PROGRESS REPORT ON WELDED BEAM-COLUMN CONNECTIONS

by Glenn J. Gibson*

SCOPE AND PURPOSE OF INVESTIGATION

This investigation which is a continuation of the seat angle investigation of welded beam-to-column connections, was made to secure experimental information from which a rational method of design could be found for flexible, semi-rigid, and rigid joints. The types of connections studied were top angles and top plates for beams supported by seat angles. For simplicity the top angles and plates were first tested in a direct tension set-up, in which various lengths and thicknesses could be studied extensively. The connection as a whole was studied in a series of cantilever tests which simulated the action of the top connection in the beam more closely than the tension test, and a final series of complete beam tests was introduced to check on the relative stiffness of the various top connections, to check the theory of partial restraint, and to indicate the factor of safety of the connection. The connections tested were designed to be applicable over a wide range of size of beams and span length. The problem resolved itself into a correlation of the three series of tests, and the application of the results to the design of connections for a definite rigidity.

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TEST PROGRAM

Many investigations have been carried out in the field of beam connections both in the riveted and welded type. In this investigation an attempt was made to avoid duplication of welded connections previously studied.

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 used for finding 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. It was expected that the action produced in this test rig would, to a certain extent, simulate the action of the top angle 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, and the top connection will restrain this movement. All angle specimens were tested in duplicate to check on the uniformity of results.

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. 1) was very similar to that used for testing angles, only it was heavier.

Eleven connections were tested as cantilevers using 12-in. beams and 1/2-in. seat angles with outstanding legs of 4, 6, or 8 in. A double cantilever arrangement was used, with plate or stub column connections also shown in Fig. 1. Various lengths of lever arm were used in a study of the combined action of seat angles with top connections. A few tests were also made of web crippling of beams supported on seat angles.

Fig. 1 shows a regular beam-column test, three of which have been made to date. The set-up consisted of using a B 12-28-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 more readily.

PREPARATION OF SPECIMENS

The angles were fastened in the tension 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 jig with the tension specimen clamped in place is shown in Fig. 2. The welds were made in the flat 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 made 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 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 on the location and distribution of the scaling at various loads. 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. 3. 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. Riehle 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 on 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 complete 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 Jop 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. 6). The 3/3-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. 7 and 8. 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. 9. 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 as shown in Fig. 10, while the flexibility is still maintained.
The fillet welded plates broke at much lower loads than what specifications call for (see Table 1). 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 start to bow out very decidedly about two inches from the weld which starts ripping 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 destroys the structure of the metal in the face plate and causes grain growth which weakens the metal to such an extent that chunks up to 5/8-in. deep are 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. 3. These initial stresses lowered the elastic limit to practically nothing for the thicker plates as shown in Fig. 11. The results were not consistent enough to show many trends of the variable studied, although Fig. 12 indicates that the type of welding has a much greater effect on the stress-strain relations than has the length of the plates. The strength table shows that thick plates
developed relatively less strength than thinner ones. So many difficulties and uncertainties were encountered in the 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.

THEORETICAL CONSIDERATIONS

At this point it seems necessary to consider how the results of the tension tests can be applied quantitatively to the beam connections. The effect of the top connection on the moment distribution in the beam can be determined theoretically if we know the end moment developed for any rotation of the end of the beam. These relations are shown very clearly in Fig.13* where it can be seen that for 75 per cent restraint the maximum moment is cut in half. Degree of restraint or percentage of rigidity may be defined as:

\[ R = \frac{M}{M_F} \quad \text{or} \quad R = 1 - \frac{\Theta}{\Theta_A} \]

\( M \) = end moment of a beam at a given load
\( M_F \) = fixed end moment for the same load
\( \Theta \) = end rotation of the beam
\( \Theta_A \) = end rotation of an unrestrained beam for 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. For a uniformly loaded beam:

\[ M_T = M_C + M = \frac{WL}{8} \]

* Taken from the paper: RELATIVE RIGIDITY OF WELDED AND RIVETED CONNECTIONS by C.R. Young and M.B. Jackson
The ideal condition would be when the center moment was equal to the end moments.

