CLOSURE

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
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Closure by HOWARD L. EPSTEIN

The discussion by Prof. Grondin addresses many aspects of our paper but also deals with previous research conducted at the University of Connecticut by the author and his graduate students. This lengthy discussion contains many statements indicating that either the intent of the paper was not fully understood or additional clarification may aid the discusser. So, in as succinct a closure as practical, the salient points of the discusser are now addressed.

SHEAR LAG VERSUS BLOCK SHEAR

The first several paragraphs of the discussion deal with the "confusion" and "overlooked … fundamental differences" between shear lag and block shear. There is no confusion on the authors' part. The reason the author became involved with block shear was the discovery of failed angles resulting from the collapse of the Hartford Civic Center roof. It was apparent that the block shear failures of angles should not be treated the same as those in coped beams because of the out of plane eccentricity. The original research (Epstein, 1992) and many works that followed by the author and other investigators showed that block shear failure loads were influenced by this eccentricity. The original study varied the length of the unconnected leg for many different connection geometries to clearly show this effect.

The discusser states that, "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." If the discusser came to that conclusion from reading the collective research of the author, then it needs to be emphasized here that nowhere has the author ever stated that shear lag is the problem in block shear. Instead, quoting from the original work with angles (Epstein, 1992), "The proposed inclusion of the shear lag reduction coefficient (U) for the tension area appears to produce appropriate results."

Call it what you will: Shear lag or reduced block shear capacity or, as was done in the paper of this closure, moment-axial interaction. For angles, due to the out of plane eccentricity of the load, "shear lag" magnifies the average tensile stresses causing failure at a load that is less than the ultimate strength times the net area. As has been shown by this and other researchers, block shear capacity also is decreased by this same eccentricity of loading. This same conclusion was also reached for tees (Epstein and Stamberg, 2002). While not citing this work on tees specifically, the discusser states that, "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." Exactly the argument that has been made for tees as well as angles! Why then does the discusser go on to state that, "This is a normal stress effect from the secondary moment created in the member, not a shear lag effect," a conclusion that has never been purported. Interestingly, the latest proposed treatment for block shear by AISC seems to have also recognized the need for reducing the previous block shear capacities by the inclusion of a "shear lag-like" reduction factor, U_{bs}, to be applied to the tension area (AISC, 2003).

AISC's latest draft specification (AISC, 2003) also puts more stringent requirements on the shear lag coefficient

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used in net tension capacity equations. The discusser states that, "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." Apparently the AISC draft standard writers do not agree.

The discusser next talks about tees connected only by the stem and states that this is "not a sound design since only a small portion of the cross-section is connected and significant eccentricity is created." The author agrees with this statement along with the belief that this would be an uncommon (and might add unwise) design in practice. The discusser then asks, concerning block shear, "how the authors would deal with the common case of wide flange sections either connected by both flanges or by the web only." The authors' engineering judgment would be to treat the flange connected tee as AISC does for (here is that term again) shear lag. That is, the eccentricity is that of the half W (a tee). For web connections, there is (or should be) no eccentricity of load and, hence, no need for any reduction.

Finite Element Investigations

The author questions why this whole aspect of previous research (not the paper for which this is a closure) is brought up and discussed here. But since it is, most of the discusser's comments need to be refuted. The author and his graduate students have used finite element investigations as tools for studying the effect of various geometric parameters, period. Never has it ever been implied that failure loads could accurately be predicted. In fact, quoting from one of these papers (Epstein and McGinnis, 2000), "Normalizing both the plots in this way was done to see the relative trends and does not imply that the finite element studies can accurately predict actual failure loads." Similar quotes can be found in every other paper by the author dealing with finite element studies of tension connections. In all these studies, various failure criteria were employed and "trends" (parametric effects, predicted failure mode, relative failure loads, etc.) were found to be similar.

The discusser also questioned mesh refinement and shows (in his Figure 4) one of the meshes for tees that was employed by Epstein and McGinnis (2000). Once again, quoting from that paper concerning the meshing around the holes, "Patches consisting of 36, 52, 76, 132 and 244 elements were investigated." The paper goes on to justify the mesh chosen. The overall length of models was also questioned, but these were in agreement with test specimens that were shown to be sufficiently long to develop uniform axial stresses at mid-length. The discusser also questioned the strain values used in a previous work. The author is well aware of the ductility of steel, but once the strain at the failure initiation location reaches several times yield, the additional load to failure is minimal and the relative failure loads are virtually unchanged. Some recent work by one of the author's graduate students (Barrett, 2002 and submitted for publication) seems promising in more accurately predicting actual failure loads.

