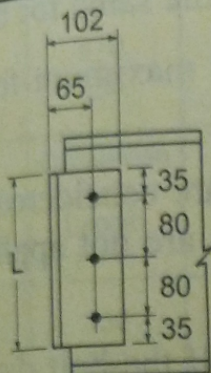


Use: L102x76x7.9 connection angle, 310 mm long, 76 mm leg welded to supporting member with 5 mm E49XX fillet welds; 102 mm leg bolted to web of supported beam with four M20 A325M bolts.

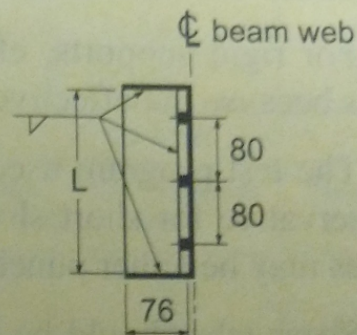


Web-Framing Leg  
(Bolted to Supported Web)

## SINGLE-ANGLE BEAM CONNECTIONS

Table 3-40

M20, M22 A325M Bolts  
3/4, 7/8 A325 Bolts  
E49XX Fillet Welds



Outstanding Leg  
(Welded to Supporting Member)

| Bolts<br>per<br>Vertical<br>Line             | BEARING-TYPE CONNECTION<br>Factored Load Resistance (kN)<br>Threads included |     |     |     | WELD CAPACITY<br>Factored Load Resistance (kN)  |     |     | Connection<br>Angle<br>Length<br>L<br>(mm) |
|--|--|-----|-----|-----|---|-----|-----|--|
|  | Bolt Size  |     |     |     | Fillet Size D (mm)  |     |     |  |
|  | 3/4  | M20 | M22 | 7/8 | 5   | 6   | 8   |  |
| 2  | 158  | 175 | 212 | 215 | 149   | 179 | 238 | 150  |
| 3  | 237  | 263 | 318 | 323 | 223   | 268 | 357 | 230  |
| 4  | 316  | 350 | 424 | 430 | 298   | 357 | 476 | 310  |
| 5  | 395  | 438 | 530 | 538 | 372   | 447 | 595 | 390  |
| 6  | 474  | 526 | 636 | 645 | 447   | 536 | 715 | 470  |
| 7  | 553  | 613 | 742 | 753 | 517   | 621 | 827 | 550  |
| 8  | 632  | 701 | 848 | 860 | 570   | 684 | 912 | 630  |
| Material<br>F <sub>u</sub><br>(MPa)          | Minimum Required Web Thickness<br>of Supported Beam <sup>1</sup> (mm)        |     |     |     | Minimum Required Thickness<br>of Supporting Member with Beams<br>Framing from One Side (mm) |     |     | Material<br>F <sub>y</sub><br>(MPa)        |
| 450  | 3.8  | 4.1 | 4.5 | 4.5 | 3.7   | 4.5 | 5.9 | 345  |
| F <sub>y</sub> = 300<br>F <sub>u</sub> = 450 | Minimum Required Thickness<br>of Framing Angle (mm)                          |     |     |     |   |     |     |  |
|  | 6.5  | 7.2 | 8.7 | 8.9 |   |     |     |  |

1. Coped beams may have additional requirements. See page 3-59.



connection angle. The tests also demonstrated that the use of horizontal slotted holes in the connection angle reduced the moment at the bolts without affecting the ultimate capacity of the connection.

Table 3-40 is based on this research and assumes the use of 102x76x9.5 connection angles with the 76 mm leg welded to the supporting member and the 102 mm leg bolted to the supported web. Bolt capacities for bearing-type connections are provided for M20 and M22 A325M bolts and  $\frac{3}{4}$  and  $\frac{7}{8}$  inch A325 bolts based on their factored shear resistance for the appropriate number of bolts. The weld capacities have been established by assuming that a connection with  $\frac{1}{4}$  inch fillet welds has the same shear capacity as the  $\frac{3}{4}$  inch A325 bolts (based on  $\phi_b = 0.67$  in S16.1-94 and with the threads excluded), and then pro-rating for the three sizes of fillet welds shown in the table (i.e. weld capacity  $= V_r \{ \frac{3}{4} \text{ inch bolts} \} \times D / 6.35$ , where  $D$  is the fillet weld size in mm).

## References

- LIPSON, S.L. 1980. Single-angle welded-bolted beam connections. Canadian Journal of Civil Engineering, 7(2), June.
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## Example

Single-angle welded-bolted beam connection (Table 3-40)

### Given:

- W410x60 beam of ASTM A992 steel, factored reaction = 290 kN, web = 7.7 mm  
L102x76x7.9 connection angle of G40.21 300W steel.  
M20 A325M bolts, E49XX electrodes.

## Single-angle welded-bolted beam connections

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This paper reports the results of the last phase of an experimental investigation of load tests on full-size single-angle connections in which one leg of the angle is connected to a beam with ASTM A 325 high-strength bolts and the other leg is welded to the supporting member. In the present study, one series of tests has shown that connections with "short slotted" holes are as reliable as similar connections with round holes but provide a greater measure of flexibility. Two other series of connections with round holes, but with different weld patterns, proved to be flexible but could not carry the desired shears.

Cet article donne les résultats de la dernière phase d'une expérience d'essais de charges sur des assemblages à cornière simple de vraie grandeur dans lesquels une aile de la cornière est assemblée à une poutre avec boulons ASTM A 325 très rigides et l'autre aile est soudée à l'élément portant. Dans la présente étude, une série d'essais a montré que les assemblages avec petits trous ovalisés sont aussi bons que les assemblages semblables avec trous ronds, mais fournissent une plus grande mesure de flexibilité. Deux autres séries d'assemblages à trous ronds, mais ayant des modèles de soudures différents, se sont montrés flexibles mais ne pouvaient pas supporter le cisaillement prescrit.