\[ M_0 = M \quad \text{then} \quad 2M = \frac{WL}{8} \implies M = \frac{WL}{16} \]

\[ MF = \frac{WL}{12} \quad \text{so} \quad R = \frac{M}{MF} = \frac{\frac{WL}{16}}{\frac{WL}{12}} = \frac{12}{16} = 0.75 \]

Then end rotation for any corresponding load and end moment is given by the general equation:

\[ \theta = \frac{WL^2}{24EI} - \frac{ML}{2EI} \]

from which the design of a connection would have to be based. The problem of design would probably have to follow the following steps:

1. Decide on a definite restraint.
2. From the load and degree of restraint determine the end and center moments and the size of the beam from the maximum moment.
3. Compute the end rotation by equation (a).
4. Design a connection that will develop the required end moment while permitting the computed rotation to take place.

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 was too flexible the center would be overstressed. Fig. 13 shows that the maximum moment between restraints of fifty and one hundred percent is \( \frac{WL}{12} \).
would be comparatively easy to design a joint that would have a restraint within this wide range, while there would still be a saving of one-third the section modulus of the beam.

The cantilever investigations were carried out to determine the moment-rotation characteristics of the top connection combined with the seat angles, and to see if the characteristics of the tension tests applied to the action of the top connections in the cantilever. The computations to make the comparison are shown in Fig. 14.

Complete beam-to-column connections would have to be carried out to determine whether (1) the end moment-rotation characteristics were the same as by the cantilever system, (2) the end rotation did give the correct degree of restraint, (3) the action of the top connection was the same as in the tension tests. The comparisons were made by the same method of computation as in the cantilever tests except that the end moments were taken from the strain data on the beam. Third-point loading was chosen for the test beams because for the same connection and the same maximum stress in the beam, the degree of restraint would be identical with a uniformly loaded beam, and only the load on the test beam would have to be multiplied by 4/3 to give the corresponding load on the uniformly loaded beam. The relation between end moment and rotation for third-point loading is given by the expression: \[ \theta = \frac{WL^2}{16EI} - \frac{ML}{2EI} \]
DISCUSSION OF RESULTS OF THE CANTILEVER TESTS

In the first four cantilevers tested an attempt was made to keep the end shear and moment conditions approximately the same as in a complete beam. This attempt involved the use of short lever arms which indicated a much higher strength and stiffness of the top connection than the tension tests gave (see Table 2). The reason for this discrepancy was thought to be due to the fact that the reaction of the seat angle on the beam was not at the face of the column but somewhere out along the outstanding leg of the seat. To locate this reaction, cantilevers were tested at different lever arms within the elastic limit of the connection. For a constant applied moment the rotation was decreased as the lever arm was decreased. Fig. 15 shows this for a top plate connection, and Fig. 16 for a top angle connection. Assuming that the entire effect is due to the reaction being out a distance \( x \) from the face of the column, and assuming that \( x \) has a constant value for each connection, then the rotations will be directly proportional to the effective moment which is considered to be:

\[
\frac{W}{2} (\text{Lever Arm} - x)
\]

\[
\frac{W_1 (L_1 - x)}{W_2 (L_2 - x)} = \frac{\theta_1}{\theta_2}
\]

Solving for \( x \) for various combinations of lever arms, the results came out between 5.8 and 6.3 in. for the top plate cantilever, which seems strange because the outstanding leg of
the seat angle was only 4 in. long. However, when all of the lever arms were reduced 5.8 in. and the corrected moments computed and plotted against the rotations, they all fell on the same line as shown in Fig. 17. The same procedure was carried through for a more flexible top connection and the value of the lever arm reduction varied between 4.5 and 6.0 in. When the applied moment is plotted against the lever arm for a constant rotation the curve approaches an asymptote at approximately 5 in. (see Fig. 18). When this correction is made to the applied moments which are plotted against the rotations, the points fall fairly well on one line which is shown in Fig. 19. The causes for this action were difficult to determine, but they were probably due to a combination of the following effects: (1) the top connection might be taking some of the vertical load especially for the thicker angles, (2) the lower flange of the beam is welded rigidly to the seat angle, (3) the moments in the welds of the top connection might cause the different distribution of stress than that assumed in computation.

