DISCUSSION

Design of Single Plate Shear Connections

Paper by ABOLHASSAN ASTANEH, STEVEN M. CALL and KURT M. McMULLIN
(1st Quarter, 1989)

Discussion by **Ralph M. Richard**

The paper develops a design procedure for single plate shear connections based upon the results of a shear-rotation device (shown in Fig. 4 of the original paper). The claim is made that in previous studies ". . .the shear connectors have been subjected to moment and rotation or only direct shear without rotation." This is not true.

This writer developed a design procedure for single plates based upon stub beam tests and full scale beam tests that included realistic connection shears.[1] Shown in Figs. 13 and 14 of this writer's paper[1] are moment-rotation curves which show the effect of shear and given on page 45 of that paper is the analytical moment-rotation curve which indeed includes the effect of shear. It was found, however, that for practically all single plate designs the ratio, *e/h,* (eccentricity divided by bolt pattern depth), was 0.5 or greater and as shown in Fig. 13, the moment-rotation relationship is not significantly affected by the connection shear. The reason for this is that the maximum moment in single plate shear connections occurs at about 1.5 times the service load. This is shown for a three and a five bolt connection in Figs. 1 and 2, respectively, of this discussion paper and is in agreement with Astaneh's observation that ". . .based on observations made during the tests, it appears that shear tabs go through three distinctive phases of behavior. At the very early stages, a shear tab acts as a short cantilever beam with moment being dominant. As load increases, the shear tab acts as a deep shear beam with the shear yielding effect dominant." Had Astaneh performed a full scale test, he would have observed that the shear tab does not begin the shear yielding phase of action before application of 1.5 times service load. This linear connection action is shown in the shear-rotation plots of Fig. 9 in Astaneh's paper. Moreover, consider Astaneh's Design Example 1. His design procedure results in a 21 in. × ½ in. × 4½ in. plate with a shear of 102 kips service load. At 1.5 times service load, the shear stress in this 3 in. long and 21 in. deep cantilever beam is approximately 15 ksi which is less than the shear yield stress of 21.6 ksi for A36 steel. In his Design Example 2, he uses a 12 in. × ¼ in. × 4½ in. plate with a service shear load of 33 kips. The shear stress in this plate at 1.5 times service load is 16.5 ksi which again is well below the yield stress of 21.6 ksi for A36 steel.

The research at the University of Arizona, based upon stub beam tests, full scale beam tests, and inelastic finite element analyses that used experimentally determined bolt-deformation results, found that the maximum connection moment

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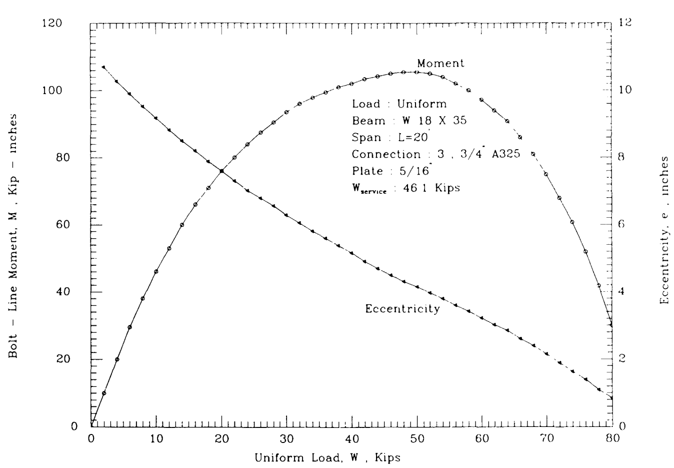

Fig. 1. Single plate moments and eccentricities.