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[Traduit par la revue]

### Introduction

Traditionally, the "double-angle" beam framing connection has been used almost universally to join each end of a steel beam to the supporting members. Initially it was used with rivets through both legs, but more recently it has been used either with bolts, or welds, or a combination of both. These connections have been investigated thoroughly by a number of researchers (American Society of Civil Engineers 1967).

Since the last war the single-angle beam connection has come into use, but has not been accepted widely because of a lack of hard evidence that it is reliable under working load, and that it provides a sufficiently high factor of safety against failure. An experimental program to provide some evidence on both of these counts was begun in the Civil Engineering Department at the University of British Columbia a number of years ago and the first report was presented at a Canadian structural engineering conference (Lipson 1968). At the same time, a great deal of other work was being carried out in various locations on different types of connections and on the behavior of welds and high-strength bolts (Fisher and Struik 1974).

In the last 4 years additional work was performed by the writer on full-scale connections to failure (Lipson 1977; Lipson and Haque 1978), both experimentally and analytically. Each of these connections was a single angle bolted to the beam with high-strength bolts and welded to the supporting

member. The studies have shown the magnitude of restraining moments developed by the various sizes of connections, which may be of some concern for cases where a simple shear connection is desired. For that reason the present study was undertaken to modify some of the details of the connection in order to increase its flexibility. Two separate modifications were investigated. The first modification consisted of using slotted holes in the beam leg of the angle, all other details remaining unchanged. The second modification consisted of two different weld patterns on the column leg, and, in addition, one of these patterns was used with a different size of angle. As in the previous work, all mating surfaces were grit-blasted.

### Description of Tests

The testing procedures have been described previously (Lipson 1977) in detail and will not be repeated here. For completeness, however, the test setup is shown in Fig. 1. Briefly, it was desired to simulate the actual service condition on the connection using a single unyielding test beam and column. In such an arrangement the characteristics of the angle connection itself were obtained. Each angle connection was welded to a 1 in. (25.4 mm) thick plate, which was bolted to the test column, and the test beam was then bolted with 3/4 in. (19 mm) high-strength ASTM A 325 bolts to the angle using the "turn of the nut" method with a pneumatic wrench to turn the nut through a one half turn after hand-tightening to a snug condition.

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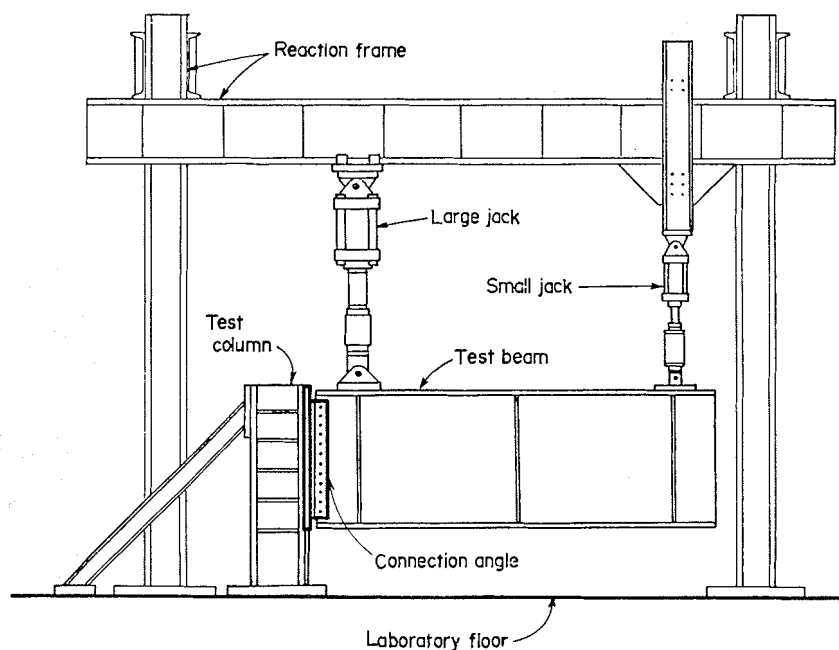


FIG. 1. Test setup.

Shear loads were applied by means of the 450 kip (2002 kN) large jack and the rotation of the test beam was controlled by means of the 50 kip (222 kN) small jack. In all cases, the two jacks were under a single control so that the shear and rotation were varied simultaneously. The loading was continuous, with each cycle of loading taking about 20 min, so that the tests were simulating static conditions. At intervals throughout each loading cycle, a set of readings was taken of the force and displacement of each jack, of the displacements of the top and bottom of the loading beam with respect to the column, and of the rotation of the test beam with respect to the connection angle. All displacements were measured with linearly variable differential transformers and recorded on punched paper tape, with each set of readings being made in a period of 3–5 s.

The nature of each test series is described in the following sections.

### Description of Specimens

All test specimens were ordered from a large structural steel fabricating shop with a request that they be fabricated using the usual procedures, together with other work and to the same tolerances. The material was specified to be to Canadian Standards Association Specification G40.21, grade 44W, and the welding was to be performed with E70XX electrodes. Three series of specimens were tested, each of which consisted of 11 specimens varying in size from two to 12 bolts, inclusive. Each

series was ordered at a different time, but in each case it was specified that all test angles as well as a 3 ft (914 mm) length of plain angle be cut from a single stock length. Standard tension coupons were cut from the plain angle and the measured properties are given in Table 1.