The results show that less error from these effects will be present if the cantilevers are tested with long lever arms, and all comparisons made with the tension tests were taken from such cantilevers. Fig. 20 and 21 show the comparison of cantilever tests with tension tests. The angles in cantilevers yielded at a lower load than in tension, but the ultimate strengths were approximately the same (see Fig. 10). The
comparisons of top plates show very comparable stiffness, but
the strength is much higher in the cantilever tests. A sum-
mmary of results is tabulated in Table 2. The strength and
stiffness comparisons check better with the unreduced moments
which indicates that more investigation needs to be done to
solve this problem.

So far only end moment conditions had been considered
and the question of shear was neglected. The cantilevers were
finally connected by top plates that could take very little
shear, and tested with short lever arms until the web cripped
above the seat. Fig. 22 shows a specimen after failure due to
web crippling, in which each web was tested separately by ec-
centric loading. The space between the top plate and the top
flange indicates the amount of vertical deflection. The sum-
mary of results in Table 3 indicates that when seat angles are
welded to the lower flange of the beam the permissible reactions
might be taken the same as for a stiffened seat, because there
was no sign of failure of the seat angle or welds up to the ul-
timate load. More tests along this line might be worthwhile to
justify some conclusions and method of design.

DISCUSSION OF COMPLETE BEAM-TO-COLUMN TESTS

The beam used was first tested as a simple beam to de-
termine its section modulus. Assuming a 29.5 million modulus,
center deflections by the mirror-wise and scale method gave a
section modulus of 34.0. Whittemore strain readings gave 35.9,
while the handbook value = 35.6. The seat angles used were 6 by 6 by 1/2 in. by 7-1/2 in. long. The 6-in. outstanding leg was chosen so as not to overstress 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 an absolute check on the end restraint developed by the connection. A sample of the data for one load is shown in Fig. 23. 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.

The end rotations did not give as reliable data as the strain readings. The columns tilted one-quarter as much as the relative rotation between the beam and the column for the 5/8-in. top angle connection, and the column rotation had to be added to give the rotation of the end of the beam. The restraints computed from the beam rotations did not agree very well with the measured restraints, especially for the more rigid connections as shown in Fig. 24. This indicates that in succeeding tests the columns should be kept plumb for every load and that greater refinement is necessary in measuring the rotations.

A comparison of the action of the top angles between tension, cantilever, and complete beam tests is shown in Fig. 25. The beam test checks the cantilever test for 5/8-in. top angles, and the cantilever test checks the tension test for 3/8
in angles. Obviously more tests are needed to find any definite trends. The point of rotation of the beam above the seat angle closely checks the cantilever tests.

The problem of the factor of safety of top angles is a rather complicated one. Fig. 25 shows that at a design stress of 18,000 p.s.i. in the beam, the top angle has passed its yield point, but it must be understood that the factor of safety of a connection of this type depends on its flexibility as well as on its strength. The tension and cantilever tests proved that 3 by 3-in. top angles did not fail until the deflection of the heel was at least 1/2 in., while holding the yield-point load. The diagram shows that at working loads the deflection of the most flexible angle was only 0.067 in. All additional load is carried by increase in stress at the center of the beam. Remembering that beams fail when the flange stress reached the yield point, the maximum deflection of the heel of the angle from the column face at the ultimate load could be computed by assuming that the end moment remains constant at the yield-point load of the angle, and the point of rotation was at the seat angle which would give the worst conditions. The design conditions in Table 4 are taken from data while the ultimate load conditions are computed with the above assumptions. The beam fails in the center by yielding due to flexural stress at a load causing a deflection in the top angles of only one-third their maximum which shows that for this particular span the top angle connection would not fail.
The test beam was stressed until there was yielding in the flanges due to lateral deformation, but no sign of failure occurred in the top angles.

