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WELDED BEAM-COLUMN CONNECTIONS

by Inge Lyse* and G. J. Gibson**

INTRODUCTION

This investigation was sponsored by the Structural Steel Welding Committee of the American Welding Society of which Mr. Leon Moisseiff is chairman and Mr. W. Spraragan secretary; and was carried out at the Fritz Engineering Laboratory on a cooperative basis between this Committee and Lehigh University. Acknowledgment is due all members of this committee who assisted in the preparation of the program and rendered valuable assistance during the progress of the work and the preparation of this report.

In a previous report* the results of a cooperative investigation of the behavior of welded seat angles are presented. The present paper deals with beam-column connections designed for certain end restraints. Seat angles were chosen to carry the vertical load because they seemed to be the simplest and the most economical. The problem was to determine the type of connection which would restrain the beam and thus produce a negative moment at the end of the beam. The

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* AN INVESTIGATION OF WELDED SEAT ANGLE CONNECTIONS
  Inge Lyse & Norman G. Schreiner, October 1934
desired qualities of the connections were stiffness capable of giving an appreciable end moment, and flexibility sufficient to allow the end of the overloaded beam to rotate enough to insure flexural failure of the beam at its center before failure occurred in the connection. Another desired quality of the connection was its applicability over a large range of spans and sizes of beams.

Angles and plates joining the beam web to the column were not investigated because they seemed too expensive and their stiffness was limited to the stiffness of the web. The types of connections studied were top angles and plates welded along the top flange of the beam and to the face of the column as shown in Fig. 1. This type of top connection was chosen because the material is at a point where it is most effective in restraint, the welding is at a minimum and is all horizontal, and the design for restraint is relatively simple and applicable to various depths and lengths of beams. These types of top connection were first studied in a series of tension tests where the variations in size and thickness of the angles and plates could be studied extensively. It was expected that the action produced in these test rigs would, to a certain extent, give some indication of the action of a top angle or plate in a beam connection, because as the beam is loaded it deflects downward causing the ends to rotate. The top flange of the beam tends to move directly away from the face of the column causing a tension pull in the top connection which in turn produces the end restraint.
A closer study of the connection was made by a series of cantilever tests which are sketched in Fig. 1. The purpose of this series was to see if the action of the top connection was similar to the tension tests, to investigate the effect of the combined action of the seat angle and the top connection, and to determine the maximum strength and rotation at failure. A few tests were also made of web crippling of beams supported on seat angles to investigate the shearing strength of the connection.

A final study of the end connections was made on full-sized beam-column connections, also shown in Fig. 1.

In the series of tension tests the variables were held to one at a time. For the angles, the length of the horizontal leg was held constant at 3 in. throughout; the thickness was held constant at 3/8-in. in one group in which the length of the vertical leg was varied from 1 to 5 in.; the vertical leg was held constant at 3 in. when the thickness of the angle was varied from 1/4 to 3/4 in.; and 3 by 3 by 3/8 angles were tested to determine the effect of the length of the side weld. The angles were 6 in. long and were tested in pairs as shown in Fig. 1.

The variables studied in the tension tests of plates were the thickness and length of the plates, and the type of weld. The thickness was varied from 1/4 to 3/4 in., the different lengths were 3, 6, 12 and 24 in., and the plates were welded to the vertical face by either fillet or single V butt
welds. The tension rig used (Fig. 2) was very similar to that used for testing angles, only it was heavier.

Only the cantilever connections that were identical with the beam connections are included in this paper. The other variables studied in cantilevers will be included in a later report. Fig. 3 shows a regular beam-column test, five of which have been made to date. The set-up consisted of using a 3 12-26-lb. beam of an 18-ft. span framed between two stub columns. The columns were prevented from rotating by the arrangement shown, in order that the rigidity of the connection might be determined.

PREPARATION OF SPECIMENS

In the tension tests the angles were fastened to the rig by clamps, and welded in a tilted position with heavy coated electrodes. In all cases the size of the weld was made equal to the thickness of the angle. All angles were welded along the full length of both toes, and in the case of side weld tests, also along the edges of the vertical legs for specified lengths. No difficulties were encountered in welding; the quality of the weld metal was excellent, and the weld sizes were very uniform. After each specimen had broken the welds were cut off in a shaper and the rig was used over again.

