriate for this application. There is no apparent reason why the second part of the interaction equation is "not appropriate." Perhaps what the authors should have said is that the second equation will not apply in most cases because the bending moment due to eccentricity of the load will be small in most situations. This does not make the second segment of the interaction curve inappropriate.

The moment calculated in Equation 11 is used with the beam-column interaction equation to determine the capacity of a tension member. The derivations starting with Equation 13 make a distinction between the LRFD and ASD interaction equations. Looking at Equation 16, the shear lag coefficient becomes a function of the resistance factors. The authors have taken a probabilistic approach and treated it as a deterministic approach where the resistance factors have been reduced to simple constants. This is not a load and resistance factor design approach. Furthermore, the authors make the statement (page 97 in the Results section) that when the professional factor is less than 1.0 "design specifications need to be seriously questioned." It is also claimed that the professional factor should ideally be 1.33. The objective of a prediction model is to predict accurately the capacity and behavior of the member. This implies that the

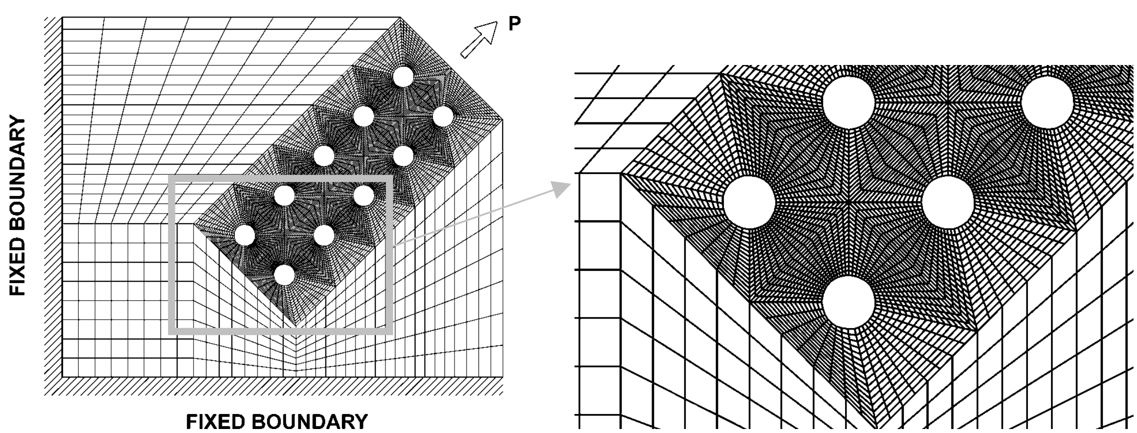

Fig. 5. Typical converged finite element mesh used by Huns and others (2002).

mean test-to-predicted ratio should be close to 1.0 and the coefficient of variation should be small. LRFD does not advocate the use of poor prediction models. Once a satisfactory model is obtained, the appropriate resistance factor that provides an acceptable level of safety can be obtained through a reliability analysis. A description of this procedure has been presented in the current edition of the SSRC Guide (Galambos, 1998).

The authors have plotted a so-called professional factor versus $1-x/l$ (Figure 9 of Epstein and D'Auito). It is the writer's opinion that the term professional factor is not appropriate here. A professional factor is the ratio of a measured capacity (typically a test result) to the predicted capacity (Galambos, 1998). It is generally accepted that a professional factor is intended to be a measure of the ability of the prediction model to predict the test results. Consequently, the capacity must be calculated using measured material properties and dimensions and, of course, would not contain any resistance factor. It is noted that Equation 16 contains all the resistance factors used in the LRFD interaction equation. Despite these shortcomings of the procedure used by the authors, Figure 9 shows that the very tedious approach proposed in this paper does not appear to be better than the $1-x/l$ approach. It seems that the small improvement in the predicted test results does not justify the complexity of the approach proposed by the authors. It would be interesting to see what value of resistance factor would be required from each model to obtain an appropriate safety index. In the selection of the best design approach, one should also consider the simplicity of the capacity prediction model.