This argument would not apply to rigid top plate connections which are not flexible because the welds could not develop the yield point of the plates with any certainty or satisfactory factor of safety. If used, they should be designed for complete restraint with cognizance of the effect of eccentricity of the welds. For a factor of safety of 4 on the welds for the beam studied, a strength of 33,000 lb per in. would have to be developed for the extreme case of full rigidity. This strength was much greater than any attained in the tension test of plates.

**SUMMARY OF RESULTS**

The results obtained to date may be summarized as follows:

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 strength of butt and fillet welded plates did not give a factor of safety of 4 on the design loads of the welds.

7. The type of weld had more effect on the unit strain in the plates than had the length of the plates.

8. High initial welding stresses were found in the welded plates.

9. It was found to be very difficult to make a single V butt weld between the plate and the flange of the columns.

B. Cantilever Tests.

The center of reaction of the seat angle on the beam seemed to be out near the end of the leg of the seat.

2. Top angles yielded at lower computed tensile loads than in the tension tests, but the strengths were approximately equal.
3. The maximum deflection of the top angles was about the same as in the tension tests.

4. The center of rotation of the beam was about at the seat for top angle connections, and about one-third the depth of the beam above the seat for top plate connections.

5. Web crippling of the beam occurred before there was any sign of failure in the seat angle or welds for the particular connections studied.

C. Complete Beam-Column Tests.

1. The reduction in stress of the beam due to the restraint offered by the top angles was worth considering (see Fig. 26).

2. Flexible top angles had little reserve strength, but were flexible enough to ensure 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 1
STRENGTH OF BUTT AND FILLET WELDED PLATES

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate Strength per inch of weld</th>
<th>Maximum Stress in Plates</th>
<th>Welding Strains in Plates 2 in. from Welds in millionths</th>
<th>Remarks</th>
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<tbody>
<tr>
<td>P 24</td>
<td>6,250</td>
<td>25,000</td>
<td></td>
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</tr>
<tr>
<td>P 31</td>
<td>11,600</td>
<td>31,000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P 31</td>
<td>19,700</td>
<td>52,500</td>
<td>770</td>
<td>Plate passed yield point</td>
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<tr>
<td>P 32</td>
<td>7,200</td>
<td>19,200</td>
<td>740</td>
<td></td>
</tr>
<tr>
<td>P 32</td>
<td>15,700</td>
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<tr>
<td>P 33</td>
<td>7,750</td>
<td>20,700</td>
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<td>P 33</td>
<td>15,800</td>
<td>42,200</td>
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<tr>
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<td></td>
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<tr>
<td>P 52</td>
<td>23,000</td>
<td>36,800</td>
<td>2500</td>
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<td>P 62</td>
<td>23,900</td>
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<tr>
<td>P 63</td>
<td>14,800</td>
<td>19,700</td>
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<tr>
<td>P 63</td>
<td>28,200</td>
<td>37,600</td>
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</tr>
</tbody>
</table>

- P indicates Fillet Weld
- P- indicates Butt Weld
- First Number indicates thickness of plate in eighths
- Second Number indicates length of plate
  - 1 = 3 in.
  - 2 = 6 in.
  - 3 = 12 in.
  - 4 = 24 in.

Welded in Tilted Position
Butt weld enlarged to 3/4 in.
Plate passed yield point
Plate passed yield point
Plate passed yield point
Mo wire used, failure in rig
Plate passed yield point
Mo wire
Butt weld enlarged to 1-1/8 in.

All Fillet Welds showed good fusion
All Butt Welds had poor fusion at the root
### Table 2