For the plate specimens a special welding jig was used. The jig was mounted on pivots in a horizontal position so that each plate could be welded progressively to avoid warping of
the specimen. The welds were made in a horizontal position and built up to size by a series of string beads. Much difficulty was encountered in welding to the vertical plate because of the magnetic blow. A satisfactory fillet weld could be made but the quality of a single V butt weld was very uncertain. It was practically impossible to secure fusion within 1/8 in. of the root of the butt, and it was very difficult to keep slag pockets from forming in the upper part of the weld. In all cases the size of the fillet weld was at least equal to the thickness of the plate, and some of the butt welds were enlarged. When the specimen was broken the welds were cut off in the shaper and the rig used over again.

The cantilever specimens were welded as shown in Fig. 1. The lower flange of the beam was fillet welded to the seat angle to take the compressive thrust of the lower flange. When the top connection had failed, the welds were cut off in the shaper and new top connections welded in place. Four cantilever rigs were used; three being connected to plates and one to a stub column which had a flange thickness of 5/8 in. The difficulty of welding top plates to the face of the plate by means of butt welds was even greater than in the tension rig.

The complete beam-column specimens were prepared in the same manner as the cantilever. The top connections were removed by hand chipping, and replaced as before. This method made it possible to test a number of connections at a minimum expense.
TEST PROCEDURE

The tension tests for the angles were made in the 300,000-lb. Olsen testing machine. The angles and welds were whitewashed to indicate yielding, and observations were recorded. The deflections between the top and bottom sections of the rig were measured by Ames dials reading to \( \frac{1}{10,000} \) in.

When the deflections had exceeded 0.1-in. they were measured by a steel scale. The dials were attached as shown for the plate tension rig in Fig. 2. Identical observations were recorded on the plate and angle specimens, but in addition the yield point of the plates was taken at the drop of the beam of the testing machine. The plate tension rig was constructed to be used in the 800,000-lb. Richle machine because of the greater capacity required.

The cantilevers were tested in the 300,000-lb. Olsen machine in an upside-down position to simplify the set-up. Rockers were provided under the ends of the beams and a spherical bearing block was used in the center plate or column. Ames dials reading to \( \frac{1}{10,000} \) in. were attached to both sides of the web close to both flanges to measure the rotation of the connection. The plungers of the dials rested against the face of the column as shown in Fig. 4. Additional dials were placed as shown in the picture for observing the deformations of the flanges of the stub column.
The beam used in the full-sized connection was first tested as a simple beam to determine accurately its moment of inertia. With the connections in place, the rotations of the beam relative to the column were measured by Ames dials as described for the cantilever tests. The rotation of the column was measured by a level bar sensitive to 0.00002 radians. For stiff connections it was found necessary to jack up the short beams projecting from the columns to keep the column from rotating, because a small rotation of the column had a large effect on the stress distribution in a beam. Third-point loading was chosen for the beam because this most closely simulates uniform load. Whittemore strain gages were used for observation along both sides of each flange to determine the amount of restraint developed by the connection.

DISCUSSION OF RESULTS OF TENSION TESTS

In the tension test of top angles the initial scaling of the whitewash occurred on the throat of the weld on the vertical leg (the leg along the face of the column in the connection), at a load of about three-fourths the ultimate and at an average deflection at the heel of the angle of 0.1 of an inch. The next point of scaling was on the vertical leg of the angle at about the edge of the fillet, close to the maximum load and at a deflection of the heel of 1/4 in. All the 3 by 3-in. angles were remarkably tough and held the maximum load until an ultimate deflection that was always greater than 1/2-in. had occurred (see Fig.5). The 3/8-in. angles tested with
shorter legs, scaled on the horizontal leg at about the edge of the fillet, and their maximum deflection was somewhat less. The duplicate specimens agreed with the ultimate loads of the first set within five per cent, which was considered very satisfactory. The effect of length of the vertical leg on the stiffness and strength of the angle is shown in Fig. 6 and 7. Both the stiffness and the strength increased very markedly with the decrease in length of the vertical leg. Side welds caused a considerable difference in the action of the angle. The end of the side weld nearest the heel started to scale at low loads, and at the ultimate load the side welds start ripping at small deflections of the heel. When the side weld extended along the entire length of the vertical leg the angle was very stiff but had very little flexibility. The duplicate specimens showed wide variations with the first set as shown in Fig. 8. The concentration of stress at the end of the side weld, the uncertainty of strength, and the small flexibility seem to indicate that this type of weld is undesirable. Increased thickness of the angle increased the stiffness and strength of the angle to a very marked extent while the flexibility is still maintained.

