Treatment of Eccentrically-loaded Connections in the AISC Manual

T. R. HIGGINS

As IN EARLIER editions, the recommended loads for eccentrically loaded connections and the coefficients tabulated in the AISC *Manual of Steel Construction*, 7th Edition, for their design are more conservative than might be expected in view of recent studies and tests. (See the paper by Carl L. Shermer[1] in this issue of the *Engineering Journal.*) However, there are a number of factors — including the need for further research into the plastic behavior of large connections subject to considerable eccentricity of loading — which discouraged across-the-board adaptation of these findings during preparation of the 7th Edition of the Manual.

Unfortunately, calculation of the plastic strength of eccentrically loaded fastener patterns requiring the determination, by an iterative procedure, of the center of rotation satisfying the three equations of equilibrium[1] would be far too tedious using ordinary slide-rule methods. Therefore, resort to the familiar vector method of analysis, even at the sacrifice of some economy in fabrication cost, would still be justified in evaluating unusual fastener patterns.

To be sure, with the advent of the computer the solution of typical configurations subject to varying amounts of eccentricity can be readily obtained and presented in tabular form. However, care must be taken in choosing appropriate "yield stress" levels.

In the case of mechanical fasteners, the fastener displacement or "strain" is attributable in part to the deformation of the fastener and the connected parts (which may have significantly different mechanical properties) and in part by slip between the connected parts. The relative magnitude of each of these strain components will vary as the force applied to each fastener and the size of connection is increased.

With relatively small eccentricities, the distance from centroid of fastener pattern to center of rotation is large and instantaneous differences in strain rate at the several elements comprising the pattern are small, so that variations in the stress-strain function with increasing load is of minor significance. As Prof. Shermer has pointed out[1], with large eccentricities this distance, *m*, becomes small and differences in strain throughout the fastener pattern become much larger. In this case, depending upon the size of the pattern, strains in the outer elements could reach the point of rupture before elements near the centroid of the pattern are stressed to their assumed "yield" value.

Using, as the basis for comparison, the failure load on four different eccentrically loaded rivet patterns[2], a computed factor of safety against failure is given for three design criteria in Table 1. In the table, P_f is the load which caused failure of the weaker of two identical beam framing connections; P_1 is the corresponding design load using an elastic (vector) analysis and actual eccentricity; P_2 is the design load using the same analysis and the "effective" eccentricity; P_3 is the design load using plastic analysis, as proposed by Prof. Shermer, and actual eccentricities.

For the relatively small eccentricities in loading involved, all three design criteria afford a more-than-adequate margin against failure. As measured in terms of standard deviation, the third (plastic) criterion provides the best fit to the test results. Also, the average factor of safety for the ten tests, using this criterion, comes closest to the average for five control tests performed on single-rivet specimens. However, as can be seen, use of an effective eccentricity and an elastic analysis provides reasonably good correlation with a plastic analysis in the case of moderate eccentricities. They have been retained in Tables X through XIII in Part 4 of the AISC Manual.

It is obvious that, where large eccentricities are involved, the tabulated values become increasingly conservative, since the extent to which the actual eccentricity is modified is constant for a given number of fasteners in a group and independent of the amount of eccentricity. Before revising these tables on the basis of a plastic analysis, it would be prudent to investigate the behavior of patterns with various combinations of

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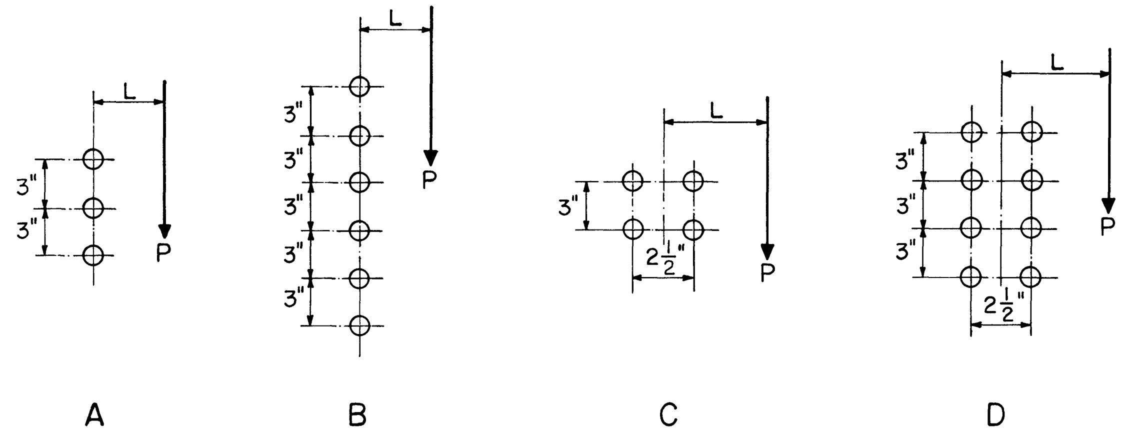
T. R. Higgins is a consulting structural engineer, New York, N.Y. and was formerly Director of Engineering & Research for AISC.

