Design of Single Plate Shear Connections

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INTRODUCTION

Single plate shear connections, often referred to as shear tabs, have gained considerable popularity in recent years due to their efficiency and ease of fabrication. Shear tab connections are primarily used to transfer beam end reactions to the supporting elements. The connection consists of a plate welded to a support at one edge and bolted to a beam web. Figure 1 shows typical applications of single plate shear connections. This paper presents the summary of a research project on the behavior and design of single plate shear connections. Based on experimental and analytical studies, a new design procedure is developed and presented.

The AISC–ASD[15] as well as AISC–LRFD[16] specifications have the following provisions with regard to shear connections:

> "Except as otherwise indicated by the designer, connections of beams, girders, or trusses shall be designed as flexible, and may ordinarily be proportioned for the reaction shears only.
>
> "Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action in the connection is permitted."

Steel shear connections not only should have sufficient strength to transfer the end shear reaction of the beam but according to above provisions, the connections should also have enough rotation capacity (ductility) to accommodate the end rotation demand of a simply supported beam. In addition, the connection should be sufficiently flexible so that beam end moments become negligible. Thus, like any shear connection, single plate shear connections should be designed to satisfy the dual criteria of shear strength and rotational flexibility and ductility.

Shear-Rotation Relationship in a Shear Connection

To investigate the behavior and strength of a shear connection, it is necessary that realistic *shear forces and their corresponding rotations* be applied to the connection. In an earlier research project,[2] the shear-rotation relationship for the end supports of simply supported beams was studied. A computer program was developed[1] and used to simulate increased monotonic uniform loading of the beams supported by simple connections until the beams collapsed.[1,2]

The studies indicated that the relationship between the end shear and end rotation is relatively stable and depends primarily on the shape factor Z_x/S_x of the cross section, L/d of the beam and the grade of steel used. Figure 2 shows a series of curves representing shear forces and corresponding rotations that will exist at the ends of simply supported beams. The curves correspond to beams of A36 steel having cross sections from W16 to W33 and L/d ratios of 4 to 38. Also shown in Fig. 2 is a tri-linear curve "abcd" suggested to be a realistic representative of the shear-rotation curves. The tri-linear curve "abcd" is proposed to be used as a standard load path in studies of shear connections. Curve "abcd" is used instead of the more conservative curve "aef" because it is felt that curve "abcd" represents a more realistic maximum span-to-depth ratio for most steel structures. For special cases of very large span-to-depth ratio or high strength steels, the rotational demand may be greater than that of curve "abcd". For such cases special care must be taken to assure the rotational ductility demand of the beam is supplied by the connection.

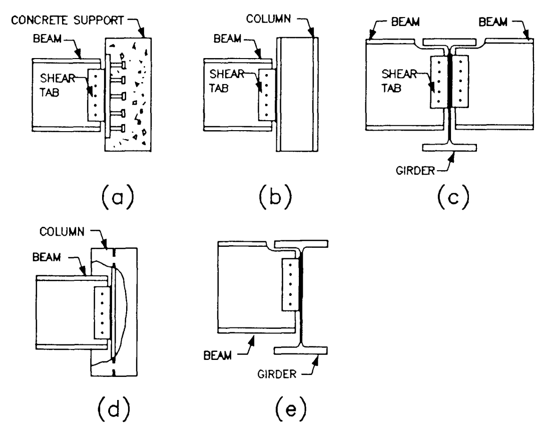

Fig. 1. Typical Single Plate Shear Connections

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The shear-rotation curves plotted in Fig. 2 are established based on the assumption of elastic-perfectly-plastic bending moment capacity for the beam. To include the effect of strain hardening, the segment "cd" in curve "abcd" is included.

The behavior of shear connections has been studied in the past by several investigators.[8,10-12] However, in most cases, the shear connections have been subjected to moment and rotation or only direct shear without rotation instead of a realistic combination of shear and rotation. Figure 3 shows the shear rotation relationships that existed in several studies including this research project.

EXPERIMENTAL RESEARCH

In order to identify limit states of strength and to verify the validity of the design procedures that were developed and proposed, five full scale beam-to-column connection assemblies were tested. A summary of the experimental studies follows. More detailed information on the research project can be found in References 3 and 6.

Test Set-up

The test set-up shown in Fig. 4 was used to apply shear-rotation relationship of curve "abcd" in Fig. 2 to the specimens.

The main components of the test set-up were a computer based data acquisition and processing system, two actuators R and S and support blocks. Actuator S, which was close to the connection, was force controlled and provided the bulk of the shear force in the connection. Actuator R, which was displacement controlled, provided and controlled the beam end rotation.

Test Load Path

The proposed standard shear-rotation relationship shown as curve "abcd" in Fig. 2 was applied to the connections in all of the test specimens. To establish the curve, coupon tests of the plate material were conducted prior to connection tests and the yield point and ultimate strength of the plate material were obtained. The shear yield capacity of the single plate in each test specimen was calculated by multiplying the von Miess criterion of shear yield stress, $1/\sqrt{3}F_y$, by the shear area of the plate. The shear yield capacity of the plate, denoted as R_y, was taken as equal to the shear at point "c" of curve "abcd" in Fig. 2. Thus the shear yield capacity of the shear tab was assumed to occur when the moment at midspan was equal to M_p. As a result, a corresponding M_p can be calculated for each connection to be equal to $R_yL/4$. The end rotation of the beam when midspan moment reached M_p was set equal to 0.03 radians.

To establish point "b" in curve "abcd", the shear at this point was set equal to $4M_y/L$ and the rotation was set equal to 0.02 radian. This implies that when beam midspan moment reaches M_y, the end rotation will be equal to 0.02 radian. The value of M_y, the end rotation will be equal to 0.02 radian. The value of M_y for each specimen was calculated by dividing M_p by the shape factor. A shape factor of 1.12 was used in all specimens.

Segment "cd" in Fig. 2 corresponds to strain hardening of the beam and the increased moment at beam midspan which results in increased shear at the beam ends. To establish "cd", it was assumed that when the midspan moment reaches a value of $(F_u/F_y)M_p$, the beam end rotation will be equal to 0.1 radian.

In summary, load path "abcd" in Fig. 2 reflects the behavior of the beam and its effect on connection shear and rotation. Segment "ab" corresponds to the elastic behavior of beam. At point "b", midspan moment of the beam reaches M_y and the beam softens. Segment "bc" corresponds to inelastic behavior of the beam. At point "c", the midspan moment reaches M_p. Segment "cd" represents extra beam capacity that can develop due to beam strain hardening.

