Discussion

Plastic Behavior of Eccentrically-Loaded Connections

Paper presented by C. L. SHERMER (April, 1971, issue)

Discussion by **G. L. KULAK**

The article by Shermer contains two serious misconceptions which the writer would like to discuss.

The first misconception, although not central to the main thrust of the presentation, lies in the discussion of non-uniform fastener stresses in centrally loaded connections. The statement that "When the connection begins to yield, however, the stresses become quite uniform" is factually incorrect. Indeed, just the opposite is true; as the highly stressed regions of the joint become inelastic, the unequal distribution of load among fasteners is accentuated. This unequal loading is a function of joint length, among other parameters. Since it is customary to assume that all fasteners carry an equal portion of the load, this means that the largest, and perhaps, therefore, the most important joints are being designed with the lowest factor of safety. That the resulting factor of safety is still apparently adequate is a reflection of the conservative values that have been assigned to allowable fastener stresses, whether rivets, high-strength bolts, or welds. The phenomenon described above is well documented, particularly for cases involving rivets or high-strength bolts, and the writer will not pursue it further here.[1, 2, 3]

The attainment of a yield point stress in shear for weld material, rivets, or high-strength bolts is a basic premise of the method proposed by Shermer. Although it is not clear whether or not this point was examined experimentally by the author, work by others has clearly established that neither rivets, nor high-strength bolts, nor fillet welds subjected to a shear-type loading exhibit any well-defined yield point.[4, 5, 6] Furthermore, the ultimate strength and deformation of fillet welds loaded in shear are known to be dependent upon the direction of the force applied to the weld. The results of tests on ¼-in. fillet welds made using E60XX electrodes, as shown in Fig. 1, indicate this very clearly.[6] The figure also serves to illustrate the point that no well-defined shear yield is present. Solutions for the ultimate strength of eccentrically loaded bolted or welded connections which take into account the actual response of the fastening elements are available in the literature.[6, 7]

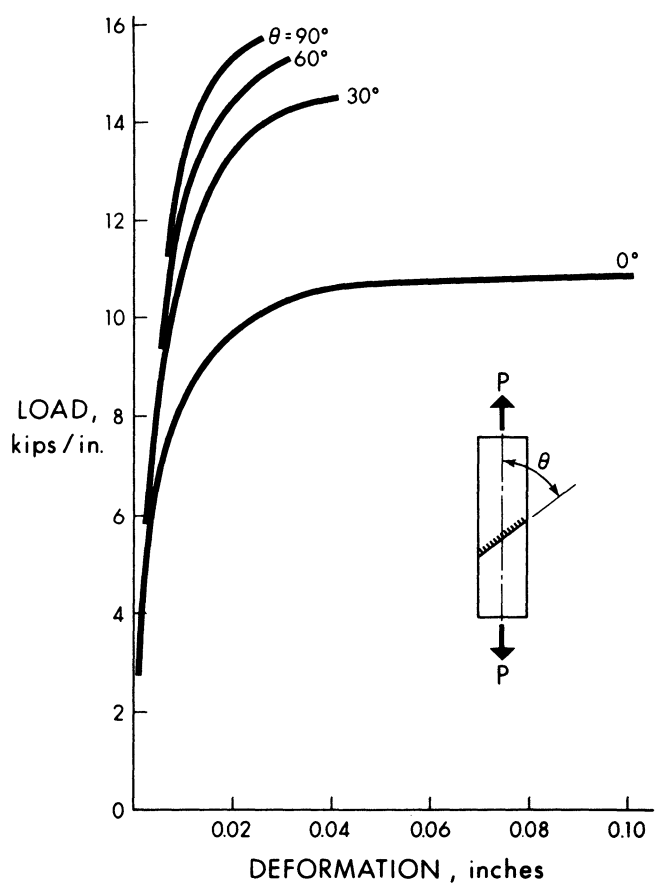

Fig. 1. Load-deformation response of welds

G. L. Kulak is Associate Professor of Civil Engineering, University of Alberta, Edmonton, Canada.

In the case of high-strength bolts, the author's assumption that a definite shear yield exists should result in design values having a more or less constant factor of safety for a given type of bolt. However, the factor of safety might not be the same when comparing, say, A325 versus A490 fasteners. For the bolt patterns and eccentricities listed in Tables 1 and 2, and using the method outlined in Reference 7, the factor of safety against ultimate is found to be approximately 3.6 for A325 bolts, but only about 3.0 for A490 bolts. Just what level the factor of safety should be is arguable, but there would seem to be no justification for having a lower factor of safety for the A490 bolts than for the A325. Indeed, because of the slightly reduced ductility of the A490 bolt, a case could be made for requiring a higher factor of safety.