In what follows, the three series are designated by the letters C, D, and E. In all cases, the welding on the column leg was specified as 1/4 in. (6.35 mm) fillet, and the holes in the beam leg were punched at a 2½ in. (63.5 mm) gauge. Series C specimens were made of standard 4 in. × 3 in. × 3/8 in. (102 mm × 76 mm × 9.5 mm) angles with the 3 in. (76 mm) leg being connected to the column and the holes in the beam leg being 13/16 in. × 1 in. slots (20.6 mm × 25.4 mm), with the short axis in the long direction of the angle. Details of this series are shown in Fig. 2. Series D differed from series C in two respects: the holes in the beam leg were round, 13/16 in. (20.6 mm) in diameter, and the welding on the column leg was as shown in Fig. 3. Series E was made of standard 4 in. × 4 in. × 5/16 in. (102 mm × 102 mm × 7.9 mm) angles; the holes in the beam leg were round as in series D; and the welding on the column leg was as shown in Fig. 3. In all cases, the edge distance to a rolled edge was greater than the minimum allowable, and the end distance to a sheared edge was equal to the minimum allowable by the CSA Standard S16-1969.

The weld patterns for series D and E connections were designed in the commonly accepted manner



TABLE 1. Material properties

| Series No. | Coupon No. | Yield stress (ksi (kPa)) | Ultimate stress (ksi (kPa)) | Elongation in 8 in. (%) |
|------------|------------|--------------------------|-----------------------------|-------------------------|
| C          | 1          | 48.64 (335.4)            | 71.98 (496.3)               | 28.06                   |
|            | 2          | 48.85 (336.8)            | 71.53 (493.2)               | 28.00                   |
| D          | 1          | 50.10 (345.4)            | 79.14 (545.7)               | 25.16                   |
|            | 2          | 54.86 (378.3)            | 80.57 (555.5)               | 25.16                   |
| E          | 1          | 54.61 (376.5)            | 83.05 (572.6)               | 23.13                   |
|            | 2          | 55.06 (379.6)            | 83.05 (572.6)               | 24.11                   |

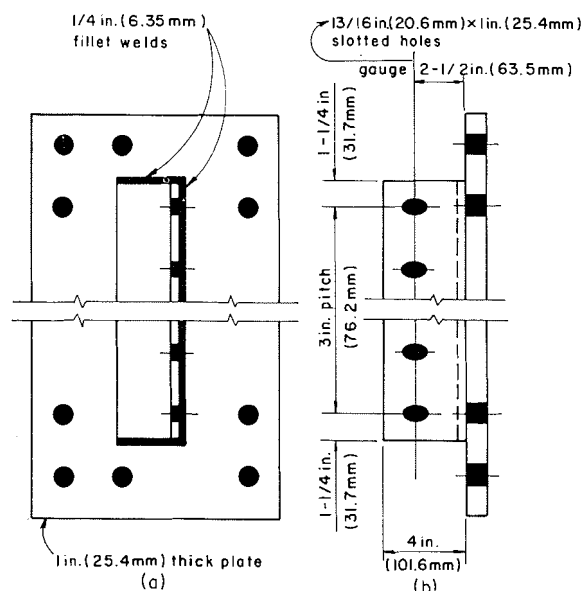


FIG. 2. Details of series C specimens: (a) weld pattern on column leg; (b) beam leg.

quacy was the maximum weld stress, calculated as the resultant of the direct shear force and the torsional moment due to the eccentricity in the plane of the welds only. Under these conditions, the welds in the two-bolt connections of both patterns are somewhat overstressed for friction bolts. Nevertheless, they were included in the tests to determine if there is a significant difference in their behavior under load. On the other hand, series D connections larger than six bolts and series E connections larger than three bolts are adequate for bearing bolts as well. The effect of the eccentricity of the force out of the plane of the welds has been the subject of a number of studies (Archer *et al.* 1959; Higgins 1964; Johnston and Green 1940). These have shown that substantial rotations can be attained and that, for the purpose of design, either the actual eccentricity can be substantially reduced or neglected, or the calculated bending stresses can be reduced by as much as 80%. The addition of out-of-plane moments about both principal axes of the welds, using an "effective eccentricity" (Higgins), would only slightly overstress the four-bolt connections in series E for bearing bolts, but would not alter the capacity of any of the other connections.

### Test Results for Series C

In a previous study (Lipson 1977) other single-angle connections were subjected to three cycles of shear loading of about 25 kips (111 kN) per bolt with a simultaneous rotation of up to about 0.024 rad. At the beginning of that investigation this shear represented about 2.5 times the allowable design value for bearing-type bolts under service loads. Some of these connections and others were then loaded to failure. Although the allowable design value has since then been increased to 13.2 kips (58.7 kN) it was decided to use the same procedure for the current tests in order to obtain corresponding results. Thus, each of the connections was subjected to three cycles of shear of 25 kips (111 kN) per bolt with a rotation of about 0.024 rad and then a fourth

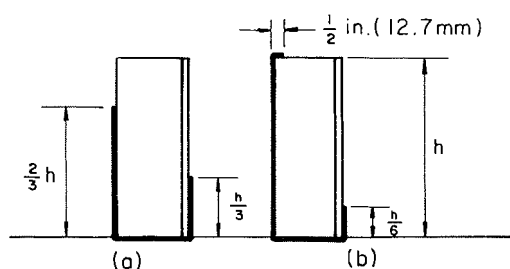


FIG. 3. Details of welding for series D (a) and E (b) specimens.