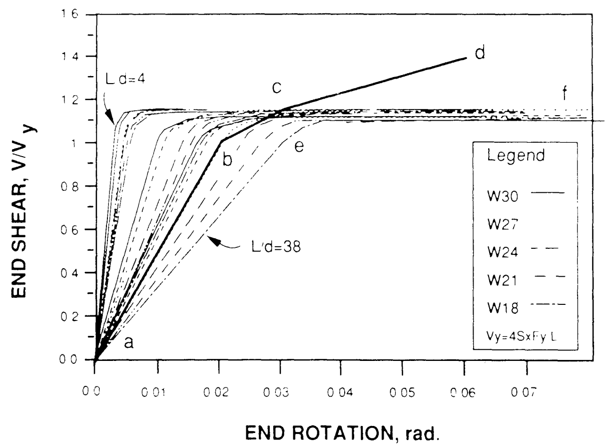


Fig. 2. Shear-Rotation Relationship for Ends of Simple Beams

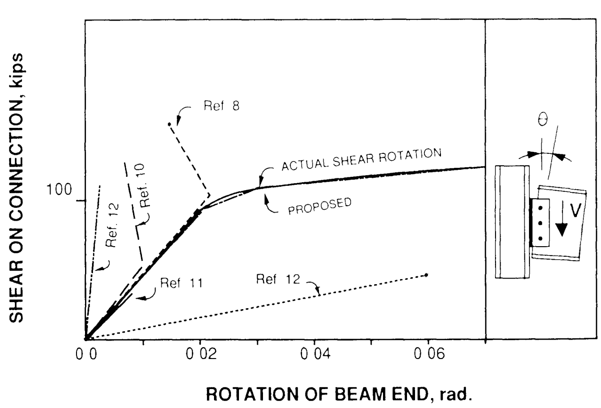


Fig. 3. Shear-Rotation Relationship used in Several Studies

Table 1. Properties of Test Specimens									
TEST GROUP	TEST NO.	NO. OF BOLTS	DIA. OF BOLTS	TYPE OF BOLTS*	PLATE DIMENSIONS	EDGE DISTANCE	‡ ACTUAL WELD SIZE	BEAM MATERIAL	PLATE MATERIAL
			in.		in. × in. × in.	in.	in.		
ONE	1	7	¾	A325-N	21 × ⅜ × 4-¼	1-½	¼	A36	A36
	2	5	¾	A325-N	15 × ⅜ × 4-¼	1-½	¼	A36	A36
	3	3	¾	A325-N	9 × ⅜ × 4-¼	1-½	¼	A36	A36
TWO	4	5	¾	A490-N	14-¼ × ⅜ × 3-⅞	1-⅛	7/32	Gr. 50	A36
	5	3	¾	A490-N	8-¼ × ⅜ × 3-⅞	1-⅛	7/32	Gr. 50	A36

**All bolts were tightened to 70% of proof load. In all specimens diameter of bolt hole was 1/16 inch larger than nominal diameter of bolt. "N" indicates that in all specimens threads were included in shear plane.*
‡Size of all welds was specified as ¼ inch.

Test Specimens

Each test specimen consisted of a wide flange beam bolted to a single plate shear connection which was welded to a column flange as shown in Fig. 1b. The properties of the test specimens were selected in consultation with a professional advisory panel. These properties are given in Table 1. The bolt holes in all specimens were standard round punched holes. All bolts were tightened to 70% of proof load using turn-of-the-nut method.[13,14] All shear tabs were cut from a single piece of steel. The yield stress and ultimate strength for material of shear tabs were 35.5 ksi and 61 ksi respectively. The condition of faying surfaces was clean mill scale. The electrodes were equivalent of E7018.

The bolt spacing in all specimens was 3 in. The edge distance in the horizontal as well as vertical direction for specimens 1,2 and 3 was 1-½ in. (two times diameter of bolt) and for specimens 4 and 5 was 1-⅛ in. (1.5 times diameter of bolts).

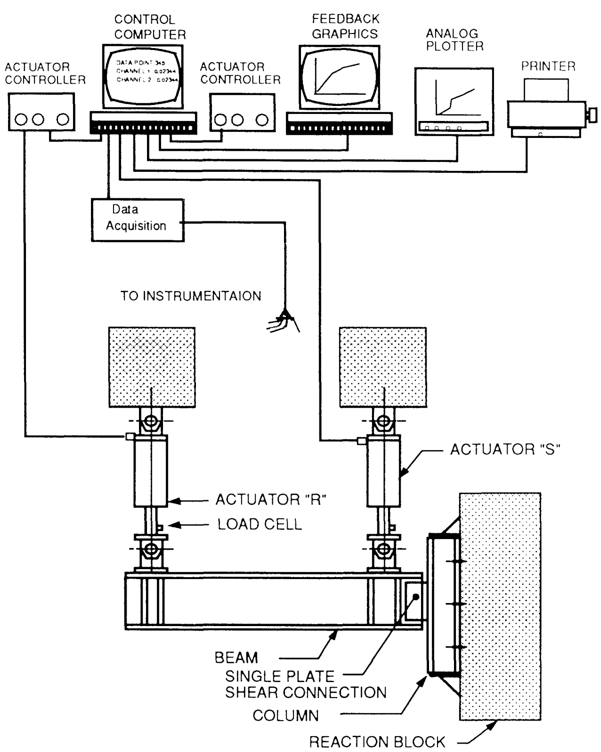


Fig. 4. Test Set-up Used in Experiments

Behavior of Test Specimens

The experiments were conducted in two groups as indicated in Table 1. The main differences of specimens in these two groups were the type of bolt (A325 or A490), material of beam (A36 or grade 50) and edge distance ($2d_b$ or $1.5d_b$). The behavior of specimens in the two groups is summarized in the following sections.

Behavior of Specimens 1,2 and 3 (Group One)

Specimens 1,2 and 3 showed very similar behavior throughout the loading. The most important observation was the significant inelastic shear deformations that took place in all three specimens as shown in Fig. 5.

All test specimens failed due to sudden shear fracture of the bolts connecting the single plate to the beam web as shown in Fig. 6a. The examination of bolts after failure indicated that the A325 bolts in these specimens had developed significant permanent deformations prior to fracture as indicated in Fig. 6b. In these three specimens the welds did not show any sign of yielding other than in specimen 3 which showed minor yielding at the top and bottom of welds prior to fracture of bolts.

A study of the bolt holes after the completion of tests 1,2 and 3 indicated that permanent bearing deformations had taken place in the plate as well as in the beam web. The magnitude of the deformations in the plate and beam bolt holes were almost equal but in opposite directions. The deformations of the plate bolt holes, drawn to scale are shown in Fig. 7. The arrows indicate the direction of the movement of the bolts which is expected to be approximately the direction of the applied force due to shear and moment. It is interesting to note that nearly vertical orientations of arrows indicate the presence of a large vertical shear accompanied by a relatively small moment in the connections.