The welds in the plate tension tests never developed more than two-thirds of the strength of thinner plates or more than one-half the strength of plates thicker than \( \frac{1}{3} \) in. This is principally due to the eccentricity of the weld on the vertical plate and to the initial welding stresses in the plates.
Just before failure the plates began to bow out about two inches from the weld which started to rip from the root. To avoid this eccentricity it was decided to try a single V butt weld. This gave much higher strength, but was more difficult to weld. In this case the failure was similar to the fillet specimens, because of the inability to secure fusion at the root, and of the high initial welding stresses in the plates. When fairly good fusion was secured in the butt welds, the failure occurred in the face plate which represents the face of the column. It seems that heat produced by welding destroyed the structure of the metal in the face plate and caused grain growth which weakened the metal to such an extent that chunks up to 3/8-in. deep were torn from the plate. It was decided to measure the initial welding stresses, and it was found that for plates thicker than 1/2 in. the surface stresses exceeded the yield point of the material and caused a decided bow in the plate which can be seen in Fig. 2. These initial stresses lowered the elastic limit to practically nothing for the thicker plates. The results were not consistent enough to show many trends of the variables studied. Neither type of welding developed the yield-point strength of plates thicker than 1/2-in. and there was never any failure of the thinner plates. So many difficulties and uncertainties were encountered in this study of welded plates, that they do not seem suitable for stiff beam connections where they must be depended upon to take the fixed end moments.
ANALYSIS OF CONNECTIONS

At this point it seems necessary to consider how the results of the tension tests can be applied quantitatively to beam connections. Referring to Fig. 9, it is shown how the end moment of a beam or cantilever is resisted by a couple consisting of a thrust located approximately at the top of the seat angle and a tension pull on the top angle located at the edge of the top leg. Undoubtedly the action is not as simple as that illustrated but the results seem close enough for all practical purposes. The rotation of the beam relative to the column was measured by the dial deflections \( y_1 + y_2 \). From these readings it is shown how the center of rotation, and the deflection of the top angle is computed for any force \( F \) on the angle. From these two values a load deflection diagram of the top angle can be plotted, and these diagrams form the basis of comparison of the action of the top angles in the tension, cantilever, and beam tests.

It will be explained farther along that yield-point strength of the angle is of great importance and a method of analysis of the yield strength of the angle is sketched in Fig. 10. The approximations involved in the analysis are based on observations of the tension and cantilever tests. The sketch of angle was traced from an actual tension specimen. The two critical points are the top weld and the section of the leg at the edge of the fillet of the angle. The whitewash first scaled at the weld so it was thought that by the time the
flexural stress of the angle had reached its yield point the bending stress distribution at the throat of the weld was approximately rectangular. The yield strength of the angle F was computed using the lever arm and the weld and angle moments computed as shown. The yield strength values for rectangular stress distribution in the angle was also figured out for comparison. The results are shown in tabular form in Table 2 and they are also plotted in Fig. 11 and will be discussed later.

The effect of the top connection on the moment distribution in the beam can be determined theoretically if the end moment is known for any rotation of the end of the beam. The effect is best measured quantitatively as degree of restraint designated as R.

\[
R = \frac{M}{MF} \quad \text{or} \quad R = 1 - \frac{\theta}{\theta_s}
\]

- \(M\) = end moment of a beam at a given load
- \(MF\) = fixed end moment for the same load
- \(\theta\) = end rotation of the beam
- \(\theta_s\) = end rotation of an unrestrained beam for the same load

In all cases the arithmetical sum of the end and center moments on the beam must be equal to the total applied moment which is determined from the amount and type of load. For a uniformly loaded beam:

\[
MT = M_c + M = \frac{WL}{8}
\]

The ideal conditions would be when the end and center moments were equal.
To develop ideal restraint the connection would have to let the end of the beam rotate a certain amount. This indicates a rather delicate design for ideal restraint because if the connection was too rigid the end of the beam would be overstressed, and if it were too flexible the center of the beam would be overstressed. However, since the maximum moment in the beam between fifty and one hundred per cent restraint is \( \frac{WL}{12} \) and the corresponding amount of end rotation of the beam has also the same range of variation, it would be comparatively easy to design a connection that would have a restraint within this range, while there would still be a saving of one-third the section modulus of the beam.