(Bresler *et al.* 1968; Gaylord and Gaylord 1972; Tall 1974) using the then allowable design values for fillet welds and bolts. These values were: 3.7 kips/lineal in. (0.65 kN/lineal mm) for 1/4 in. (6.35 mm) fillet welds; 6.63 kips (29.5 kN) per bolt for 3/4 in. (19.1 mm) A 325 friction bolts; and 9.72 kips (43.2 kN) per bolt for 3/4 in. (19.1 mm) A 325 bearing bolts. In these designs the criterion for ade-

TABLE 2. Test results at first slip—series C

| Specimen No. | Moment (kip-in.) | Shear (kips) | Vertical displacement (in.) | Rotation (rad) | Calculated horizontal force per bolt (kips) |
|--------------|------------------|--------------|-----------------------------|----------------|---|
| (1)          | (2)              | (3)          | (4)                         | (5)            | (6)   |
| 2C           | 37               | 8.3          | 0.00756                     | 0.00512        | 12.3  |
| 3C           | 91               | 8.9          | 0.00296                     | 0.00233        | 15.2  |
| 4C           | 105              | 19.6         | 0.00758                     | 0.00449        | 8.8   |
| 5C           | 201              | 18.8         | 0.00528                     | 0.00319        | 11.2  |
| 6C           | 270              | 25.2         | 0.00676                     | 0.00359        | 10.0  |
| 7C           | 382              | 33.6         | 0.00767                     | 0.00410        | 10.6  |
| 8C           | 430              | 40.1         | 0.00964                     | 0.00441        | 9.0   |
| 9C           | 616              | 42.8         | 0.00827                     | 0.00398        | 10.3  |
| 10C          | 833              | 40.5         | 0.00185                     | 0.00320        | 11.1  |
| 11C          | 787              | 50.0         | 0.01011                     | 0.00379        | 8.7   |
| 12C          | 936              | 42.3         | 0.00532                     | 0.00275        | 8.7   |

NOTES: 1 kip = 4.45 kN; 1 in. = 25.4 mm.

cycle to failure. The controls on the loading equipment were such that it was very difficult to obtain these values precisely, so that the exact value of desired rotation was not reached in most cases.

The behavior of each connection in the first cycle of loading is shown in Fig. 4 in which are plotted the moments in the bolt group for each size of connection. It is clear that during loading, the first slip, which is characterized by the high moment, takes place very early in the loading. Moments, shears, and displacements at first slip are given in Table 2. The numbering of specimens in this and subsequent tables is such that the digits preceding the letter indicate the number of bolts in the connection and the letter refers to the series of connections. The vertical shear in all cases is no more than 5 kips (22.2 kN) per bolt; the vertical displacement, in all but one case, is less than 0.01 in. (0.25 mm); and the rotation is near 0.004 rad in most cases. If it is assumed that all bolts slip at the same time and at the same frictional force for each, then the slip values per bolt are those given in column (6) of Table 2. It is most unlikely that this assumption is correct, so that these values are lower bounds for slip values for the deeper connections and reasonably good approximations for the smaller connections.

After initial slip, on the second and third cycles the moment-rotation relation is entirely different and is shown for the 12-bolt and the four-bolt connections in Fig. 5. There is a gradual change in the shape of such curves from the biggest to the smallest connections. In all cases, the moment in the bolts is near zero as the shear increases and then, for the larger connections, it increases fairly rapidly as the rotation reaches the maximum. In the smaller connections the moment is near zero and then becomes increasingly more negative, the sign convention being positive for

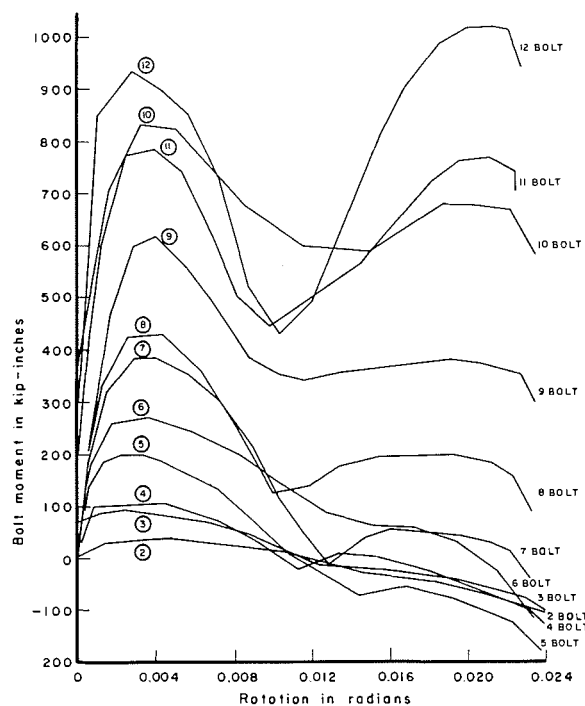


FIG. 4. Moment-rotation curves: series C. Notes: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

the top bolts being pulled away from the column. These hysteresis loops for each connection were essentially alike for the unloading portion of the first cycle and for the entire second and third cycles.

In Fig. 6 are plotted the mean of the maximum bolt moments, both positive and negative, and the bolt moments at a shear equal to 13.2 kips (58.7 kN) per bolt. It is seen that both the maximum positive moment and that at the working load are near zero for the 2-7-bolt connections, inclusive, whereas for

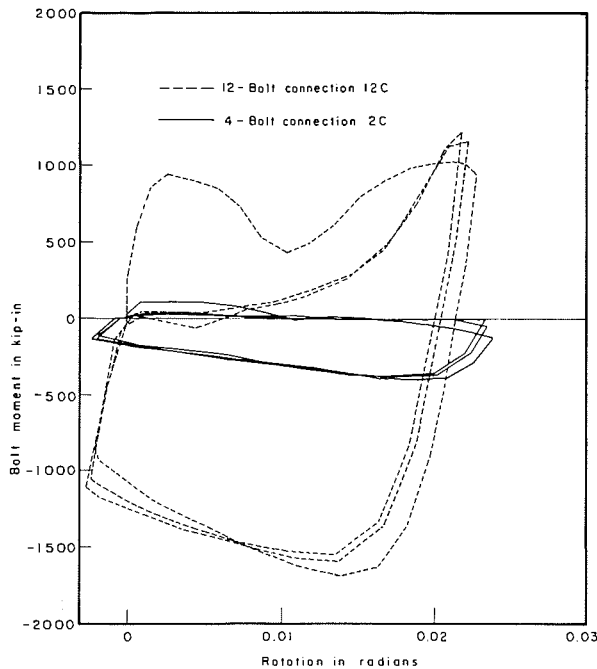


FIG. 5. Moment-rotation curves for 12-bolt and four-bolt connections: series C. Notes: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

the 8-12-bolt connections there are significant differences. In an earlier investigation (Lipson 1977), the maximum moments for 32 connections with round holes were plotted in the same way and a curve was fitted to these data. The equation of this curve is

$$[1] \quad M = 0.000435V^{2.75}$$

where  $M$  is the moment at the bolt line in kip-inches; and  $V$  is the shear in kips.