In dealing with finite elements for predicting failure load, there are several other factors not mentioned by the discusser that probably play a more important role than those he presented. For instance, the clamping action of the bolts and contact stresses that may be present probably are more important than mesh refinement. So, predicted failure loads in finite element analyses should always be questioned.

Moment-Axial Interaction in Tension Members

The discusser states that, "The authors have pointed the way to what may become a better approach to assess shear lag effects in some cross-sections," apparently referring to the shear lag factor that is theoretically derived from predicting the moment in a tension member and using AISC's interaction equations. In fact, something akin to this approach will now be required since the latest draft specification (AISC, 2003) now states that, "Members such as single angles, double angles and WT sections shall have connections proportioned such that U is equal to or greater than 0.60. Alternatively, a lesser value of U is permitted if these tension members are designed for the effect of eccentricity in accordance with H1.2 or H2.2", where H1.2 and 2.2 are the AISC moment-axial interaction equations. Thus, in trying to design or justify a connection having the combination of large eccentricity and short length (producing a low U value), a designer will be forced to somehow estimate the moments in eccentric tension connections. The authors believe that their paper presents a highly appropriate treatment for finding the moments in such connections.

The discusser questioned assumptions that were behind the derivation of Equation (11). Suffice it to say here that the authors, through finite element studies and test results, are quite satisfied with the equations presented. It was the main thrust of the paper to show that shear lag effects could reasonably be thought of in terms of the moment produced and if one can reasonably predict these moments, then interaction equations are highly appropriate. The authors gave one possible approach for finding these moments and then showed the equivalent shear lag (U) coefficient that would result. The approach is not, as the discusser stated, a "probabilistic approach … treated as a deterministic approach…" It is merely finding an equation for the moment (albeit elastic) and then substituting into existing interaction equations using the ratio of plastic to elastic section moduli when the plastic moment is required. AISC's interaction equation for $P_u / \phi P_n < 0.2$ was not used because the axial part of the interaction is dominant for these tension connections.

The Professional Factor

The discuser states that, "It is the writer's opinion that the term *professional factor* is not appropriate here." He is questioning how and where the ϕ factors should be employed in presenting results. The author considered this but it seems that no matter how results are presented (and it was done differently in earlier papers by the author) there are those who want them shown in other ways. The professional factor was used in this paper similarly to what has been found in several other recently published works. However, regardless of how the results are presented, the conclusions reached do not change.

Other Discussed Items

The discusser calls the reduction factor obtained from using a derived moment in the moment-interaction equations a "very tedious approach." The author agrees and in fact stated so in the conclusion to this paper saying that, "… without 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." Now it can be added that if the draft specification is adopted, even though "tedious", a designer faced with a connection having $U < 0.6$ will be required to use an interaction equation!

In questioning the moment capacity of the WT5×6, the discusser states "… formulation of the proposed interaction equation does not consider the possibility of failure by local buckling." Obviously, the discusser realizes, as was pointed out previously (Epstein and McGinnis, 2000) and verified by subsequent tests (Epstein and Stamberg, 2002), that buckling of the stem of the tees is possible due to bending compressive stresses more than offsetting axial tension stresses. The author has already stated that in such cases, "a limit on depth-to-width ratio for the web appears to be needed" (Epstein and Stamberg, 2002), as is done for other local buckling problems.

The discusser wrote that he "agrees that the effect of load eccentricity on the capacity of tension members should be investigated…. Epstein and co-workers have accumulated useful test data to help (in) achieving this task." So, if nothing else, we have given future researchers something useful. The discusser also states that, "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." The author is puzzled by this comment and, therefore, cannot respond.

Finally, the discusser claims to have shown a "unified approach" to dealing with block shear that provides a "practical and simple design approach" (Franchuk and others, 2002; Driver and others, 2004). The author is completely in favor of any approach that adequately accounts for the load eccentricity and is easily used by the design community. The Franchuk and others report and the Driver and others conference proceeding were not available when the discussed paper was submitted. The author would welcome the opportunity to review this work when it appears in a future (hopefully AISC) publication.

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