Behavior of Specimens 4 and 5 (Group Two)

The behavior of specimens 4 and 5 was similar to the previous three tests. However, shear yielding of the plate was more apparent. Specimen 4 failed due to shear fracture of bolts in a manner similar to previous tests shown in Fig. 6a. In addition, minor yielding was observed on the weld lines of this specimen. Specimen 5 failed by almost simultaneous fracture of weld lines and bolts as shown in Fig. 8. It appears that at the time of failure, weld lines started to fracture first while bolts were on the verge of fracture. When sudden fracture of welds occured the resulting shock caused fracture of the bolts which appeared to be almost simultaneous with weld fracture. Bolts in specimens 4 and 5 were A490 bolts. An examination of the bolts after fracture showed less permanent deformations in these bolts than the A325 bolts used in previous three tests (see Fig. 6b).

Study of bolt holes in the shear tabs of specimens 4 and 5 indicated that significantly larger bolt hole deformations had occured in these two specimens compared to specimens 1,2 and 3. However, the bolt holes in the beam web in specimens 4 and 5 had only minor permanent deformations.

In summary, 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 (as in specimens 1 through 4). If bolts and welds do not fail during the shear phase, because of large deformations, the shear tab acts similarly to the diagonal member of a truss and carries the applied shear by a combination of shear and diagonal tension effects (as in specimen 5).

Experimental Data

The results of experiments at the time of failure are summarized in Table 2.

DISCUSSION OF EXPERIMENTAL RESULTS

Shear Yielding of Single Plate

The yielding of the single plate was primarily due to shear stresses and was quite ductile. It was evident that considerable shear yielding occurred in the plate between the bolt line and weld line. The shear yielding was almost uniformly distributed throughout the depth of the plate as measured by strain gages that were attached to the plates.[3,6] Therefore, in the proposed design procedure discussed later, the shear capacity of plate is calculated by multiplying gross area of plate by uniformly distributed shear stresses.

In specimen 3, at later stages of loading and after significant shear yielding, the bottom portion of the shear tab showed signs of minor local buckling as shown in Fig. 6a. This local buckling was attributed primarily to loss of stiffness of plate material due to shear yielding. Until this phe-

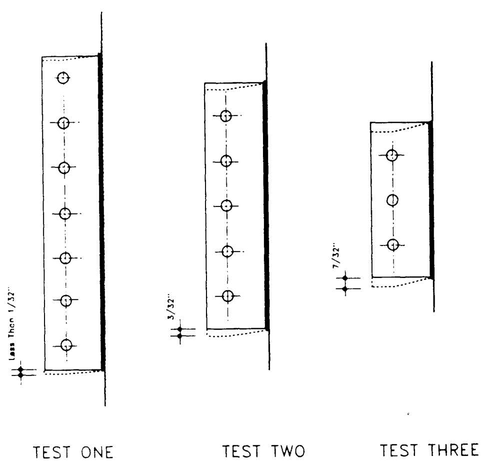
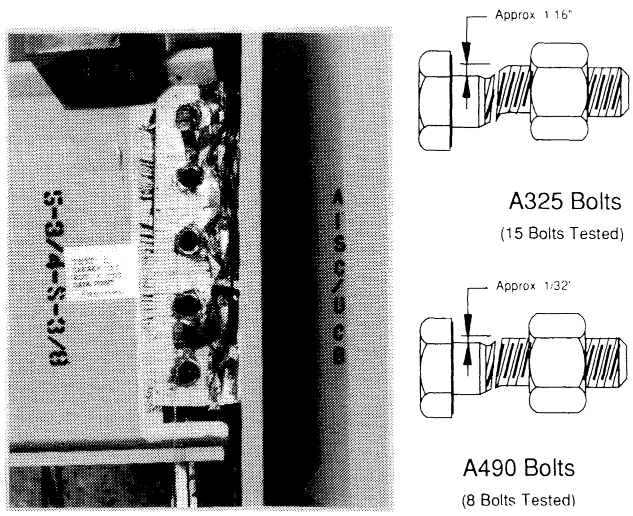


Fig. 5. Plate Shear Deformations in Specimens 1,2,3



(a) (b)

Fig. 6. Typical Bolt Failure of Test Specimens

Table 2. Results of shear strength Tests									
Specimen			Observed Failure Mode	Connection Response					
Test Group	Test No.	No. of Bolts		Shear Displacement	Shear Force	Beam End Rotation	‡ Moment at Bolt Line	‡ Moment at Weld Line	Maximum Moment at Weld Line
				in.	kips	rad.	kip in.	kip in.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
One	1	7	Bolts Fractured	0.27	160	0.026	306	745	1028
	2	5	Bolts Fractured	0.34	137	0.054	314	691	734
	3	3	Bolts Fractured	0.46	94	0.056	20	279	350
Two	4	5	Bolts Fractured	0.35	130	0.053	273	631	686
	5	3	Welds and Bolts Fractured	0.52	79	0.061	−47	170	237

**In some cases like these, moment decreased as shear and rotation increased.*
‡Positive moments cause top of connection to be in tension.

nomenon is studied thoroughly, it is suggested that local buckling be avoided. To prevent local buckling, it is recommended that the distance between the bolt line and the weld line be less than ½ of the plate length.

Fracture of Net Area of Plate

In the single plate specimens that were tested, the net area of the plate did not fracture. Only specimen 5 showed signs of approaching fracture of net section. Nevertheless, this failure mode has been observed in similar cases in several experiments on tee framing connections.[4,5] The stem in a tee framing connection behaves similarly to a shear tab. The formula currently used in calculating net area in shear fracture is:[15]

$$A_{ns} = A_{vg} - n(d_b + 1/16)t_p \quad (1)$$

The studies of tee connections indicated that the shear fracture occurred consistently by fracture of net section along the edge of the bolt hole and not along the centerline of bolts. It was suggested that[4,5] the net area effective in shear be equal to the average of net area along the bolt centerline and the gross area. Using the suggested method to calculate net area in shear, the effective net area in shear can be written as:

$$A_{nse} = A_{vg} - (n/2)(d_b + 1/16)t_p \quad (2)$$

Shear-Rotation Behavior

Figure 9 shows the actual shear-rotation relationship that was recorded during each test. It is observed that the rotational ductility of the connections increased as the

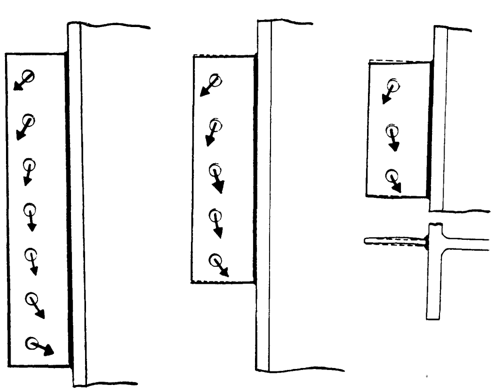


Fig. 7. Plate Bolt Hole Deformations after Tests

Fig. 8. Failure of Welds and Bolts in Specimen 5

number of bolts decreased. The rotational ductility of the connection in specimen 1 with 7 bolts was 0.026 radians which was about half the rotational ductility of the connections in specimens 2, 3, 4 and 5 with three or five bolts, all of which were able to reach rotations in excess of 0.05 radians.