**DISCUSSION OF CANTILEVER TEST**

It was found that the effect of the seat angle on the connection was very noticeable when the lever arms of the cantilever was as short as twelve inches. The effect on the moment rotation relation was very much reduced as the lever arm was increased, so all comparison of the cantilever tests was made with lever arms of 32 or 36 in. which would be at the point of inflexion of an 18-ft. fixed beam. The force or load deflection relations of the top angle were computed and
plotted as explained in the analysis. For some reason they
did not check very well with the tension test but they com-
pared quite favorably with the beam test as shown in Fig. 12.
The angles tested in tension took a considerably larger load
before they reached their yield point, but the ultimate
strength and the maximum deflection were practically the
same as in the cantilever tests. A summary of the results
is shown in Table 1.

DISCUSSION OF COMPLETE BEAM-TO-COLUMN CONNECTIONS

The beam used was first tested as a simple beam to
determine its section modulus. Assuming a 29.5 million modu-
lus, center deflections by the mirror-wire and scale method
gave a section modulus of 54.0. Whittemore strain readings
gave 35.9, while the handbook value was 35.6. The seat angles
used were 6 by 6 by 1/2 in. by 7-1/2 in. long. The 6-in. out-
standing leg was chosen so as not to over-stress the fillet
welds between the lower flange and the seat by the compressive
thrust due to the end restraint.

The strain data taken along the beam is a direct check
on the end restraint developed by the connection. A sample of
the data for one load is shown in Fig. 13. A similar graph is
plotted for every load, and from them the degree of restraint
can be taken from either the point of inflection or from the
end moments. The total calculated applied moments checked out
with the total measured moments within four per cent.
In these tests the columns were kept plumb by the jacks under the short beams of the columns as shown in Fig. 3. The restraint computed from the end rotation checked the restraint measured by the strain measurements fairly closely. When these restraints were plotted against the load on the beam as shown in Fig. 15, the degree of restraint fell off as the load increased. This effect is especially noticeable in the case of the lighter top angles. The decrease is due to the fact that when the top angle reaches its yield point the end moment remains practically constant and the increased load is taken by the moment in the center of the beam. An attempt was made to test the beam to destruction but the test proved a failure because the beam was not supported laterally. The only way to prove that the connections were flexible enough to insure the beam to fail first in the center is the calculation summarized in Table 1. The yield point of the top angle was taken arbitrarily at a deflection corresponding to a stress of 18,000 p.s.i. in the beam. Making the conservative assumptions that the end moment remains constant at this value of deflection and that the point of rotation of the end of the beam is at the seat angle, the maximum deflection of the top angle can be computed at the failure of the beam which is assumed to be at the yield point of the flanges or at approximately twice the working load resisting moment. When these computed values are compared with the maximum deflections
at failure as taken from the cantilever tests, it is seen that there is a large margin of safety against failure of the connection. Fig. 11 shows that the margin of safety on strength of the top angles is considerable.

PROPOSED METHOD OF DESIGN

Fig. 14 shows that the reduction in stress by the use of top angles is considerable and is worth designing for. As discussed under the analysis the most practical restraint to design for lies in the range between fifty and one hundred percent restraint when the maximum moment is \( \frac{WL}{12} \) for uniform load. This was readily obtained for the particular beam investigated by 3 by 3 by 3/4 by 6-1/2 in. top angles (Fig. 15). Fig. 12 shows that the angles had reached their yield point when the stress in the beam was 18,000 p.s.i., and since the top angles do not take on much load once they have passed the yield point, it seems conservative to assume that the angles only take their yield-point strength at working stresses on the beam. The proposed method of design for uniformly loaded beam would follow these steps.

1. Determine the size of the beam from the load and span based on the maximum moment of \( \frac{WL}{12} \).

2. The end moment necessary for fifty per cent \( A \) is at least \( \frac{WL}{24} \). From this end moment determine the force couple necessary to produce it.
3. Dividing this Force by the width of the top flange will give the yield-point load per inch of top angle.

4. The load per inch will determine the size and the angle and weld necessary which should be computed on the basis of rectangular bending stress distribution on the weld and triangular on the angle. Fig. 11 shows that these assumptions are all conservative because the yield points of the angles from actual tests of cantilevers and beams all fall somewhat above the computed values. A comparison of design and test results is tabulated in Table 2.

The range of weight, depth, and span of beams to which this method of design is applicable is the program contemplated for further investigation by the authors. Obviously a top angle would not have an appreciable effect on a very heavy beam or girder. A very long span would not give enough margin of safety on the deflection of the top angle and very short spans will not develop the yield point of the angles, but there seems to be a definite range of light-weight beams where this method would be applicable.