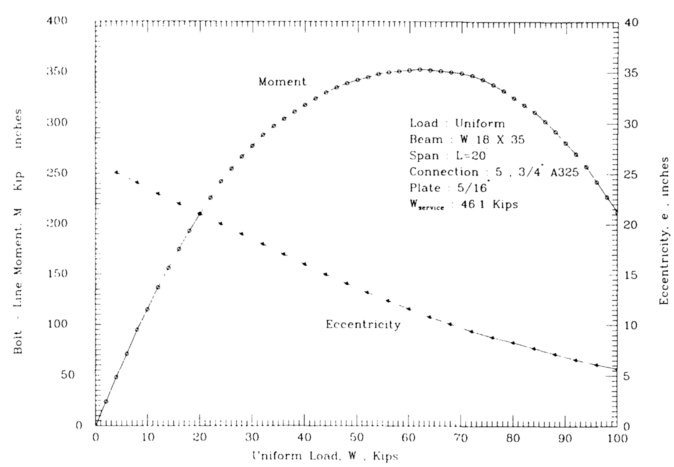

Fig. 2. Single plate moments and eccentricities.

occurred near or above 1.5 times working load as shown in Figs. 1 and 2 of this paper. The structural engineering profession requires that structural elements (connections, beam, etc.) must be designed to have the strength to resist the maximum value of the envelope of forces the element is subjected during loading. For the single plate shear connection, the maximum value of the moment the weld is subjected is at about 1.5 times the service load. Beam end rotations at these loads are of the order of 0.006 to 0.014 radians which are well below the 0.030 test values used by Astaneh. For uniformly loaded beams, it is noted that in Design Example 1, the end rotation of this beam is 0.0055 radians at service load and for Design Example 2 it is 0.0046 radians. However, Astaneh's recommended test and design procedure which is based upon shear yielding of the plate, used rotations four to six times these values.

Because of the significant difference in the design eccentricities recommended by Astaneh and those of this writer for the design of the single plate welds, this writer strongly recommends that a minimum of three full scale tests with beams subjected to a factored uniform load of 1.5 times the service load be performed by an independent laboratory to evaluate the moment generated by the single plate shear connection before this design procedure is recommended to the structural engineering profession. This writer has found that these connections generate significantly larger moments than double framing angles subjected to the same beam shear.[2] Because the bolts of the single plate are in single shear, whereas these are in double shear for double framing angles, the single plate is twice as deep and therefore much stiffer.

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Addendum/Closure by **A. Astaneh, S. C. Call and K. M. McMullin**

The discussion by Professor Richard mainly compares the research methodologies and design procedures developed by researchers at the University of Arizona (UA Method) to those developed by Authors at the University of California at Berkeley (UCB Method). The UCB Method has formed the basis of the methods currently in the 9th Edition of the AISC Manual.[11] In order to make the closure of discussion useful to the readers, the authors have responded to the statements made in the above discussion and have provided a brief comparison of the UA and UCB design methods in the following sections.

RESEARCH METHODOLOGY

In the paper, it was indicated that "...in the past in most cases, the shear connections have been subjected to moment and rotation or only shear without rotation instead of a realistic combination of shear and rotation." This statement is particularly true with experiments conducted at the University of Arizona. Figure 1 (reproduced from Fig. 3 of the paper) shows representative shear-rotation relationship that existed in the connections tested by Professor Richard and his research associates (Lines OA and OB). Also shown in the figure are actual shear-rotation relationship in a shear connection (Line OCD) and shear-rotation relationship that existed in UCB tests (Line OCE).

In the stub (cantilever) tests conducted at UA, the connections were primarily subjected to rotations with very small shear applied to the connection. The shear-rotation relationship for these tests is represented in Fig. 1 by the line OA. By comparing this shear-rotation line to the actual shear rotation line (Line OCD), it is clear that the connections in stub beam tests were subjected to unrealistically large rotations with very small shear forces applied to the connection. Since shear forces generated in stub (cantilever) beam tests are small compared to actual shear forces in shear connections, failure modes are very unrealistic, therefore, unrealistic tests should not be used to develop design procedures for shear connections.