This curve is also plotted in Fig. 6 and shows that the moments in the connections of series C are in all cases less than half of those given by [1].

The vertical displacement for the four-bolt connection is shown in Fig. 7 for all four cycles. It is interesting to note that not only is the shape of this curve the same for all connections but the maximum displacements, displacements at working load, and residual displacements after three cycles are near constant for the entire range of connections. These three values for all connections in this series are shown in Fig. 8.

After three full cycles of the above loading, each connection was then loaded to failure. The results of this test are summarized in Table 3. None of the welds failed at the ultimate load, and, aside from the two-bolt connection in which there was excessive deformation in the angle, one or more bolts sheared

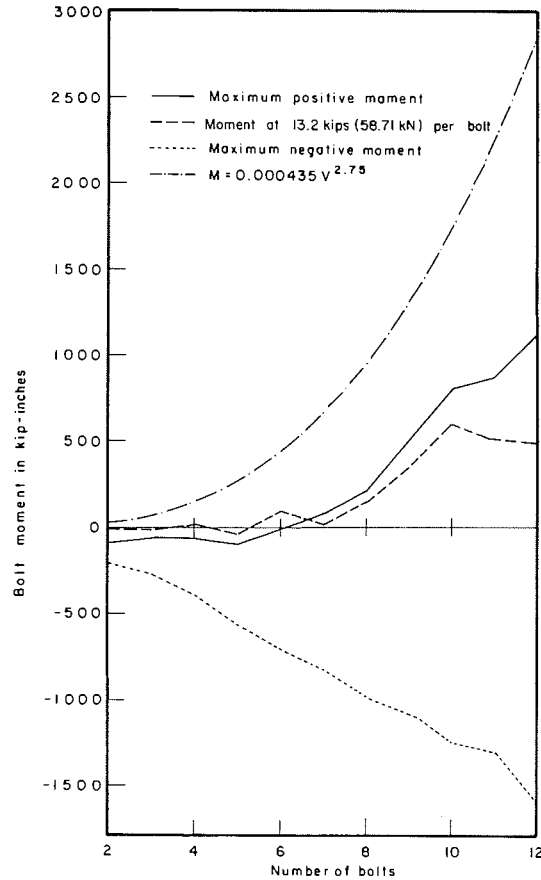


FIG. 6. Bolt moments at maximum rotation: series C. Notes: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

in each of the other connections. In five of the specimens, the bottom bolt tore the angle and all the other bolts sheared. In four specimens, all the bolts sheared and, in the last specimen (12C), only the top bolt sheared when the loading was stopped. Undoubtedly the rest of the bolts would have sheared had the loading been continued somewhat further. The average load per bolt at failure is shown in column (3) and this is compared with the corresponding values in column (4) for a similar set of connections with round holes 13/16 in. (20.6 mm) in diameter instead of slotted holes as in this series. Calculations of individual bolt forces in the case of round holes using the measured deformation of the holes indicated that, at the ultimate load, the variation among the various bolts was not very significant so that the average is a good approximation of the true value. There is no significant difference in the failure shears between round and slotted holes. The average for all specimens in this series is, in fact, about 2% higher than the corresponding value for round holes, and the standard deviation is almost 5% smaller. If the

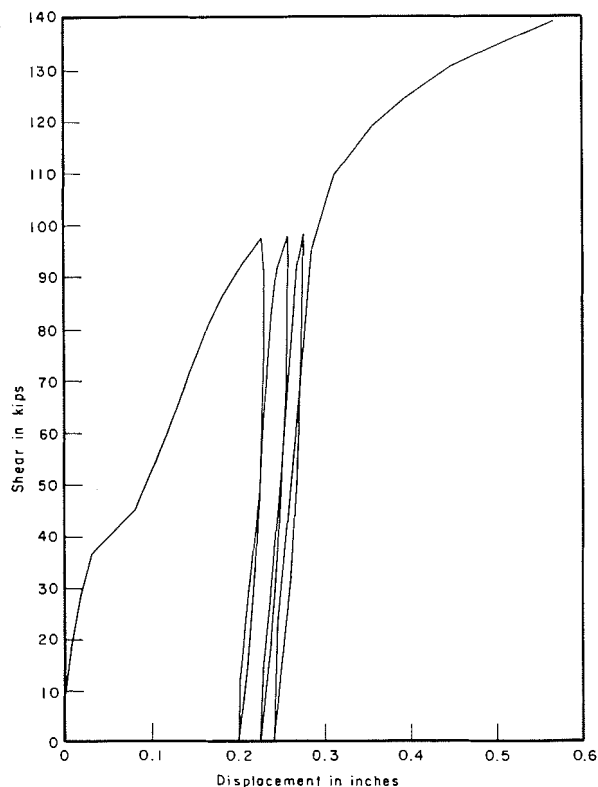


FIG. 7. Shear-displacement curve for four-bolt connection: series C. Notes: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

two-bolt connection is omitted from both sets of connections, the average failure load per bolt and the standard deviation are 35.2 and 2.06 kips (156.7 and 9.18 kN) for the slotted holes, and 35.0 and 3.36 kips (155.9 and 14.95 kN) for round holes, respectively.