Table 1



Rivet Patterns

Test No.	Rivet Pattern	L (in.)	Indicated Factor of Safety		
			P_f/P_1	P_f/P_2	P_f/P_3
TP1	A	2½	4.36	2.90	4.00
TP2	A	3½	4.08	2.69	3.73
TP3	A	6½	4.27	3.24	4.10
TP4	B	2½	4.25	3.46	3.87
TP5	B	4½	4.51	2.93	3.77
TP6	B	6½	4.80	3.10	3.85
TP7	C	3½	5.30	3.83	4.31
TP8	C	6½	4.57	3.71	3.77
TP9	D	3½	4.87	3.09	3.88
TP10	D	6½	4.68	3.31	3.63
		Avg. =	4.56	3.23	3.89
		Std. deviation =	0.34	0.34	0.19

mechanical properties for fasteners and connected part, as well as various combinations of fastener patterns with large eccentricities.

In the case of fillet welds, stress-strain characteristics can be quite dissimilar, depending upon orientation of the stress in the weld with respect to its longitudinal axis. The ultimate strength of a transverse fillet weld has been shown[3] to be approximately 55 percent greater than that of a longitudinal weld of the same size and composition. Its strain at rupture, however, is approximately one-fourth that of the latter.

Based on a series of tests in which force was applied to fillet welds at various angles with respect to their longitudinal axis, Butler and Kulak[4] have derived a stress-strain function expressed in terms of this angle. Using the angle which a force normal to the ray from the center of rotation to a finite element in a C-shape weld pattern makes with the longitudinal axis of that element, Pal and Kulak[5] have evaluated the contribution which each element of the welds in a number of framed-beam-type test specimens made towards the ultimate strength of the beam web weld group as a whole.

Agreement between predictions based upon this approach and the actual test results was remarkably close, ranging from 5 percent below to 6.7 percent above the test results. Each test specimen consisted of a beam supported at each end, after the manner shown as Case I for Table III on page 4-28 of the AISC Manual, 7th Edition. The ratio k ranged from 0.33 to 1.0 and the ratio a from 0.71 to 1.67 in this series of tests, where k and a are as defined in Table XVI on page 4-70.

Dividing the same predictions by coefficients obtained from Prof. Shermer's Table 3*, an average factor of safety of 3.8 is indicated. Using the same allowable fillet weld stress and coefficients derived from a vector analysis, the corresponding average factor of safety would be 5.4, a very conservative value.

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* *Modified for E60XX electrodes used in fabrication of the test specimens.*

Within the range of proportions covered by the tests on C-shape weld patterns, the influence of the fillet weld stress-strain function at no time exceeded 2.2 percent of the predicted strength. For larger ranges of the ratios a and k, however, and for other weld patterns, a greater influence could be expected. Pending further study of this factor, the coefficients given in Tables XIV to XXI in the 7th Edition of the AISC Manual were revised only by increasing them in the ratio of current allowable stresses for welds made with E70XX electrodes to former stresses permitted for E60XX electrodes, i.e., 21.0:13.6 ksi.

However, use was made of ultimate strength analysis in determining the minimum effective web thickness for the framed beam connections in Tables III and IV. This limitation on web thickness is intended to prevent overstressing of the web in shear at points of maximum weld stress. However, as Prof. Shermer has pointed out[1], variation of stress in the several elements comprising a weld pattern is more nearly proportional to their distance from an instantaneous center of rotation than their distance from the centroid of the pattern. Hence, in the case of framed beam connections with relatively small eccentricities in loading, the distribution of stress throughout the pattern is more uniform than implied by an elastic analysis.

If the capacity of "Welds A" were calculated on the same basis as were the coefficients in Prof. Shermer's Table 3, the required minimum web thickness, t, would be determined by the equation

$$t = \frac{2 \times 0.707\, D \times 0.3\, (\text{T.S.})}{0.4\, F_y} \qquad (1)$$

where

D = size of fillet weld, inches
T.S. = Specified minimum tensile strength of electrode used
F_y = Specified minimum yield point of beam

However, with the increase in fillet weld stress permitted in the 1969 AISC Specification, it would be found that such a web thickness would generally exceed that of the beams for which the framed connections listed in Tables III and IV in the Manual are intended. Therefore the capacities shown are based upon an elastic (vector) analysis of Weld **A**, as heretofore. But the corresponding minimum web thickness is obtained by multiplying the value obtained from Eq. (1) by the reduction factor C_1/C_2, where C_1 is the coefficient appropriate to Weld **A**, computed in accordance with the procedure used in developing Table XVI, and C_2 is the corresponding coefficient, computed in accordance with the procedure used in developing Prof. Shermer's Table 3.

REFERENCES

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