From published data apparently a total of four tests have been conducted using the test set-up shown in Fig. 2. Similar test set-ups have been used in the past by several researchers to apply large shear forces to the connection. However, if the beam shown in Fig. 2 is not loaded to failure, the amount of rotation that will be developed in the con-

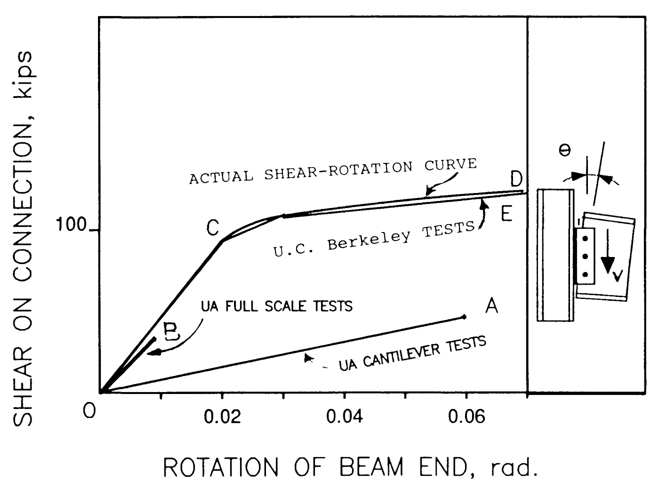

Fig. 1. Shear-rotation relationship in UA and UCB tests.

nection will be very small and will be limited to elastic end rotations which are very small compared to realistic rotations that will be imposed on the connection at the time of beam collapse.

In the full-scale tests conducted at the University of Arizona, the amount of maximum shear applied to the connections is unexplainably very low. A representative of the shear-rotation relationship applied to the connections in UA full-scale tests is shown in Fig. 1 as Line OB. Due to application of very low shear to the connection in these full-scale tests, no realistic failure mode has been observed or reported and apparently only some minor yielding of bolt holes and deformation of bolts have been observed.

It is unfortunate that full-scale tests conducted at the University of Arizona have not been loaded to failure. Apparently, the loading was not even enough to cause significant yielding in the connections. If the tests were destructive, several failure modes observed by us as well as by other researchers[1-6,10] might have been observed and invaluable data on strength of connection could be obtained. The reason for stopping the loading at such a low level apparently was a decision to load the specimens up to 1.5 times yield capacity of the beam. From published information, it is not clear why strength of the connections were studied under such an arbitrary and unrealistically low load level. Therefore, in our view, full-scale tests conducted at the University of Arizona were incomplete and have not provided information regarding strength and failure modes of the connections.

The details of full-scale tests conducted at the University of Arizona and the results are not published. However, from published data, it appears that the objective of full-scale tests at the University of Arizona may have been to study movement of point of inflection of the beam and moment-rotation behavior. Since these full-scale tests have been non-destructive and no connection failure modes have been observed, it is not clear how the information obtained from loading of specimens in elastic range could be used to develop design procedures concerning failure modes and the corresponding shear strength capacities.

The inelastic finite element program used in UA studies is an analysis program and could only provide useful information on the state of the strain and or stress. The program is not capable of predicting failure modes and strengths such as weld fracture, bolt fracture, fracture of net section or fracture of the edge distance. Apparently, the finite element program is used to simulate moment-rotation response. Again, similar to full-scale tests, in the finite element analyses the maximum load was about 1.5 times service load of the beams.

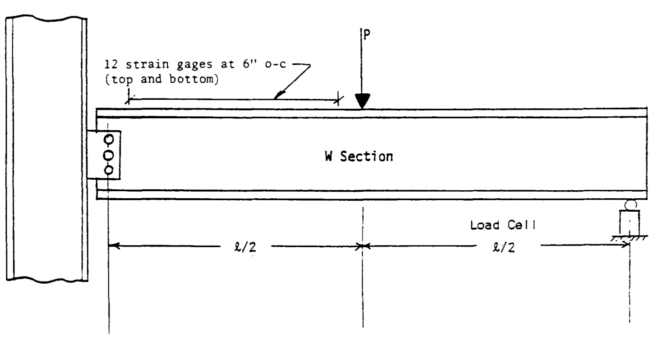

Fig. 2. Test setup used in UA tests (Ref. 8).

As far as behavior of the connection is concerned, the maximum load of 1.5 times service load of the beam used in UA tests and finite element analyses is very small. For example, the connection studied in Fig. 2 of the Discussion is loaded up to about 50 kips shear force (100 kips total beam load) whereas according to information obtained from our destructive tests of similar connections and by using well established design concepts, the shear capacity of the connection is about 130 kips (260 kips total beam load). It appears that the University of Arizona studies were limited to the initial stage of loading where beam and connection are almost elastic. Then the results of these studies are applied to full range of loading up to the failure. Since the problem is highly nonlinear, the validity of this extrapolation is questionable.

