WELDED INTERIOR BEAM-COLUMN PROJECT

FOUR-WAY WELDED INTERIOR BEAM-COLUMN CONNECTIONS

by

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This is Progress Report No. 2 of an investigation at Lehigh University sponsored by the American Institute of Steel Construction.

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Fritz Engineering Laboratory
LEHIGH UNIVERSITY
Bethlehem, Pennsylvania
September 1957

Fritz Laboratory Report No. 233.13
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FOUR-WAY WELDED INTERIOR BEAM-COLUMN CONNECTIONS

1. INTRODUCTION

This report is on the second phase of a research project on Welded Interior Beam-Column Connections. The main purpose of this phase is to determine the behavior of the four-way connections, whereas the overall purpose is to determine when stiffening is required, and, if so, how to design it. The first phase of the project was limited to study of "two-way" direct welded connections, and is reported in Progress Report No. 1(13). In this second part of the project the addition of two beams framing into the column web was made both by direct welded and by top-plate and stiffened seat type of connections. The test procedure was in many respects similar to that used in the two-way connections.

The column sections were chosen similar to those used in the two-way, so as to compare, if possible, the behavior of each four-way beam-column connection with its two-way counterpart. As far as known, no four-way tests have been made prior to these. The interest, therefore, is centered in the comparative behavior of the two and four-way tests, namely whether or not the triaxial stresses in column web would cause premature failure, or whether a beneficial effect might be obtained in the unstiffened connections through the partial stiffening action of the beams framing onto the column web, the latter applying to the four-way connections which are unstiffened. The criterion for a satisfactory connection established in Part I has been retained and is stated completely under THEORETICAL ANALYSIS.
**Fig. 1 - GENERAL VIEW OF TEST IN PROGRESS**

**Fig. 2 - LATERAL SUPPORT FOR NORTH AND SOUTH BEAMS**
2. TEST PROGRAM

This program consisted of three specimens with details as shown in Table I and in Figure 4. Test AA is similar to Test A-14 of the "Two-Way" series except for two additional 16WF36 beams framing into the column web and directly welded thereto. In the same manner Test DD is similar to Test D-12 of the Two-Way Series. Test BB was exploratory in nature and does not have its two-way counterpart. The beams framing to the column flanges were 16WF36 as before and were direct-welded, but the other pair of beams were 12WF27, the tension flanges of which were welded to horizontally placed column plate stiffeners, the compression flanges resting on a tee-type seat which also acted as a column stiffener (but 4" away from its ideal location as a stiffener).

TABLE I - PROGRAM OF FOUR-WAY WELDED BEAM-COLUMN TESTS

<table>
<thead>
<tr>
<th>Test</th>
<th>Column Size</th>
<th>Web Thickness</th>
<th>Flange Thickness</th>
<th>Beam Size</th>
<th>Web Thickness</th>
<th>Flange Thickness</th>
<th>Stiffener</th>
<th>Stiffener Dimensions</th>
</tr>
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<tbody>
<tr>
<td>AA</td>
<td>12WF65</td>
<td>0.39</td>
<td>0.606</td>
<td>16WF36</td>
<td>0.299</td>
<td>0.428</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>BB</td>
<td>12WF40</td>
<td>0.294</td>
<td>0.516</td>
<td>16WF36</td>
<td>0.299</td>
<td>0.428</td>
<td>Horiz. plates that served as top plate and as seat (plate)</td>
<td>1/2 &quot; thick</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>12WF27</td>
<td>0.240</td>
<td>0.400</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DD</td>
<td>12WF40</td>
<td>0.294</td>
<td>0.516</td>
<td>16WF36</td>
<td>0.299</td>
<td>0.428</td>
<td>Split tee stiffener</td>
<td>ST6WF32,5 22&quot; long</td>
</tr>
</tbody>
</table>
3. TEST ARRANGEMENT

The specimens were fabricated of the WF sections indicated in Table 1, the beams being each 4'3" long and the columns 9 ft. long.

The testing was done in the 5 million pound Baldwin-Hamilton machine which provided ample space for placing these specimens and for the lateral supports, Figure 1 showing a test in progress.