Movement of Point of Inflection

Figure 10 shows movement of point of inflection of the beam toward the support as the shear force was increased. Even under relatively small load, in all specimens, the point of inflection moved toward the support and remained almost stationary for the remainder of each test.

Using experimental data, the following empirical equation was developed to define the location of the point of inflection for test specimens.

$$e = (n-1)(1.0), \text{ in.} \qquad (3)$$

where n is the number of bolts used in the connection, and e is the distance of point of inflection from the support (i.e. from the weld line).

It is important to realize that in the experiments reported here, the columns were fixed to supports and rigid body rotation of the connections was prevented. If due to frame action or other causes, the support to which a shear tab is connected rotates, due to rigid body rotation, the location of point of inflection may be affected. However, the concurrent values of shear and moment acting on the shear tab at any given time cannot exceed the values obtained from plasticity conditions (interaction curves) of plate for shear and moment.

Behavior and Design of Bolts

In all specimens, an examination of bolts and bolt holes after failure indicated that bolt shanks had experienced considerable shear deformations before failure.

Studies on the behavior of single bolts in shear[11] have indicated that for A325 bolts and A36 plate, if the thickness of the plate is not greater than $\frac{1}{2}$ times the diameter of the bolt, considerable but tolerable bolt hole deformations will take place. The limited bolt hole deformations are desirable since they increase rotational flexibility and ductility of the connections. In studies of tee connections[4,5] in three specimens, $\frac{1}{2}$ in. thick tee stems were used with $\frac{7}{8}$ in. diameter bolts. The behavior of these tee specimens indicated that even when thickness of stem was equal to $d_b + \frac{1}{16}$ in., desirable bearing deformations took place in the bolt holes. Therefore, based on these studies, and to obtain flexible and ductile single plate connections, the thickness of the plate is recommended to be less than or equal to $\frac{1}{2}$ of the bolt diameter plus $\frac{1}{16}$ in.

An examination of the deformations of bolts and bolt holes at the completion of the tests indicated that the bolts were primarily subjected to direct shear accompanied by a small moment (see arrows in Fig. 6a).

As Fig. 10 indicates, the point of inflection for test specimens was almost stationary, fluctuating between an eccentricity of n and $n - 1$ in. At the time of failure of the bolts in all specimens, the location of the point of inflection was close to $n - 1$ in. Therefore, it is recommended that bolts be designed for combined effects of direct shear and a moment equal to the shear multiplied by the eccentricity of the bolt line from point of inflection given by:

$$e_b = (n-1)(1.0) - a \qquad (4)$$

where,

a = distance between the bolt line and weld line,

e_b = distance from the point of inflection to the bolt line.

Behavior and Design of Welds

Table 2 gives values of shear and moment at failure for each test. The fillet welds mainly experienced a direct

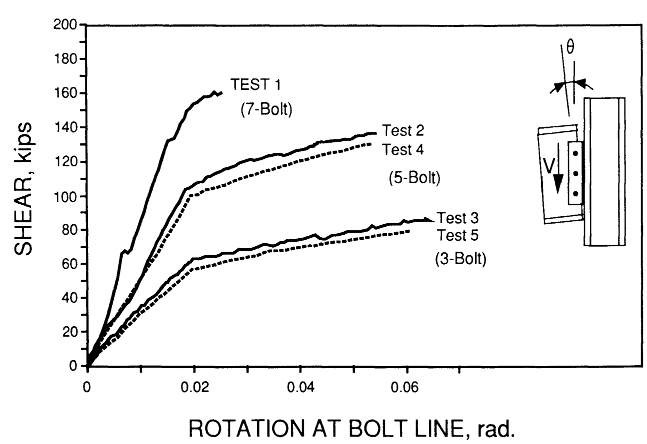


Fig. 9. Shear-Rotation Curves for Test Specimens

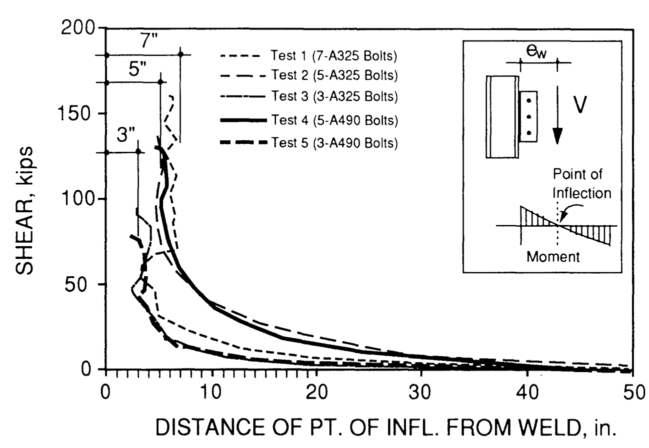


Fig. 10. Movement of Point of Inflection

shear accompanied by a relatively small moment. The strain measurements adjacent to the welds also supported this conclusion.[3,6] Therefore, fillet welds are recommended to be designed for the combined effects of shear and a small bending moment.

The main goal of the proposed design procedure is to ensure yielding of shear tab prior to failure of welds. In order to achieve this goal the welds should be designed to be stronger than the plate. Thus, the design shear force acting on the welds is recommended to be equal to the shear capacity of the plate and not the applied shear force. Therefore, the maximum shear force acting on the weld is equal to $1/\sqrt{3}\ F_y L_p t_p$. In Allowable Stress Design, the design shear force for welds is equal to $0.40 F_y L_p t_p$. The moment acting on the weld is equal to shear force multiplied by the eccentricity of the point of inflection from the weld line. To be conservative, it is recommended that the eccentricity of the point of inflection from the weld line be equal to n inches,

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

Since the design of welds in the proposed method is a capacity design, it is not necessary to use welds that can resist forces much greater than the plate capacity. As part of phase two of this investigation, a study was conducted to establish minimum and maximum weld requirements to develop the strength of single plate. The study indicated that for A36 plate and E70 electrodes the weld size need not be more than $0.75t_p$ and should not be greater than t_p. The upper limit of t_p on the weld size was imposed to prevent excessive welding of the plate which will be costly and might cause heat damage to the plate without achieving extra strength in the connection.