To remove the above difficulties, the authors have developed and used a test set-up that has enabled them to apply realistic combinations of shear and rotation to the connection until the connection fails. The shear-rotation relationship used by the authors is shown in Fig. 1 as Line OCE. The details of test set-up as well as authors' methodology are given in several references (1 to 6) and are not repeated here. The experimental work has resulted in establishing realistic failure modes and corresponding design procedures as reported in the paper.

COMPARISON OF UCB DESIGN PROCEDURES WITH UA PROCEDURES

The destructive tests conducted by a number of researchers including the authors have indicated that single plate shear connections have six failure modes as follows:

a) shear yielding of plate
b) bearing failure of bolt holes
c) failure of edge distance
d) shear fracture of net section
e) bolt failure
f) weld failure

The following sections provide a discussion of each failure mode and corresponding design equations in UCB Method and UA Method. In summarizing UA Method, the authors have used the available published information.[7,8,9]

a. Shear Yielding of Plate

In UCB method, this failure mode, which is very ductile and desirable, is intentionally made to be the governing failure mode.

The equation to be used to calculate the ultimate shear strength of connection for this failure mode is:

$$R_y = (L)(t)(0.6F_y) \tag{1}$$

In UA method, this failure mode is not recognized.

b. Bearing Failure

In the UCB studies,[1-6] bearing failure was observed in some specimens. In the corresponding design procedures bearing failure mode is recognized and equations that already exist in the AISC Specification[11] are used to predict bearing failure capacity of the connections.

In UA method, this failure mode is not considered. Using UA method, since there is no lower limit on the thickness of the shear tab, it is quite possible that designer unknowingly can use a thin plate with relatively large diameter bolt and cause bearing failure to be governing without ever noticing it.

The UCB design procedures as well as UA method recognize the beneficial effects of limited bearing yielding at the bolt holes. As a result both methods have an upper limit of thickness of plate relative to the bolt diameter. In UCB method the limit is $d_b/2 + \frac{1}{16}$ inch and in UA method the limit is $d_b/2$. The limited bearing yielding provides rotational ductility and causes release of moment in the connection.

c. Shear Fracture of Net Area

In UCB method this failure mode is fully recognized and the following design equation is recommended to be used to predict ultimate shear capacity of the net area:

$$R_{ult} = [L - N(\tfrac{1}{2})(d_b + \tfrac{1}{16})](t)(0.6F_u) \tag{2}$$

In a conservative approach, Eq. 3 which reflects the philosophy used in the AISC Specification[11] for shear failure of net area can be used.

$$R_{ult} = [L - N(d_b + \tfrac{1}{16})](t)(0.6F_u) \tag{3}$$

The UA method apparently does not consider this failure mode. Again, similar to bearing failure mode, it is possible that by using thin plates, net section failure can govern without the knowledge of the designer.

d. Edge Distance Failure

As a result of experiments conducted by the authors at UCB, it was realized that due to dominance of shear, the vertical edge distance below the lowest bolt is the most critical edge distance and should not be less than $1.5d_b$ nor 1.5 in. In UCB design method, it is recommended that this limitation be applied to all edge distances (see Fig. 3a).

In UA method, it is recommended that horizontal edge distance should not be less than $2d_b$ (see Fig. 3b). Apparently this recommendation is derived from results of cantilever tests where beams are subjected to large rotations and small shear forces. In our tests, the horizontal edge distances did not show signs of being critical whereas vertical edge distances particularly the lower vertical edge distance proved to be very important and critical.

e. Failure of Bolts

In UCB method, bolts are designed for the combined effects of direct shear and bending moment along the bolt line. Our tests indicated that as beam is loaded, connections yield and bending moment in the connection continuously is released to the midspan of the beam. As a result, point of inflection of the beam continuously moves toward the connection and is stabilized at a distance of e_b from the bolt line. The value of e_b can be obtained from the following equation.

$$e_b = (n - a - 1)(1.0) \tag{4}$$

Therefore, in UCB method, bolts are designed to resist combined effects of shear reaction of the beam and a moment equal to reaction multiplied by e_b.