**Summary of Cantilever Tests**

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
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<tr>
<td>CA 3</td>
<td>3/8 x 8</td>
<td>9</td>
<td>63,600</td>
<td>0.3</td>
<td>0.0425</td>
<td>288,000</td>
<td>2,310</td>
<td>119,000</td>
<td>960</td>
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<td>CA 7</td>
<td>3/8 x 6-1/2</td>
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<td>1,900</td>
<td>159,000</td>
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<td>12,300</td>
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<td>587,000</td>
<td>5,300</td>
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<td>5/8 x 6-1/2</td>
<td>32</td>
<td>28,500</td>
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<td>456,000</td>
<td>4,550</td>
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<td>7,040</td>
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<td>6,430</td>
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<td>5,430</td>
</tr>
<tr>
<td>CP 1</td>
<td>Butt Welded</td>
<td>36</td>
<td>41,600</td>
<td>4.0</td>
<td></td>
<td>750,000</td>
<td>15,200</td>
<td>639,000</td>
<td>13,000</td>
</tr>
<tr>
<td>CP 3</td>
<td>1/2 x 6</td>
<td>36</td>
<td>57,700</td>
<td></td>
<td></td>
<td>1,040,000</td>
<td>13,900</td>
<td>894,000</td>
<td>11,800</td>
</tr>
<tr>
<td>CP 4</td>
<td>MC Mild. wire</td>
<td>1/2 x 6</td>
<td>36</td>
<td>63,000</td>
<td>1,220,000</td>
<td>15,000</td>
<td>968,000</td>
<td>12,900</td>
<td></td>
</tr>
</tbody>
</table>

Lugs of all angles 3 x 3 in.
Rotation of a 12-in. 13-ft. span simple beam at failure = 0.0110
CA indicates Top Angle Connection
CP indicates Top Plate Connection
### TABLE 3

**WEB CRIPPLING ON SEAT ANGLES**

<table>
<thead>
<tr>
<th>Beam</th>
<th>Web Thickness</th>
<th>Seat Angle Thickness</th>
<th>Outstanding Leg</th>
<th>Length of Bearing</th>
<th>Maximum Reaction</th>
<th>Design Reaction For Stiffened Seat A.I.S.C.</th>
<th>Reaction Per Inch of Bearing</th>
</tr>
</thead>
<tbody>
<tr>
<td>B 12-45</td>
<td>0.336</td>
<td>1/2 x 4</td>
<td></td>
<td>3-1/2</td>
<td>97,500</td>
<td>32,400</td>
<td>27,800</td>
</tr>
<tr>
<td>B 12-28</td>
<td>.240</td>
<td>1/2 x 4</td>
<td></td>
<td>3-1/2</td>
<td>44,500</td>
<td>19,800</td>
<td>12,700</td>
</tr>
<tr>
<td>B 12-28</td>
<td>.240</td>
<td>1/2 x 6</td>
<td></td>
<td>5-1/2</td>
<td>51,900</td>
<td>25,900</td>
<td>9,450</td>
</tr>
<tr>
<td>B 12-28</td>
<td>.240</td>
<td>1/2 x 8</td>
<td></td>
<td>7-1/2</td>
<td>55,400</td>
<td>32,000</td>
<td>7,400</td>
</tr>
</tbody>
</table>

Lower flanges welded along seat

\[
f_b = \frac{18,000}{1 + \frac{a}{6000 t^2}}
\]

\[
R = f_b t (a + \frac{d}{4})
\]

*a = length of bearing*

*t = web thickness*

*d = depth of beam*
Fig. 1.
Types of Test Specimens

Fig. 2
Jig for Welding Plate Specimens
Fig. 3
Plate Tension Specimen Ready for Testing

Fig. 4
Cantilever Specimen Ready for Testing
Fig. 5
The Complete Beam-Column Specimen

Fig. 6
Angle Tension Specimen After Failure
Fig 7

Relative Stiffness with Variable Length of Vertical Leg

Thickness of Angles Constant of \( \frac{1}{16} \)

Load in lbs. per inch of angle

Deflection of heel in thousandths of an inch
Fig 8

Effect of Length of Vertical Leg on Strength of Angle

Angle Thickness Constant at $\frac{3}{8}$

Ultimate Load per inch of angle in pounds

4000
3500
3000
2500
2000
1500
1000
500

Length of Vertical Leg in inches

0 1 2 3 4 5
Effect of Side Welds

Ultimate Load per Angle in lbs.