Moment-Rotation Curves

Moment-rotation curves for the test specimens are shown in Fig. 11. Moments and rotations were measured along the bolt line. As the plots indicate, connections with fewer bolts developed smaller moments and exhibited larger rotational ductility. During the elastic range of behavior, moment increased with shear. As the load increased, due to connection deformations, rotational stiffness and bending moment decreased and then gradually increased at a much smaller rate. The decrease is attributed to slips and inelastic deformations in the connections and the increase is attributed to strain hardening.

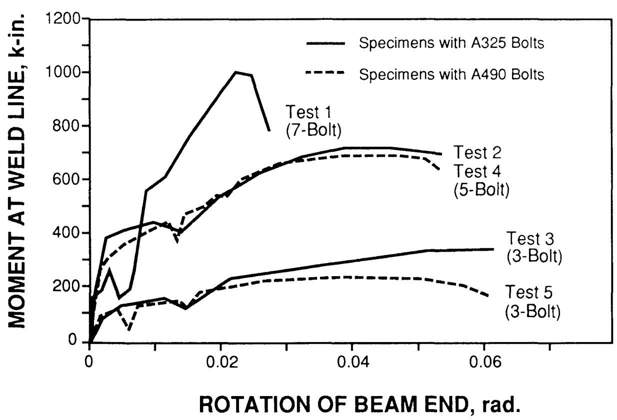


Fig. 11. Moment-Rotation Curves for Test Specimens

PROPOSED DESIGN PROCEDURE

The following design procedure is based on the analyses of the experimental results and the information available on the actual behavior of shear connections.[1-6,9]

General Requirements

The single plate framing connections covered by these procedures consist of a plate bolted to a beam web and welded to a support on one edge of plate.

In design of a single plate framing connection, the following requirements should be satisfied:

1. The connection has only one vertical row of bolts and the number of bolts is not less than 2 or more than 7.
2. Bolt spacing is equal to 3 in.
3. Edge distances are equal to or greater than $1.5d_b$. The vertical edge distance for the lowest bolt is preferred not to be less than 1.5 in.
4. The distance from bolt line to weld line is equal to 3 in.
5. Material of the shear plate is A36 steel to facilitate yielding.
6. Welds are fillet welds with E70xx or E60xx electrodes.
7. Thickness of the single plate should be less than or equal to $d_b/2 + 1/16$.
8. The ratio of L_p/a of the plate should be greater than or equal to 2 to prevent local buckling of plate.
9. ASTM A325 and A490 bolts may be used. Fully tightened as well as snug tight bolts are permitted. The procedure is not applicable to oversized or long slotted bolt holes. Standard or short-slotted punched or drilled holes are permitted.

Consideration of Limit States in Design

The following limit states are associated with the single plate framing connections.

1. Shear failure of bolts.
2. Yielding of gross area of plate.
3. Fracture of net area of plate.
4. Fracture of welds
5. Bearing failure of beam web or plate.

Shear Failure of Bolts

Bolts are designed for the combined effects of direct shear and a moment due to the eccentricity e_b of the reaction from the bolt line. The eccentricity e_b for single plate connections covered by these procedures can be assumed to be equal to 3 in., which is the distance from bolt line to weld line. The value is conservative when the single plate is welded to a rigid support. The value is more realistic when the supporting member is a relatively flexible element.

More realistic values for e_b can be calculated from the following equations:

if single plate is welded to a rotationally rigid element, e_b is obtained from:

$$e_b = (n-1)(1.0)-a \tag{6}$$

if single plate is welded to a rotationally flexible element, e_b is larger value obtained from:

$$e_b = \text{Max} \left| \begin{array}{ll} (n-1)(1.0) - a & \quad (7a) \\ a & \quad (7b) \end{array} \right.$$

where,
n = number of bolts
a = distance from bolt line to weld line, in.
e_b = eccentricity, in.

By using methods outlined in Reference 7 including using Tables X of the AISC-ASD Manual[13] the bolts are designed for the combined effects of shear R, and moment equal to Re_b.

Yielding of Gross Area of Plate

The equation defining this limit state in allowable stress design (ASD) format is:

$$f_{vy} \le F_{vy} \tag{8}$$

where,

$$f_{vy} = R \,/\, A_{vg} \tag{9}$$

$$F_{vy} = 0.40\ F_y \tag{10}$$

$$A_{vg} = L_p\ t_p \tag{11}$$

Fracture of Net Area of Plate

The equation defining this limit state in allowable stress design (ASD) format is:

$$f_{vu} \le F_{vu} \tag{12}$$

where,

$$f_{vu} = R \,/\, A_{ns} \tag{13}$$

$$F_{vu} = 0.30\ F_u \tag{14}$$

$$A_{ns} = [L_p - n(d_b + 1/16)]t_p \tag{15}$$

If the beam is coped, the block shear failure of the beam web also should be considered as discussed in the AISC-ASD Specification.[15]

Weld Failure

The welds connecting the plate to the support 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 equal to the larger value obtained from:

$$e_w = \text{Max} \left| \begin{array}{ll} (n)(1.0) & \quad (16a) \\ a & \quad (16b) \end{array} \right.$$

where,
n = number of bolts
e_w = eccentricity, in.
a = distance from bolt line to weld line, in.

By using methods outlined in Reference 7 including using Tables XIX of the AISC-ASD Manual,[13] the fillet welds are designed for the combined effects of shear equal to R and moment equal to Re_w.

Bearing Failure of Plate or Beam Web

To avoid reaching this limit state, it is recommended that the established rule of horizontal and vertical edge distances equaling at least 1.5 the bolt diameter be followed. The bolt spacings should satisfy requirements of the AISC-ASD Specification.[15] The bearing strength of connection can be calculated using the provisions of the AISC-ASD Specification.[15]

Summary of Design Procedure

The following steps are recommended to be taken in design of single plate framing connections:

1. Calculate number of bolts required to resist combined effects of shear R, and moment Re_b using Table X of the AISC-ASD Manual.[13]

If the single plate is welded to a rotationally rigid support e_b is the value obtained from Eq. 6.

If the single plate is welded to a rotationally flexible element, e_b is the value obtained from Eq. 7:

2. Calculate required gross area of plate:

$$A_{vg} \ge R \,/\, 0.40F_y \tag{17}$$

Use A36 steel and select a plate satisfying the following requirements:

a. l_h and $l_v \ge 1.5d_b$. (18)
b. $L_p \ge 2a$ (19)
c. $t_p \le d_b/2 + 1/16$ (20)
d. $t_p \ge A_{vg}/L_p$ (21)
e. Bolt spacing =3 in.