The specimens were placed in an inverted position in the testing machine to permit the use of mechanical jacks through which loads were applied to the beams. The magnitude of these loads was determined by the use of calibrated dynamometers connected to a strain indicator.

The column load was read by a load capsule in the crosshead of the machine. Thus this load-indicating device gives readings which represent the sum total of the loads acting on the upper half of the column as situated in the machine. When load is applied to the beams through the jacks, the total load acting on the upper half of the column is the sum of the four beam loads plus the axial load existing in the lower half of the column. Inversely, the lower half of the column will be under an axial load of the total axial load minus the sum of the four beam loads. This condition of loading had to be watched in the course of the test to avoid a "floating" or unstable case. This could have occurred if the sum of the four beam loads is equal to the load indicated by the machine i.e. a case whereby the lower half of the column is under no load and free to kick out from its normal position. This "floating" condition did not exist in the specimen AA, but would normally have existed for the BB and DD specimens using 12WF40 columns since the adopted working column stress of 14.5 ksi times the area of a 12WF40 is less than the sum of the maximum beam loads. By exceeding the sum of the beam loads on the upper half of the column, by about 20 kips, it was assured that the
Fig. 3 - TEST ARRANGEMENT
Test AA
Column 12WF65
Beams: 16WF36 on Flanges
16WF36 on Web
Direct Welded

Test BB
Column 12WF40
Beams: 16WF36 on Flanges
12WF27 on Web
1/2" Top Plate
and Stiffened Seat

Test DD
Column 12WF40
Beams: 16WF36 on Flanges
16WF36 on 12WF65
Split Tee Stiffener
(ST6WF32.5)

Fig. 4 - THE FOUR-WAY SERIES OF BEAM-COLUMN CONNECTIONS
lower half of the column always had enough axial load to supply the contact pressure needed for frictional forces preventing the column base from shifting. Thus with 52 kips on each beam it was necessary to place 226 kips on the column causing an axial stress of 19.2 Kσi instead of the desired 14.5 Kσi. This merely made the test somewhat more severe on the column.

Figure 3 is a drawing of the test arrangement, righted to show the positioning of loads as found in a typical building connection. Note that in this drawing it is the lower section of the column that was subjected to an axial stress of 14.5 Kσi in Test AA and that this stress had to be exceeded somewhat for Tests BB and DD to be assured that the top of the column was in firm contact with the testing machine to prevent possible lateral shifting should the jacking loads be applied unequally.

3.1 Fabrication:

All welds were in accordance with the A.W.S. specifications and were made by a qualified weldor. The general procedure for welding was as follows:

1. Beams and column flame cut to length.
2. Base and cap plates fillet-welded to the column.
3. Stiffener plates welded to the beams at the jacking points.
4. Column stiffeners, if specified, welded in place. In Specimen DD, for the welding of the stems of the ST6WF32.5 stiffeners to the column webs, the column was rotated 45° to permit making the filled welds in the true downhand position.
5. Beam flanges flame bevelled for butt welds, and cut-outs made in web to allow for insertion of backing plate for
the top flange butt welds and for welding past the web in the bottom flange weld.

6. Beams fitted square to the column flanges; $1'' \times \frac{1}{4}''$ backup plates tack welded to beam flanges with $3/16''$ min. root gap; plates carried out beyond edge of flange.

7. Butt welds formed at beam-column interface; $1/4''$ fillet welds between the beam web and the column. All welding in the downhand position using $3/16''$ dia. E-6012 electrodes for the first pass, and E-6020 electrodes for successive passes.

8. For Specimens AA and DD, a repeat of items (6) & (7) for the direct welding of the second pair of beams.

9. For Specimen BB, web beams (12WF27) tack-welded to tee seats with $1/2''$ clearance between beam and column web.
   a. Welds completed between beams and tee-seats.
   b. Top stiffener plates positioned, and tacked to column and to beams, and welds completed.