In UA method, bolts are designed for direct shear only. This implies that bolt line is the location of point of inflection of the beam where moment is zero and only shear exists. Our experiments, as well as other tests conducted in Canada,[10] have clearly indicated that some moment develops along the bolt line.

Figure 4 shows variation of shear force and bending moment in a typical shear tab connection. The connection used to plot the curves is the same used in Fig. 2 of the Discussion. Figure 4 shows an experimental curve, UA finite element results and design equations according to UCB and UA methods. It should be mentioned that test results shown in Fig. 4 are plotted using test results for exactly similar specimen but with $\frac{3}{8}$ in. thick plate rather than $\frac{5}{16}$ in. The test results for $\frac{3}{8}$ in. plate are multiplied by $\frac{5}{6}$ to adapt them to $\frac{5}{16}$ in. plate and then are plotted in Fig. 4.

It is not known why UA's design method neglects the moment that exists along the bolt line. Even the finite element

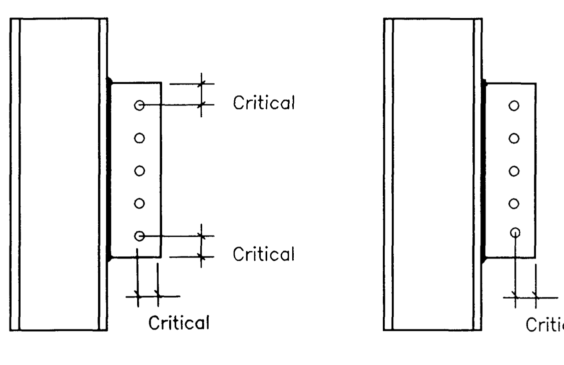

Fig. 3. Edge distance requirements in UCB and UA methods.

analysis given by Professor Richard in Figs. 1 and 2 of the Discussion shows that considerable moment is present along the bolt line. In our view, based on seven tests conducted so far by us and several other tests by other researchers on the shear tabs, neglecting moment along the bolt line is not justifiable and can result in unconservatively overestimating shear capacity of the bolts.

f. Weld Failure

In UCB method welds are designed for the combined effects of direct shear and a moment due to the eccentricity of the reaction from the weld line, e_w. The eccentricity e_w is given by the following equation. The equation is based on results of tests.

$$e_w = n(1.0) \tag{5}$$

In UA method welds are designed for combined effects of shear and moment, however, the moment that is established for design of the welds is unrealistically very large.

Figure 5 shows shear and moment variation along the weld line for the same shear tab shown in Fig. 2 of the Discussion. Similar to bolt design, the figure shows test results, UA finite element analysis (adapted from Fig. 2 of the Discussion) as well as design equations according to both methods. The plots clearly shows that if one follows UA method in design of welds, the design point will be somewhere in the vicinity of point A where moment is much larger and shear force is smaller than the realistic values that actually occur in the connection (test curve).

The reason UA method results in using very large and unrealistic moment in design of welds is the use of large eccentricity. Notice that in Fig. 5, slope of lines drawn from the origin (such as OA and OB) represent values of constant eccentricity. In the Discussion Professor Richard indicates that connection should be designed for maximum possible values of shear and moment. This statement is correct, but in UA's method rather than designing connection for maximum combination of shear and moment, the connection is designed for shear corresponding to 1.5 times service load of the beam and an eccentricity of shear that exists at the point of 1.5 times service load of the beam. What this actually means is that as beam is loaded, eccentricity moves toward the support and when shear force exceeds a value corresponding to 1.5 times service load of the beam, the eccentricity remains constant. This is shown in Fig. 5 by Line CA. This is not realistic. As Fig. 5 indicates in actual loading shown by test curve, after onset of the bolt slip and yielding in the connection (Point D), eccentricity decreases continuously and stabilizes at much smaller value than the eccentricity corresponding to Point C. This can easily be seen by comparing slope of Line CA (e_w = 13 in.) and Line EB (e_w = 5 in.).