3. Check effective net section:

Calculate allowable shear strength of the effective net area:

$$R_{ns}=[L_p-n(d_b+1/16)](t_p)(0.3F_u) \quad (22)$$

and satisfy that $R_{ns} \geq R$.

4. Calculate actual allowable shear yield strength of the selected plate:

$$R_o = L_p t_p\ (0.40F_y) \quad (23)$$

Design fillet welds for the combined effects of shear R_o and moment $R_o e_w$ using Table XIX of the AISC Manual.[13] e_w is given in Eq. 16 as:

$$e_w = \text{Max} \left| \begin{array}{ll} (n)(1.0) & (16a) \\ a & (16b) \end{array} \right.$$

The weld is designed for a capacity of R_o, and not for the applied R, to ensure that the plate yields before the welds. However, for A36 steel and E70 electrodes the weld size need not be larger than ¾ of the plate thickness.

5. Check bearing capacity of bolt group:

$$(n)(t)(d_b)(1.2F_u) \geq R \quad (24)$$

If the bolts are expected to resist a moment (as they normally would), this calculation should reflect the reduced strength as determined by Table X of the AISC Manual[13] as demonstrated in the following examples.

6. If the beam is coped, the possibility of block shear failure should be investigated.

Application to Design Problems

The following examples show how the design procedure can be implemented into the design of steel structures.

Design Example 1

Given:

Beam:	W27 × 114, t_w = 0.570 in.
Beam Material:	A36 steel
Support:	Column flange (Assumed rigid)
Reaction:	102 kips (Service Load)
Bolts:	⅞ in. dia. A490-N (snug tight)
Bolt Spacing:	3 in.
Welds:	E70XX fillet welds

Design a single plate framing connection to transfer the beam reaction to supporting column.

Solution:

1. Calculate number of bolts:

Shear = R = 102 kips

Let us assume M = 0, (will be checked later)

$n = R/\ r_v$= 102/16.8 = 6.1

Try 7 bolts

The distance between the bolt line and the weld line a is selected equal to 3 in.

Check moment:

$e_b = (n-1)1.0 - a = 7 - 1 - 3 = 3.0$ in.

Moment = 3 × 102 = 306 kip-in.

Using Table X of the AISC-ASD Manual[13] with eccentricity of 3 in., a value of 6.06 is obtained for effective number of bolts (7 bolts are only as effective as 6.06 bolts).

Therefore,

R_{bolt} = 6.06 × 16.8 = 101.8 ≈ 102 kips **O.K.**

Use: Seven ⅞ in. dia. A490-N bolts.

2. Calculate required gross area of the plate:

$A_{vg} = R\ /\ 0.40F_y$

A_{vg} = 102/(0.40 × 36) = 7.08 in.^{2}

Use A36 steel and select a plate satisfying the following requirements:

a. l_h and $l_v \geq 1.5d_b$

$l_h = l_v$ = 1.5(⅞) = 1.32 in.

$W = a + l_h$ = 3 + 1.32 = 4.32; **use W = 4½ in.**

b. $L_p/a \geq 2.0$

L_p = 2 × 1.32 + 6 × 3.0 = 20.6 in.; **use L_p = 21 in.**

Check: L_p/a = 21/3 = 7 > 2 **O.K.**

c. $t_p \leq d_b/2 + 1/16$

$t_p \leq$ (⅞)/2 + 1/16 = ½ in.

d. $t_p = A_{vg}\ /L_p$

t_p = 7.08/21 = 0.337 in.

Try PL 21 × ⅜ × 4-½

3. Calculate allowable shear strength of the net area:

$R_{ns} = [L_p-n(d_b + 1/16)](t_p)(0.3F_u)$

R_{ns} = [21−7(⅞ + 1/16)](⅜)(0.3 × 58) = 94 < 102 kips **N.G.**

Try ½ in. thick plate:

R_{ns} = [21−7(⅞ + 1/16)](½)(0.3 × 58) = 125 > 102 kips. **O.K.**

Use: PL 21×½×4½, A36 Steel.

4. Calculate the actual allowable yield strength of the selected plate:

$R_o = L_p t_p\ (0.40F_y)$

R_o = 21 × 0.5 × 0.40 × 36 = 151 kips

Design fillet welds for the combined effects of shear and moment:

Shear = R_o = 151 kips

$$e_w = \text{Max} \left| \begin{array}{ll} n(1.0) = 7(1.0) & = 7 \text{ in.} \\ a & = 3 \text{ in.} \end{array} \right.$$

Therefore, e_w = 7.0 in.

Moment = $R_o e_w$ = 151 × 7 = 1057 kip-in.

Using Table XIX AISC Manual[13]

$a = 7/21 = 0.333$
$C_1 = 1.0$
$C = 1.07$
$D_{16} = R_o/CC_1L_p = 151/(1.0 \times 1.07 \times 21) = 6.72$
Since weld size need not be greater than $0.75t_p$,

Use: ⅜ in. E70 Fillet Welds.

5. Check bearing capacity:
For plate:
$r_v = d_b t_p\ (1.2F_u) = .875 \times .5 \times 1.2 \times 58 = 30.45$

$R_{brg} = 6.06(30.45) = 184.5 \text{ kips} > 102 \text{ kips}.$ **O.K.**
Since the beam web is thicker than the plate, the web will not fail.

6. Beam is not coped, therefore, there is no need for consideration of block shear failure.

Design Example 2

Given:

Beam:	W16×31, $t_w = 0.275$
Beam Material:	A572 Gr. 50 steel
Support:	Condition of support is unknown
Reaction:	33 kips (Service Load)
Bolts:	¾ in. dia. A325-N or A490 (snug tight)
Bolt Spacing:	3 in.
Welds:	E70XX fillet welds

Design a single plate shear connection to transfer the beam reaction to the support.

Solution:

1. Calculate number of bolts:
Shear = 33 kips
Let us assume $M = 0$, (will be checked later)
Try A325-N bolts with 9.3 kips/bolt shear capacity:
$n = R/\ r_v = 33/9.3 = 3.5$
Try 4 bolts.
The distance between bolt line and weld line a is selected equal to 3 in.
Check moment:
Since condition of support is not known, the support is conservatively assumed to be flexible for bolt design. Therefore e_b is equal to 3 in.
Moment = 3 × 33 = 99.0 kip-in.
Interpolating from Table X[13], $C \approx 2.81$
$R_{all} = 2.81 \times 9.3 = 26.1 \text{ kips} < 33$ **N.G.**
Which indicates 4 A325 bolts are not enough. Let us try 4 A490-N bolts:
$R_{all} = 2.81 \times 12.4 = 34.8 \text{ kips} > 33$ **O.K.**

Use: Four ¾ in. dia. A490-N bolts.