For weld groups, the assumption that a definite shear yield exists should produce a more variable factor of safety, since the influence of the force direction on the weld strength has apparently not been included in the author's approach. Comparison of the values given in Table 3 with ultimate loads computed by the method described in Ref. 6, shows a maximum variation in the factor of safety of about 31 percent. The mean value of the factor of safety is apparently about 4.5, and this could be expected to be approximately the same for welds made either with E60 or E70 electrodes, since their specified elongations are nearly the same. Welds of higher strength electrodes proportioned using Table 3 would have a lower factor of safety, however, and the same comments as were made for high-strength bolts apply here.

REFERENCES

1. *Francis, A. J.* The Behaviour of Aluminium Alloy Riveted Joints *Research Report No. 15, The Aluminium Development Association, London, 1953.*
2. *Fisher, J. W. and Rumpf, J. L.* Analysis of Bolted Plate Splices *Journal of the Structural Division, ASCE, Vol. 91, No. ST5, October, 1965.*
3. *Fisher, J. W. and Beedle, L. S.* Criteria for Designing Bearing-Type Bolted Joints *Journal of the Structural Division, ASCE, Vol. 91, No. ST5, October, 1965.*
4. *Kulak, G. L. and Fisher, J. W.* Discussion of "Plastic Design of Eccentrically Loaded Fasteners" by A. L. Abolitz *Engineering Journal, AISC, Vol. 4, No. 2, April, 1967.*
5. *Wallaert, J. J. and Fisher, J. W.* Shear Strength of High-Strength Bolts, *Journal of the Structural Division, ASCE, Vol. 91, No. ST3, June, 1965.*
6. *Butler, L. J. and Kulak, G. L.* Behavior of Eccentrically Loaded Welded Connections *Proceedings of the Specialty Conference on Steel Structures, ASCE, University of Missouri, Columbia, 1970.*
7. *Crawford, S. F. and Kulak, G. L.* Eccentrically Loaded Bolted Connections *Journal of the Structural Division, ASCE, Vol. 97, No. ST3, March 1971.*

Discussion by **Carl L. Shermer**

Mr. Kulak is correct. A strict interpretation of some of my statements would suggest misconceptions. However, I assume that readers of the *Engineering Journal* know that load-deformation curves for rivets, bolts or anything else do not follow the idealized curves assumed for them. As a matter of fact, individual parts don't even follow the "exact" curves which they are supposed to follow although the deviation might not be quite as great. Stresses in similar materials do approach a common range of ultimate values and I am of the opinion that assuming these to be uniform is not too far from the truth for practical connection design.

I assume also that readers know that "neither rivets nor high-strength bolts nor fillet welds subjected to a shear-type loading exhibit any well defined yield point." Nevertheless, I believe that in the region of the maximum load, errors introduced by approximating the load-deformation curves with horizontal lines are quite small compared with the uncertainties of all connection design.

Then too, the proof of the pudding is in the eating of it. In Mr. Kulak's Ref. 7, he has used what he calls some "rather complicated expressions" to compute tables corresponding to Tables X and XI on pp. 4-62 and 4-63 of the AISC *Manual*. These also correspond to Tables 2 and 3 which appeared in my article and which were computed from quite simple expressions. My tables and those in the Manual give working values, whereas Mr. Kulak's tables give ultimate values for the same connections. The ratios between the Kulak values and my values throughout the whole range of sizes and eccentricities covered by the tables are practically constant. The maximum deviation from the average is less than 5% except for a few values for single-line three-fastener or four-fastener connections with loads eccentric by 20 in. or more, and even here the deviation is not great. Thus, I am of the opinion that while refinements of the design pose an interesting academic problem, it is difficult to justify them in ordinary design of connections where one may consider himself fortunate if his computed ultimate loads are anywhere in the neighborhood of the truth.

Mr. Kulak's work on welded connections is not at hand and so I have no way of comparing his work and mine. However, I would expect close agreement here also.

Carl L. Shermer is Chairman, Department of Civil Engineering, Ohio University, Athens, Ohio.