Linear displacements of a point on the loading beam at the centre of each bolt group were calculated from the observed displacements of the top and bottom at the connected end of the beam, and these are shown in columns (6) and (7) of the table. Although the small jack supporting the free end of the beam was arranged in such a way that it offered little restraint for longitudinal displacement, such displacements were very small and, in all but specimen 9C, were less than 0.10 in. (2.54 mm). There does not appear to be any noticeable trend in the magnitudes of these horizontal displacements among the various sizes of connections. The vertical displacement, on the other hand, is considerably larger and appears, in the main, to vary with the magnitude of average shear per bolt. The rotation of the test beam was measured using the displacements of the pistons in the two jacks, and separately by means of two linear transducers mounted on the beam and bearing

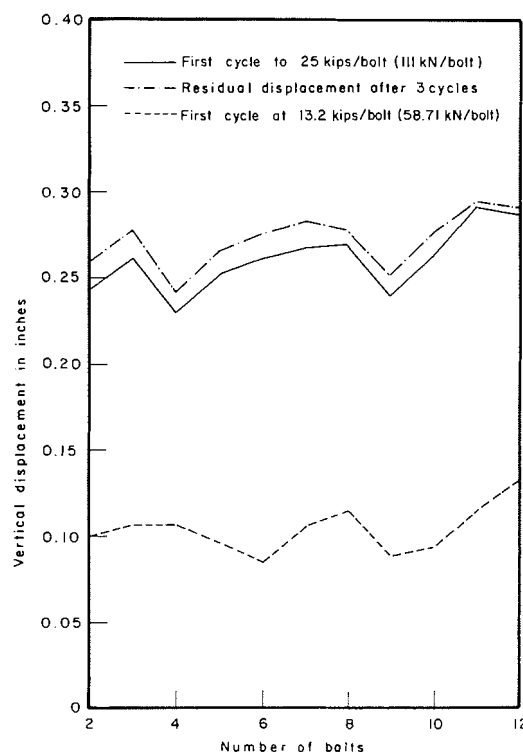


FIG. 8. Vertical displacements: series C. Notes: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

against the edge of the angle. At all loads, the rotation relative to the angle was very nearly equal to the absolute rotation, leading to the conclusion that the "flexibility" of the connection is in the slip of the beam relative to the angle and distortion of bolt holes and not in total deformations of the angle. The appearance of specimens 12C and 4C is shown in the photograph in Fig. 9, which shows clearly the local deformations at each slotted hole. Normal fabricating tolerances, no doubt, account for the small differences in the magnitudes of the individual deformations, but the effect of rotation is clearly visible in both specimens, in that at the top holes the deformations are further away from the column leg than at the bottom holes. All other specimens in this series exhibit a similar appearance.

Finally, the nominal shear stress in the beam leg of the angle was calculated by dividing the shear at failure by the cross-sectional area of the angle leg. The resulting values are in column (8) of Table 3. If the yield stress in shear is taken as  $1/\sqrt{3}$  of the tensile yield stress the resulting value is 28.2 ksi (194.1 kPa), which would indicate that all the angles should be yielding at least in some region. Visual inspection of all connections indicated at least some plastic deformation in addition to the local deformation at each



TABLE 3. Test results at failure: series C

| Specimen No.       | Shear at failure |                 |              | Displacements at failure, slotted holes |                     |                      | Average shear in angle (kips/in. <sup>2</sup> ) | Type of failure                            |
|--------------------|------------------|-----------------|--------------|---|---------------------|----------------------|---|--|
|                    | Slotted holes    |                 | Round holes* | Rotation (rad)                          | Vert. displ.† (in.) | Horiz. displ.† (in.) |   |  |
|                    | Total (kips)     | Per bolt (kips) |              |   |                     |                      |   |  |
| (1)                | (2)              | (3)             | (4)          | (5)                                     | (6)                 | (7)                  | (8)   | (9)  |
| 2C                 | 87.6             | 43.80           | 38.00        | 0.0197                                  | 0.800               | 0.089                | 42.5  | Excessive deformation of angle             |
| 3C                 | 107.9            | 35.97           | 38.83        | 0.0167                                  | 0.758               | 0.024                | 33.8  | All bolts sheared                          |
| 4C                 | 139.3            | 34.83           | 34.38        | 0.0201                                  | 0.543               | 0.015                | 32.3  | Bottom bolt tore angle, all others sheared |
| 5C                 | 176.9            | 35.38           | 38.00        | 0.0188                                  | 0.602               | 0.031                | 32.5  | Bottom bolt tore angle, all others sheared |
| 6C                 | 216.7            | 36.12           | 36.67        | 0.0188                                  | 0.633               | 0.053                | 33.0  | Bottom bolt tore angle, all others sheared |
| 7C                 | 254.7            | 36.39           | 35.19        | 0.0187                                  | 0.634               | 0.055                | 33.1  | Bottom bolt tore angle, all others sheared |
| 8C                 | 292.9            | 36.61           | 37.71        | 0.0188                                  | 0.686               | 0.083                | 33.2  | All bolts sheared                          |
| 9C                 | 339.7            | 37.74           | 37.15        | 0.0192                                  | 0.682               | 0.144                | 34.2  | All bolts sheared                          |
| 10C                | 359.0            | 35.90           | 33.50        | 0.0209                                  | 0.609               | 0.039                | 32.4  | Bottom bolt tore angle, all others sheared |
| 11C                | 357.4            | 32.49           | 30.00        | 0.0211                                  | 0.501               | 0.019                | 29.3  | All bolts sheared                          |
| 12C                | 370.1            | 30.84           | 29.03        | 0.0226                                  | 0.501               | 0.037                | 27.8  | Top bolt sheared                           |
| Mean               |                  | 36.01           | 35.33        |   |                     |                      |   |  |
| Standard deviation |                  | 3.09            | 3.24         |   |                     |                      |   |  |

NOTES: 1 kip = 4.45 kN; 1 in. = 25.4 mm.