2. Calculate required gross area of plate:
$A_{vg} = R\ /\ 0.40F_y$
$A_{vg} = 33/(0.40 \times 36) = 2.29 \text{ in.}^2$
Use A36 steel and select a plate satisfying the following requirements:
a. l_h and $l_v \geq 1.5d_b$.
$l_h = l_v = 1.5(3/4) = 1.125$ in.
$W = a + l_h = 3 + 1.125 = 4.125$ in.
Use: W = 4½ in.
b. $L_p/a \geq 2.0$
$L_p = 3 + 3 \times 3 = 12$ in.
Check: $L_p/a = 12/3 = 4 > 2$ **O.K.**
c. $t_p \leq d_b/2 + 1/16$
$t_p \leq (3/4)/2 + 1/16 = 7/16$ in.
d. $t_p = A_{vg}\ /L_p$
$t_p = 2.29/12 = 0.19$ in.

Use: PL 12×¼×4½, A36 Steel.

3. Calculate allowable shear strength of the net area:
$R_{ns} = [L_p - n(d_b + 1/16)](t_p)(0.3F_u)$
$R_{ns} = [12 - 4(3/4 + 1/16)](1/4)(0.3 \times 58) = 38.1 \text{ kips}$
$R_{ns} \geq R$ is satisfied.

4. Calculate actual allowable yield strength of the selected plate:
$R_o = L_p t_p\ (0.40F_y)$
$R_o = 12 \times 0.25 \times 0.40 \times 36 = 43.2 \text{ kips}$
Design fillet welds for the combined effects of shear and moment:
Shear = R_o = 43.2 kips

$$e_w = \text{Max} \left| \begin{array}{ll} (n)(1.0) = 4(1.0) = 4 \text{ in.} \\ a \qquad\qquad\qquad = 3.0 \end{array} \right.$$

Therefore, $e_w = 4.0$ in.
Moment = $R_o e_w = 43.2 \times 4 = 172.8$ kip-in.
Using Table XIX AISC Manual[13]
$a = 4/12 = 0.33$
$C_1 = 1.0$
$C = 1.07$
$D_{16} = R_o/CC_1L_p = 43.2/(1.0 \times 1.07 \times 12) = 3.36$
Since weld size need not be greater than $0.75t_p$,

Use: 3/16 in. E70 Fillet Welds.

5. Check bearing capacity.
For plate:
$nd_b t_p\ (1.2F_u) = 2.81 \times .75 \times .25 \times 1.2 \times 58$
$= 36.7 \text{ kips} > 33 \text{ kips}.$
and for beam:
$nd_b t_w(1.2F_u) = 2.81 \times .75 \times .27 \times 1.2 \times 65$
$= 44.4 \text{ kips} > 33 \text{ kips}.$

6. Beam is not coped, therefore, no need for consideration of block shear failure.

CONCLUSIONS

Based on the studies reported here, the following conclusions were reached:

1. The experimental studies of single plate connections in-

dicated that considerable shear and bearing yielding occurred in the plate prior to the failure. The yielding caused reduction of the rotational stiffness which in turn caused release of the end moments to midspan of the beam.

2. The limit states associated with single plate connections are:
   a. Plate yielding.
   b. Fracture of net section of plate.
   c. Bolt fracture.
   d. Weld fracture.
   e. Bearing failure of bolt holes.
3. A new design procedure for single plate shear connections is developed and recommended. The procedure is based on a concept that emphasizes facilitating shear and bearing yielding of the plate to reduce rotational stiffness of the connection.
4. To avoid bearing fracture, the horizontal and vertical edge distance of the bolt holes are recommended to be at least 1.5 times diameter of the bolt. The study reported here indicated that vertical edge distance, particularly below the bottom bolt is the most critical edge distance.
5. Single plate connections that were tested were very ductile and tolerated rotations from 0.026 to 0.061 radians at the point of maximum shear. Rotational flexibility and ductility decreased with increase in number of bolts.

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NOMENCLATURE

A_{ns} Net area in shear, in.^{2}
A_{nse} Effective net area of plate in shear, in.^{2}
A_{vg} Gross area of plate in shear, in^2.
C Coefficient in the AISC Manual Tables X and XIX
C_1 Coefficient in the AISC Manual Table XIX
D_{16} Number of sixteenth of an inch in fillet weld size
F_u Specified minimum tensile strength of steel, ksi
F_{vy} Allowable shear stress for plate in yielding = $0.40F_y$, ksi
F_{vu} Allowable ultimate shear strength = $0.30F_u$, ksi
F_y Specified yield stress of steel, ksi
L Length of span, in.
L_p Length of plate, in.
M_p Plastic moment capacity of cross section = Z_xF_y
M_y Yield moment of beam cross section, kip-in.
R Reaction of the beam due to service load, kips
R_{bolt} Allowable shear capacity of bolt group
R_{ns} Allowable shear fracture capacity of the net section
R_o Allowable shear yield strength of plate, kips
R_y Reaction corresponding to plastic collapse of beam, kips
S_x Section modulus in.^{3}
V Shear force, kips
W Width of plate, in.
Z_x Plastic section modulus, in.^{3}
a Coefficient in the AISC Manual Table XIX
a Distance between bolt line and weld line, in.
d Depth of beam, in.
d_b Diameter of bolt, in.
e Eccentricity of point of inflection from the support
e_b Eccentricity of beam reaction from bolt line, in.
e_w Eccentricity of beam reaction from weld line, in.
f_{vy} Computed shear stress in plate gross area, ksi
f_{vu} Computed shear stress in plate effective net area, ksi
l_h Horizontal edge distance of bolts, in.
l_v Vertical edge distance of bolts, in.
n Number of bolts
r_v Allowable shear strength of one bolt, kips
t_p Thickness of plate, in.
t_w Thickness of beam web, in.

DISCUSSION

Design of Single Plate Shear Connections

Paper by ABOLHASSAN ASTANEH, STEVEN M. CALL and KURT M. McMULLIN
(lst 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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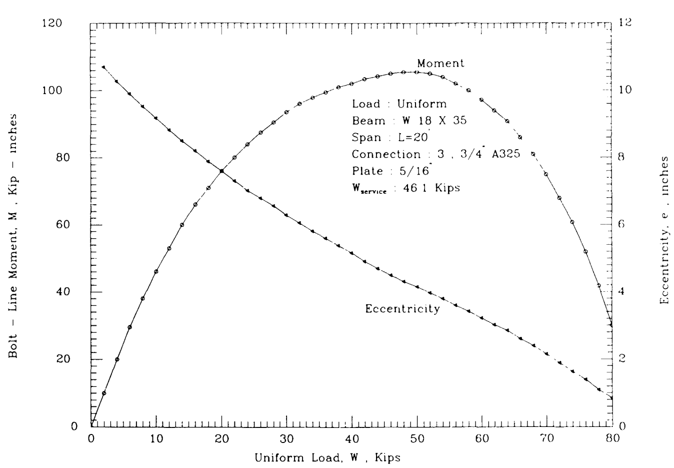

Fig. 1. Single plate moments and eccentricities.