CONCLUSIONS

The results obtained in this investigation indicated that:

A. Tension Tests.

1. The stiffness and the strength of the angles increased with the decrease in length of the vertical leg.

2. The stiffness and strength of angles increased very appreciably with the increase in thickness of the angles.
3. Welds of size equal to the thickness of the angle produced yielding of the angle before failure.
4. Angles with vertical legs of 3 in. or more deflected 1/2 in. or more before the load decreased.
5. Side welds on angles were uncertain because of heavy stress concentrations at the ends of the side welds.
6. The results of butt and fillet welded plates showed that this type of connection was not advisable.

B. Cantilever Tests.
1. Top angles yielded at lower computed tensile loads than in the tension tests, but the strengths were approximately equal.
2. The maximum deflection of the top angles was about the same as in the tension tests.
3. For top angle connections the center of rotation of the beam was about at the seat angle.
4. A considerable reserved strength was found in the angle connections when compared with the proposed design methods.

C. Complete Beam-Column Tests.
1. The reduction in stress of the beam due to the restraint offered by the top angles was very pronounced (Fig.14).
2. Top angle connections were flexible enough to insure failure in the center of the beam instead of in the angle connection.
3. An end rigidity of at least fifty per cent was readily obtained when thick top angles were used.
<table>
<thead>
<tr>
<th>Top Angle</th>
<th>Top Weld</th>
<th>Lever Arm</th>
<th>Point of Rotation Above Seat At Y.P. Load</th>
<th>Defl. of Top Angle At Failure</th>
<th>Ultimate Load lb.</th>
<th>Ultimate Force Per Inch Of Angle p.s.i.</th>
<th>Y.P. Force Per Inch Of Angle lb.</th>
<th>Deflection of Top Angle At: 18,000 p.s.i. Failure</th>
<th>Y.P. Force By Beam Tests</th>
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<tr>
<td>3/8 x 6-1/2</td>
<td>3/8</td>
<td>32</td>
<td>1/2</td>
<td>7/8</td>
<td>11,900</td>
<td>1950</td>
<td>850</td>
<td>0.098</td>
<td>0.160</td>
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<tr>
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<td>1/2</td>
<td>36</td>
<td>1/2</td>
<td>3/4</td>
<td>12,300</td>
<td>2450</td>
<td>1300</td>
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<tr>
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<td>5/8</td>
<td>32</td>
<td>1-1/2</td>
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**TABLE 2**

**COMPARISON OF DESIGN AND TEST RESULTS**

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<tr>
<th>Angle Thickness (in.)</th>
<th>Weld Size (in.)</th>
<th>$F_{\text{a}}$ ($\Delta A$)</th>
<th>$F_{\text{a}}$ ($\Delta A$)</th>
<th>$F_{\text{a}}$ ($\Delta A$)</th>
<th>Calculated Y.P. Force Per Inch Of Angle</th>
<th>Design Load</th>
<th>Per Cent N</th>
<th>Design Load</th>
<th>Per Cent N</th>
<th>Working Load By Test</th>
<th>Per Cent R By Test</th>
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</thead>
<tbody>
<tr>
<td>3/8</td>
<td>3/8</td>
<td>300</td>
<td>350</td>
<td>530</td>
<td>830 $\Delta A$</td>
<td>19,700</td>
<td>13</td>
<td>20,200</td>
<td>17</td>
<td>21,700</td>
<td>23</td>
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<tr>
<td>1/2</td>
<td>1/2</td>
<td>1570</td>
<td>670</td>
<td>1000</td>
<td>1570 $\Delta A$</td>
<td>21,300</td>
<td>24</td>
<td>22,200</td>
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<td>27,600</td>
<td>49</td>
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<tr>
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<td>1100</td>
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<td>2610 $\Delta A$</td>
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<td>36</td>
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<td>42</td>
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<td>1750</td>
<td>2630</td>
<td>4130 $\Delta A$</td>
<td>26,800</td>
<td>49</td>
<td>29,200</td>
<td>57</td>
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<td>70</td>
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<tr>
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<td>3/4</td>
<td>1500</td>
<td>2740</td>
<td>4120</td>
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<td>33,700</td>
<td>70</td>
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<tr>
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<td>3/4</td>
<td>1500</td>
<td>3890</td>
<td>5840</td>
<td>7340 $\Delta A$</td>
<td>32,500</td>
<td>67</td>
<td>37,900</td>
<td>79</td>
<td>35,000</td>
<td>74</td>
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Assumed yield point of weld metal = 40,000 p.s.i.  
Assumed yield point of angles metal = 35,000 p.s.i.  
Seat Angles 6 x 6 x 1/2 by 7-1/4 long  
Top Angles all 3 x 3 x 6-1/2