It is also noted that the numerical example makes use of a moment capacity for a WT5×6 equal to the plastic moment capacity. This moment capacity is not realistic for a cross section that does not satisfy the requirement of a noncompact section. Thus, formulation of the proposed interaction equation does not consider the possibility of failure by local buckling. This should be addressed in the formulation, or the rationale for using the plastic moment capacity for sections that do not satisfy the requirements for a compact section should be clearly explained.

The writer agrees that the effect of load eccentricity on the capacity of tension members should be investigated. Tests on coped beams have shown that eccentricity in the plane of the block shear failure affects the block shear capacity. A simple equation that provides an acceptable level of safety has been proposed by Franchuk and others (2002) and Driver and others (2004) for coped beams with one and two lines of bolts, gusset plates, angles, and tees. Eccentricity in the out-of-plane direction, such as found in structural tees, can also affect the block shear resistance since the out-of-plane eccentricity can increase the tensile and shear stresses in the connection area. The effect of this eccentricity should be evaluated for standard tees. Epstein and co-workers have accumulated useful test data to help achieve this task. This data should be assessed critically to evaluate the effect of the out-of-plane eccentricity on block shear capacity and on shear lag effect on net section rupture. Although angles loaded 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. It is also believed that the effect of shear lag on the capacity of bolted angles in tension is adequately treated in current North American design standards.

I think that the authors' failed sections should be questioned. They appear to the ordinary reader to support the failure mode contentions put forward by the authors. I suspect they simply represent the condition wherein the parts have been physically separated. As we know, this does not usually reflect the condition at the ultimate load level.

REFERENCES

Dowling, N.E. (1999), *Mechanical Behavior of Materials—Engineering Methods for Deformation, Fracture, and Fatigue*, Second edition, Prentice Hall, New Jersey.

Driver, R.G., Grondin, G.Y., and Kulak, G.L. (2004), "A Unified Approach to Design for Block Shear", *Connections in Steel Structures V: Innovative Steel Connections*, ECCS-AISC Workshop Amsterdam, June 3-4.

Epstein, H.I. (1992), "An Experimental Study of Block Shear Failure of Angles in Tension," *Engineering Journal*, AISC, Second Quarter, pp. 75-84.

Epstein, H.I. (1996), "Effects of the Latest LRFD Block Shear Code Change," *Engineering Journal*, AISC, First Quarter, pp. 30-33.

Epstein, H.I. and Chamarajanagar (1996), "Finite Element Studies for Correlation with Block Shear Tests," *Computers & Structures*, Vol. 61, No. 5, pp. 967-974.

Epstein, H.I. and McGinnis, M.J. (2000), "Finite Element Modeling of Block Shear in Structural Tees," *Computers and Structures*, Vol. 77, No. 5, pp. 571-582.

Epstein, H.I. and Thacker, B.H. (1991), "The Effect of Bolt Stagger for Block Shear Tension Failures in Angles," *Computers & Structures*, Vol. 39, No. 5, pp. 571-576.

Franchuk, C., Driver, R.G., and Grondin, G.Y. (2002), "Block Shear Behaviour of Coped Steel Beams," *Structural engineering Report 244*, Department of Civil & Environmental Engineering, University of Alberta, Edmonton, Canada.

Galambos, T.V., editor (1998), *Guide to Stability Design Criteria for Metal Structures*, 5th Edition, John Wiley & Sons, New York.

Huns, B.B.S, Grondin, G.Y., and Driver, R.G. (2002), "Block Shear Failure of Bolted Gusset Plates," 4th Structural Specialty Conference of the Canadian Society for Civil Engineering, Montréal, June 5-8.

Khoo, H.A., Cheng, J.J.R., and Hrudey, T.M. (2000), "Ductile Fracture of Steel," *Structural Engineering Report No. 232*, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada.

Kulak, G.L. and G. Y. Grondin (2001), "AISC LRFD Rules for Block Shear—A Review," *Engineering Journal*, AISC, Vol. 38, No. 4, pp. 199-203.

Munse, W.H. and Chesson, E. (1963), "Riveted and Bolted Joints: Net Section Design," *Journal of the Structural Division*, ASCE, Vol. 89, ST1, Part 1, February.

Ricles, J.M. and Yura, J.A. (1983), "Strength of Double-Row Bolted-Web Connections," *Journal of the Structural Division,* ASCE, Vol. 109, ST1, pp. 126-142.