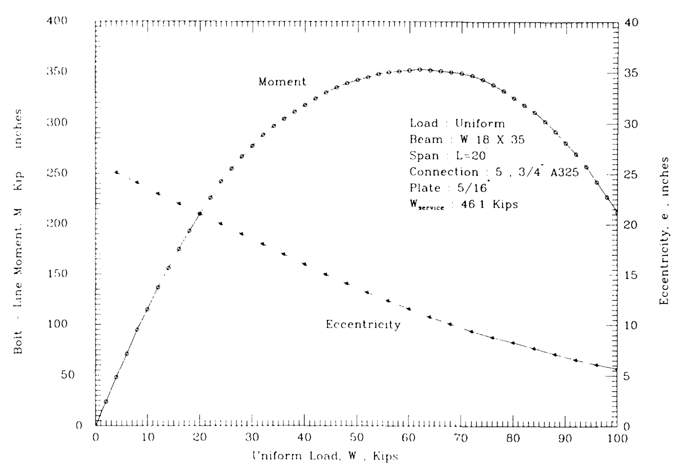

Fig. 2. Single plate moments and eccentricities.

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-

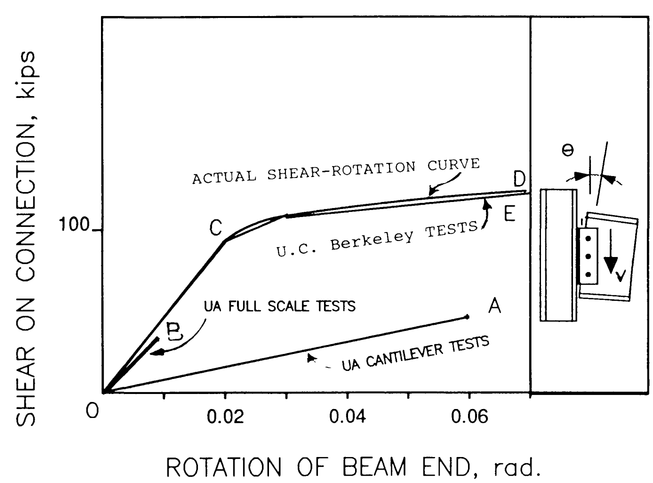

Fig. 1. Shear-rotation relationship in UA and UCB tests.

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.

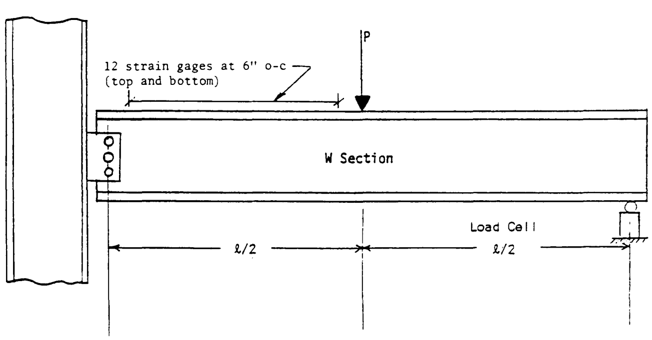

Fig. 2. Test setup used in UA tests (Ref. 8).

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:

$$R_y = (L)(t)(0.6F_y) \qquad (1)$$

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 $d_b/2 + 1/16$ 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:

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

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.

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

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.

$$e_b = (n - a - 1)(1.0) \qquad (4)$$

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 3/8 in. thick plate rather than 5/16 in. The test results for 3/8 in. plate are multiplied by 5/6 to adapt them to 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

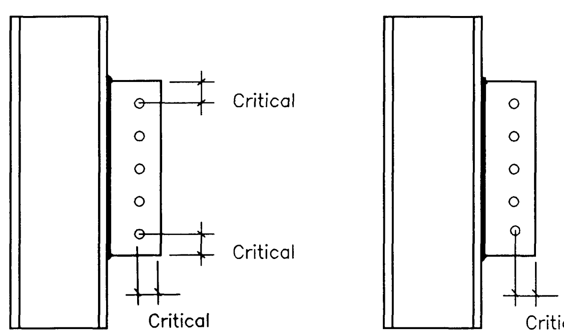

Fig. 3. Edge distance requirements in UCB and UA methods.

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.

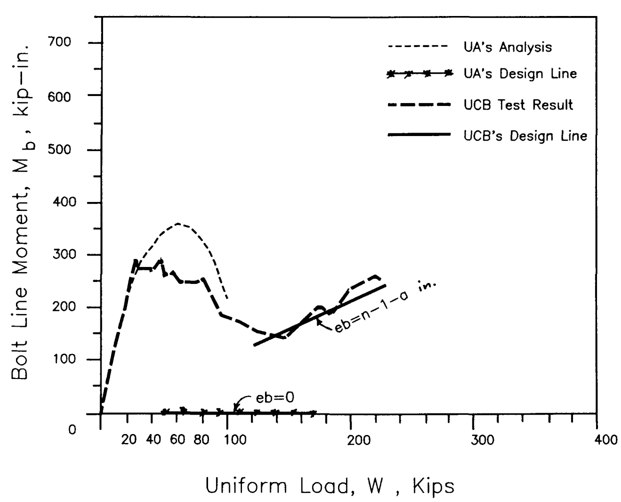

Fig. 4. Variation of shear and moment along the bolt line.

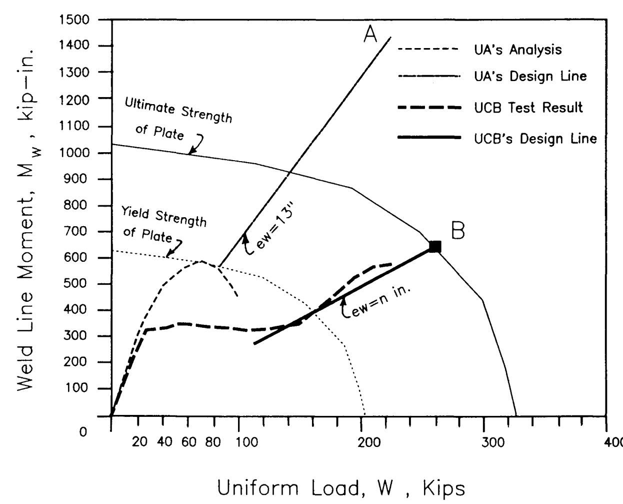

Fig. 5. Variation of shear and moment along the weld line.

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.