\*Details of these tests will be reported separately.

†Linear displacements are of the point on loading beam at centre of connection.

hole. The deformation in specimen 2C was by far the largest.

The factor of safety provided in these connections for the allowable working load of 13.2 kips (58.7 kN) is on the average 2.73 and varies between a minimum of 2.34 for specimen 12C and a maximum of 3.32 for specimen 2C.

#### Test Results for Series D

The test results for series D connections are summarized in Table 4. The initial intention was to load this series of specimens in the same way as series C. It became obvious very quickly that this would not be possible because the welds were cracking at loads well below 25 kips (111 kN) per bolt so that a single cycle was used. At the "ultimate" load, the welds at both the toe and the heel of the angle were cracked in specimens 2D, 3D, and 6D, whereas in all others cracking occurred only at the toe, but not all the way to the bottom. Every attempt was made to identify the load at which the onset of weld cracking took place, but this proved to be extremely difficult visually and impossible from the appearance of moment-rotation or shear-displacement curves. Invariably the first noticeable sign of failure in the weld was a very fine hairline crack only a fraction of an inch in length at the top of the weld. It is quite

possible that the first beginning of any cracking occurred at shears below those listed in column (2) in Table 4. Certainly a crack was visible at the shear indicated. In specimens 2D and 6D the first crack was noticed at the heel, whereas in all others it was at the toe of the column leg. In columns (3) and (4) are shown the shear per bolt and the rotation at the first noticed weld crack. It is seen that the shear in almost all cases is well below 25 kips (111 kN) per bolt, the smallest value being 14.7 kips (65.4 kN) per bolt. The loading was continued beyond the onset of weld cracking to the preset maximum rotation, at which point the cracks had grown on both the toe and heel both in width and length, but the shear continued to grow. When loading was finally discontinued at the maximum rotation, the maximum shears were those given in columns (5) and (6) and the rotations were those in column (7). Only specimen 4D reached a shear of 25 kips (111 kN) per bolt, but with the exception of specimen 2D all had reached the maximum rotation. Three additional observations should be made. Firstly, there is a large spread in the average shear per bolt at which the first weld crack was noticeable, the range being 14.7–23.8 kips (65.4–105.9 kN) per bolt. Secondly, although the accepted design procedure indicates that the two-bolt connection is inadequate even for fric-

TABLE 4. Test results: series D

| Specimen No. | At onset of cracking |                       |                | At maximum shear   |                       |                | Maximum bolt moment (kip-in.) |
|--------------|----------------------|-----------------------|----------------|--------------------|-----------------------|----------------|-------------------------------|
|              | Total shear (kips)   | Shear per bolt (kips) | Rotation (rad) | Total shear (kips) | Shear per bolt (kips) | Rotation (rad) |                               |
| (1)          | (2)                  | (3)                   | (4)            | (5)                | (6)                   | (7)            | (8)                           |
| 2D           | 33                   | 16.5                  | 0.0144         | 47                 | 23.5                  | 0.0173         | -147                          |
| 3D           | 44                   | 14.7                  | 0.0146         | 70                 | 23.3                  | 0.0229         | -314                          |
| 4D           | 95                   | 23.8                  | 0.0218         | 100                | 25.0                  | 0.0226         | -431                          |
| 5D           | 113                  | 22.6                  | 0.0229         | 116                | 23.2                  | 0.0235         | -485                          |
| 6D           | 107                  | 17.8                  | 0.0166         | 143                | 23.8                  | 0.0222         | -589                          |
| 7D           | 148                  | 21.1                  | 0.0201         | 166                | 23.7                  | 0.0222         | -675                          |
| 8D           | 163                  | 20.4                  | 0.0193         | 191                | 23.9                  | 0.0225         | -849                          |
| 9D           | 158                  | 17.6                  | 0.0163         | 213                | 23.7                  | 0.0219         | -950                          |
| 10D          | 186                  | 18.6                  | 0.0167         | 236                | 23.6                  | 0.0209         | -1110                         |
| 11D          | 197                  | 17.9                  | 0.0164         | 252                | 22.9                  | 0.0210         | -1201                         |
| 12D          | 185                  | 15.4                  | 0.0156         | 253                | 21.1                  | 0.0216         | -1234                         |

NOTES: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

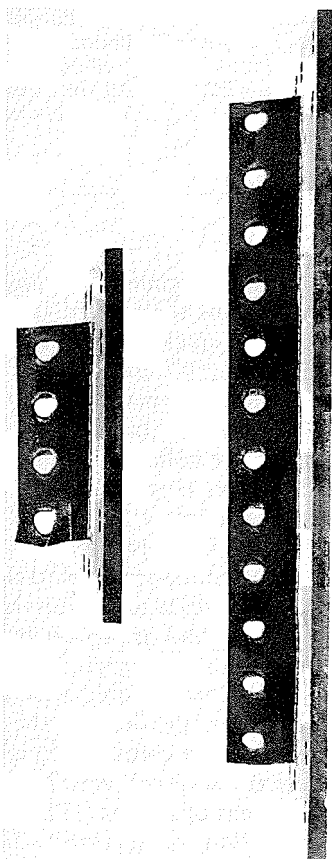


FIG. 9. Four- and 12-bolt connections after failure: series C.

tion bolts, while connections larger than six bolts are adequate for bearing bolts (with values approximately 50% greater than friction type), there is no clear trend in the value of shear at first weld crack. Thirdly, there is a much smaller spread in shear per

bolt at the maximum rotation, with the average value being 23.4 kips (104.1 kN) per bolt.