3.2 Strain Measurements:

Extensive use was made of SR-4 gages to measure strains in the specimens, the three instrumentation plans shown in Figures 5, 6, and 7 showing the location and type of gages. The A-1 type was used to measure strains in the beam flanges and for column alignment in all three tests. The A-5 type was used to measure the strains in the column web and split tee stiffener and in all places where space called for the use of a shorter gage length. In Specimen BB the AX-5 type was used, as shown, to measure strains in a vertical and horizontal direction in the column web.
Fig. 5 - INSTRUMENTATION PLAN: TEST AA
3.3 Deflections:

A precise Wild level was used to measure the vertical deflection of the beam ends. Scales were mounted vertically on the beam flanges and were easily read to the nearest 0.01". These are shown in Figure 1.

3.4 Local Buckling:

A dial gage was used to measure the local buckling of the beam compression flanges. The gage had extension rods with pointed ends which were inserted in pairs of center punch marks on the outer edge of the beam compression flange and on the fillet of the beam web and tension flange as indicated in Figures 5, 6, and 7. The relative change in length between these two points is a measure of the local buckling. Readings were taken in all the four beams of each test at a distance of 6" from the face of either column flange or web.

3.5 Lateral Buckling:

Lateral support was provided by anchoring the compression flange of the beams welded to the column flanges to the frame of the testing machine by means of tie rods at points 3" from the column as shown in Figure 1. For the beams framing to the column web, lateral support was provided by confining the free ends of the compression flanges of those beams (see Figure 2) within two pairs of angles connected to the frame of the testing machine. The maximum magnitude of the lateral forces as measured by the lateral dynamometers on the tie rods was about 2 kips.

3.6 Rotation Measurements:

Dial gages mounted on rotation posts, shown in Figure 9 were
used to measure the overall rotation of the connection and the rotation of the beams in a region very near to the connection. In the AA test the rotation of the column web was also measured.

Rotation is expressed in radians per inch. This was calculated by adding the changes in dial readings of the tension and compression portions of the beams and dividing by the product of the vertical distance between the dial gages and the horizontal distance between the rotation posts.

The rotation data was taken part way through the inelastic range to a point where local buckling in the beam near the rotation post caused obviously erroneous readings.

3.7 Test Procedure:

The first step in the test was to align the column in the testing machine. This was accomplished by placing a trial load on the column and then observing the strains in the four SR-4 gages located at the same level on the outer edge of the column flanges and shifting the alignment plates in the cross head of the testing machine until the variation was not greater than 10% at column working load.

The sequence of loading in the tests was the same as that used in the two-way connections:

1) The column load was increased in five equal increments to working load, \( P_w \), with no load on the beams. (This load was the same in the upper and lower portions of the column).

2) The beam load was increased to 6 kips and then in 3 kip increments to working load, \( V_w \), while maintaining working load, \( P_w \), in the column at all times in the portion of the column below the connection (as in a building, see Fig. 3).

3) The column was subjected to a first overload of 1.65 times
the working load in three equal increments with working load maintained in the beams. The column load was then reduced to working load.

4) The beams were loaded in increments until failure of the connection occurred. The connections were considered to have failed and the testing concluded when the decrease in beam load was about 15% below the ultimate unless weld failures took place before that stage was reached.

5) The second column overload of twice the working load was applied after the beams had received their last jack loads and the specimen test ended.

In the elastic range, the loads were applied in small increments to one pair of beams at a time. The deflections of the beams, SR-4 readings, local buckling, rotation dials and lateral support readings were taken and the whitewash patterns recorded at 6 kip increments of beam load. In the inelastic range the equal deflections were jacked into opposing beams and the loads read at suitable intervals.

Control curves of beam load vs. end deflections were maintained throughout the test.
4. TEST RESULTS

The results of the three tests conducted on this four-way connection portion of the project are plotted and illustrate the following points of interest in the behavior of the connection.

1. Beam load vs. beam deflections
2. Moment-rotation curves for the welded connection
3. Stress distribution in the beam flanges
4. Stress distribution in the stiffened connection of specimen DD
5. Stress distribution in the column web of Specimen BB
6. Comparison graph of load vs. deflections in terms for general comparison and for comparison with counterpart tests of the two-way series.