In summary, tests conducted at the University of Arizona were not destructive and thus cannot be used to establish failure modes and design procedures. And, furthermore, the corresponding design procedure considers only bolt failure and weld failure which are only two of the six failure modes that actually should be considered. In addition, the design equations suggested for the bolt failure appear to be unconservative whereas equations proposed for weld design are based on unrealistically large moment and a small shear.

The design procedures proposed by the authors are only a step in direction of improving the design methods by using more realistic test results and failure modes. Much work needs to be done in this area particularly with respect to cyclic behavior of these connections.

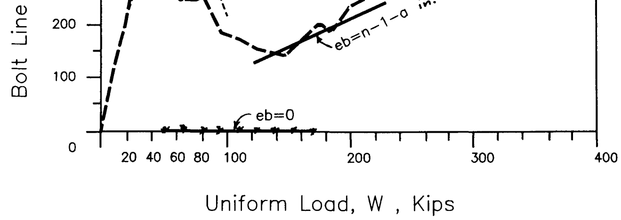

Fig. 4. Variation of shear and moment along the bolt line.

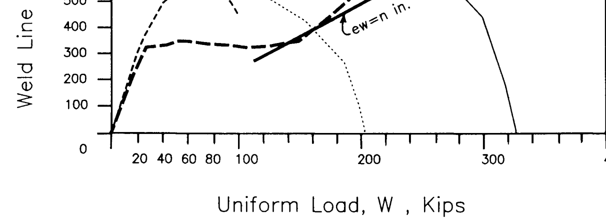

Fig. 5. Variation of shear and moment along the weld line.

ACKNOWLEDGMENTS

The opinions expressed in this closure are those of the authors and do not necessarily reflect the views of the AISC or the University of California at Berkeley. The words "UCB method" and "UA method" are only used to refer to the methods developed by the authors and by the researchers at the University of Arizona respectively.

NOTATION

- a Distance between bolt line and weld line, in.
- d_b Diameter of bolts, in.
- e_b Eccentricity of beam reaction from bolt line, in.
- e_w Eccentricity of beam reaction from weld line, in.
- L Length of shear tab, in.
- M_b Moment along bolt line, kip-in.
- n Number of bolts.
- R_y Reaction of the beam causing yielding of shear tab, kips.
- R_{ult} Reaction of the beam causing fracture of net section, kips.
- t Thickness of shear tab, in.
- W Total load carried by the beam, $W = 2R$, kips.

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AISC Commentary on Design of Shear Tabs

AISI and AISC sponsored research on single shear plate connections (shear tabs) at the University of Arizona in the late 1970s and early 1980s. At the request of the AISC Committee on Manuals and Textbooks and the ASCE Committee on Steel Building Structures, additional research was conducted at the University of California–Berkeley in 1988–89. In each case, the project scope and limit state criterion was suggested by AISC and followed by the researchers. Because the limit state was different in the two cases, the design procedure resulting from each research effort is different. This is evident by the two preceding discussions in this issue of the *Engineering Journal.* AISC assumes responsibility for these changes in the context of a natural evolution of research and improved understanding of shear tab behavior.

In the University of Arizona case, AISC directed the limit state to be a maximum connection rotation in this initial research on shear tab connections. Because AISC did not request tests to destruction, none were made. On this basis, tests and analytical studies were made and a design procedure appearing in several AISC publications was developed.

In the recent University of California–Berkeley case, the limit state was changed to ultimate load, to be determined by testing to destruction. Based on this work and previous research, a different design procedure was then developed by applying a conservative factor of safety.

The AISC Committee on Research and the AISC Committee on Manuals and Textbooks determined that the ultimate load criterion given to the University of California–Berkeley was more realistic and better represented the behavior traditionally assumed for steel connections. The ASCE Committee on Steel Building Structures concurred in this judgment.

AISC feels that both shear tab design procedures include an adequate factor of safety and either can be safely used. Because of the simpler nature of the new University of California–Berkeley method, and because its strength limit states are considered to be more complete and realistic, that method was adopted for inclusion in the Ninth Edition of the *Manual of Steel Construction.* Additional research on this method to expand its applicability to other detailing conditions is in progress.

AISC expresses its appreciation to both Professor Richard and Professor Astaneh for their contributions to the solution of this vexing design problem.