3 x 3 x 3/8" Angles
6 in. Long

Length of side welds in inches
Fig. 10

Effect of the Thickness of the Angle on the Strength

- Strength in Tension Test
- \( \times \) Computed Strength from Cantilevers

Angles with 3 in. Legs

Ultimate Load per inch of angle in pounds

- 0
- 1000
- 2000
- 3000
- 4000
- 5000
- 6000

Thickness of angle in eighths of an inch
Fig. 11

Stress-Strain Diagrams for Butt Welded Plates

Plates 6" long with variable thickness

Unit strain in thousandths
Stress in Pounds per Square Inch

Stress Strain Diagrams
for Welded Plates

Unit Strain in Thousandths
<table>
<thead>
<tr>
<th>Coefficient of Restraint</th>
<th>Type of Support</th>
<th>Deflection Diagram</th>
<th>Moment Diagram</th>
<th>Maximum Moment Diagram</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple Beam</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Partial Restraint</td>
<td>0.25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Partial Restraint</td>
<td>0.30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Partial Restraint</td>
<td>0.75</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Full Restraint</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Over Restraint</td>
<td>1.25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Two Cantilevers</td>
<td>1.50</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Fig. 13**

Effect of End Restraint

**Fig. 22**

Shear Failure by Web Crippling
Method of Analysis of Cantilever Connections

\[ F = \frac{W_2 \times \text{Lever Arm}}{f} \]

Corr. \( F = \frac{W_2 \times \text{Corrected L.A.}}{f} \)

\( F = \text{Force on Top Angle} \)

\[ \theta = \frac{(y_1 - y_2)}{b} \]

\[ e = a - \frac{y_2}{\theta} \]

\[ c = (d - e)\theta \]

\( y_1 + y_2 \) Dial Deflections

\( e \) Rotation of Beam in Radians

\( e \) Center of Rotation above Seat

\( c \) Deflection of Top Connection
Moment Rotation Chart

CPI

Cantilever with a Top Plate

Fig 15

Applied Moment in Kips inches

Rotation in Radians x 10^-6
Moment Rotation Chart

Corrected \( M = \frac{\text{Load}}{2} (\text{Lever Arms - 5.8\,'}\) \\

Symbol | Lever Arm
--- | ---
\( \cdot \) | 36 inch
\( \times \) | 27 inch
\( \times \) | 18 inch
\( \times \) | 9 inch

Rotations in Radians \( \times 10^{-6} \)
Fig. 18

Effect of Length of Lever Arm

Applied Moment in lb in

Lever Arm in inches

\[ 0.376 \text{ radians} \]

180°
Corrected moment in kip inches

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Lever Arm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>36 inch</td>
</tr>
<tr>
<td></td>
<td>27 inch</td>
</tr>
<tr>
<td></td>
<td>18 inch</td>
</tr>
<tr>
<td></td>
<td>12 inch</td>
</tr>
<tr>
<td></td>
<td>9 inch</td>
</tr>
<tr>
<td></td>
<td>7 inch</td>
</tr>
</tbody>
</table>

Moment Rotation Chart

Corrected \( M = \frac{4}{\pi d} (\text{Lever Arm} - 6"))

Rotation in radians \( \times 10^6 \)

Fig 19
Fig. 20
Relative Stiffness of Angles
With Variable Thickness

- From Tension Tests
-- From Cantilever Tests

Load in lbs. per inch of angle

Deflection of heel in thousands of an inch
Stress Strain Diagrams of Plates in Tension and in Cantilevers
Stress Distribution in Flanges

B12 23\(^{\circ}\) Beam 9\(\frac{3}{8}\) Top Angles
Load = 20,000\(\ast\)

Distance from East Column in Feet
Fig 24
Restraint of Top Angle Connectors
1/2 28" Beam 18 ft Span
Third Point Loading

Load on Beam

30000
25000
20000
15000
10000
5000

3/8" Top Angle 6 ft Long
By End Plate

3/8" Top Angle 6 ft Long
By End Tabular

3/8" Top Angle 6 ft Long
By End Tabular

Percentage Rigidly
Relative Stiffness of Top Angles

- Tension Test
- Cantilever Test
- Complete Beam Test

Deflection of Heel in Thousandths
Effect of Top Angles on Stress in Beam

B/12 23° Beam 18 ft. Span
Third Point Loading

Load on Beam in Pounds

Shear in Beam