The observed rotation of the loading beam with respect to the angle never exceeded a value of 0.0027 rad for all but specimens 2D and 3D, indicating that the flexibility of the connection is provided by the deformation of the column leg of the angle. Furthermore, the signs of the bolt moments, which indicate a compression at the top of the beam and tension at the bottom, indicate that the effective eccentricity is smaller than the actual. In the case of specimens 2D and 3D the relative rotation of the beam with respect to the angle was observed to be of opposite sign to that of the absolute rotation of the beam, indicating that there was slipping between beam and angle. If the onset of cracking is used as the limit of usefulness, then series D welds do not provide either the desired reliability of an adequate factor of safety, or a sufficiently large enough rotation capacity at design working load.

#### Test Results for Series E

As in series D, the intention was to load these specimens in the same fashion as series C, but, as in series D, it became evident very early that this type of connection could not sustain three cycles of shear and rotation. For that reason the five-, seven-, nine-, and 11-bolt connections were loaded without rotation of the test beam. The results for this series are given in Table 5. The specimens are identified by two sets of digits and one letter. The digits before the letter correspond to the number of bolts in the connection; the letter identifies the series; and the last digit designates the presence or absence of rotation, the digit 2 indicating virtually zero rotation.

It was possible to obtain three cycles of loading up to 25 kips (111 kN) per bolt only in specimens 6E1

TABLE 5. Test results at failure: series E

| Specimen No. | Shear (kips) |             | Rotation (rad) | Deflection (in.) | Bolt moment (kip-in.) | Type of failure*   |
|--------------|--------------|-------------|----------------|------------------|-----------------------|--|
|              | Total        | Per bolt    |                |                  |                       |  |
| (1)<br>2E1   | (2)<br>34    | (3)<br>17.0 | (4)<br>0.0147  | (5)<br>0.453     | (6)<br>-137           | (7)<br>Entire weld with exception of about 1 in. at bottom corner of toe cracked           |
| 3E1          | 63           | 21.0        | 0.193          | 0.326            | -299                  | Entire weld failed   |
| 4E1          | 81           | 20.3        | 0.0186         | 0.302            | -338                  | Entire weld failed   |
| 5E2          | 148          | 29.6        | -0.0041        | 0.503            | -588                  | Top two bolts sheared and bottom bolt tore angle. Angle cracked at top weld                |
| 6E1          | 163          | 27.2        | 0.0232         | 0.427            | -858                  | Bottom bolt tore angle. Top of angle cracked at weld then top 7 in. of weld at toe cracked |
| 7E2          | 198          | 28.3        | -0.0040        | 0.502            | -797                  | Top bolt sheared, bottom bolt tore angle, and top of angle cracked at top weld             |
| 8E1          | 188          | 23.5        | 0.0219         | 0.329            | -734                  | Entire weld failed   |
| 9E2          | 274          | 30.4        | -0.0043        | 0.506            | -1153                 | Same as specimen 7E2   |
| 10E1         | 194          | 19.4        | 0.0181         | 0.267            | -712                  | Top 19 in. of weld at toe cracked  |
| 11E2         | 338          | 30.7        | -0.0044        | 0.565            | -1417                 | Same as specimen 7E2   |
| 12E1         | 235          | 19.6        | 0.0181         | 0.257            | -906                  | Top 11 in. of weld at toe cracked  |

NOTES: 1 in. = 25.4 mm; 1 kip = 4.45 kN.

\*In addition to the failures described in this column the weld at the heel of the angle cracked for its entire length.

and the four specimens that were loaded without rotation. In each of these cases, a crack in the weld at the heel of the angle and in the angle itself at the top weld appeared in the first cycle and grew progressively with successive load cycles. All other specimens failed in the first load cycle in the manner shown in the table. It is interesting to note that although both the yield and the ultimate stress for series E are about 12 and 16%, respectively, greater than for series C, in five of the 11 angles there was a crack in the angle itself. The column leg was purposely made 1 in. (25.4 mm) longer and 1/16 in. (1.6 mm) thinner to obtain the desired flexibility. As the connections were fabricated, however, the added width of the column angle was not entirely effective because the return weld at the top was invariably longer than the 1/2 in. (12.7 mm) that was specified. The shortest weld was 5/8 in. (15.9 mm) in specimen 4E1, and the longest was 1 3/4 in. (44.5 mm) in specimen 10E1.

The presence of rotation in the loading procedure reduces the ultimate load per bolt significantly, about 30% on the average. Similarly, the deflections at failure are also reduced substantially. The rotation characteristics of those specimens that were loaded with rotation were about the same as those in series D, whereas in those specimens that were tested with the loading beam in near zero position there was a slip to accommodate the deformation of the column leg of the angle. These tests indicate clearly that connections of this type are no better than those in series D.

Apparently, the usual design procedure, which assumes that the effect of out-of-plane eccentricity is negligible, cannot be used for designing the types of connection represented by series D and E. The

tendency of the heel of the angle to deflect away from the supporting member introduces a high stress in the weld at the toe of the angle, and a very high stress concentration at the top end of the weld. This appears to hold even for the case of near zero rotation of the end of the supported beam.

### Conclusion

The full-scale load tests on welded-bolted single-angle connections with "short slotted" holes described in this report show that these connections are as reliable as similar connections with round holes. They provide a factor of safety against the allowable working load in excess of 2.3 and develop moments at the bolts that are in all cases less than half of those in connections with round holes. Rotations of the supported beam end as much as about 0.022 rad are provided by the slip of the beam with respect to the angle starting at very low shears. Finally, single angles with weld patterns of the types used in series D and E sustain loads considerably lower than those carried by connections of the type in series C. In all 22 specimens loaded in series D and E the welds cracked at loads that varied widely and were as low as 14.7 kips (65.4 kN) per bolt.

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