4.1 Deflections and Mode of Failure: Test AA

In Specimen AA, under equal jack loads the two beams welded to the flanges of the column (the east and west beams) deflected more below $P_y$ than the other two beams as shown in Figure 10. Local yield lines were noted in the column web near the juncture of the compression beam flanges, one of which frames to the column flange and the other to the column web. These yield lines were well developed at a beam load of 42 kips as shown in Figure 9 (yield lines are partially obscured by the rotation bars and dials). Local buckling of the beam flanges was noticed at a load of 53 kips in the beams framing to the column flanges and at a slightly higher load in the beams framing to the column web. The buckling occurred at the beginning of strain hardening range and the specimen was able to carry increased load. The falling off of the beam loads was rather slow. When the beam loads had fallen off to 15% of $V_u$, twice working load was applied to the column, the whitewash indicating
Fig. 8 - SPECIMEN AA AT END OF TEST

Fig. 9 - SPECIMEN AA, SHOWING YIELD LINES ON COLUMN WEB OPPOSITE THE BEAM COMPRESSION FLANGE
Fig. 10 - BEAM LOAD VS. END DEFLECTIONS: TEST AA
that the column suffered considerable yielding, but there was no other evidence of failure in the column.

4.2 Deflections and Mode of Failure: Test DD

Specimen DD with its 12WF65 split tee stiffeners exhibited a strong stiff connection, see Figure 11. The two beams that were connected to the stiffeners had very good load and rotation capacities, but the east and west beams that were connected to the column flanges just reached the required ultimate load and showed a lesser rotation capacity caused by a butt weld failure starting at a load of 49 kips. The first crack occurred in the west beam at the interface between the column flanges and the end of the butt weld to the beam tension flange and increased until weld failure penetrated to the fillet welds connecting the beam web to the column flange as shown in Figure 12. At that stage the beam load was 42 kips (having dropped after reaching maximum of 52 kips) and the whole tension flange of the west beam was separated from the column flange, tearing away with it part of the column flange steel. The east beam did not suffer this severe condition of weld failure. The tension flange butt welds of the north and south beams, connected to the stiffeners, had very small cracks starting at a load of 55 kips, but they did not progress any further, and the beam compression flanges buckled as shown in Figure 14. As the last deflection increment was being applied to those beams (the load on the south beam was 40 kips and that on the north beam was 50.6 kips with equal deflections of 5 inches) a sudden snapping fracture took place in the butt welds of the tension flanges. The stiffened connection consisting of the split tee welded to the column did not suffer any weld failure or excessive deformations.
Fig. 11 - BEAM LOAD VS. END DEFLECTIONS: TEST DD
Fig. 12 - SPECIMEN DD, SHOWING FAILURE AT BUTT WELD OF WEST BEAM

Fig. 13 - SPECIMEN DD, SHOWING ROTATION DIALS JUST PRIOR TO REMOVAL
Fig. 14 - FAILURE DETAILS, TEST DD
The beams did not undergo as much local buckling as in the AA and BB tests.

4.3 **Deflections and Mode of Failure: Test BB**

This specimen, consisting of a pair of 16WF36 beams direct-welded to the column flanges, and a pair of 12WF27 beams each connected to the column web by a tee seat and top plate, both also serving as column stiffeners, exhibited excellent load and rotation capacities as shown by Figure 15. The beams eventually suffered local buckling in the compression flanges; no weld failures occurred. Figure 16 shows the local yield lines and buckling which took place.

4.4 **Rotation Measurements**

The moment-rotation \( (M - \phi) \) curves for the three tests are shown in Figure 17. In calculating the applied moment a lever arm of four feet was used for the 16WF36 beams and 32 inches for the 12WF27 beams, these being the distances from the point of load application to the contact point with the column or to the seat as the case may be. Rotations were plotted in terms of the angle change per inch of horizontal distance between the gage supports. In this figure the overall rotations of the connection of Specimens AA, BB and DD are shown, and for AA the column web and welded connection rotations, as explained in the sketch in Figure 17, are also shown. The experimental data are compared with the theoretical moment-rotation characteristics of the beams based on an idealized stress-strain and \( M - \phi \) relationship.

These moment-rotation results are in general agreement with those found in Progress Report No. 1. In discussing the curves in Figure 17 from left to right the following observations are made:
Fig. 15 - BEAM LOAD VS. END DEFLECTIONS: TEST BB
Fig. 16 - FAILURE DETAILS, TEST BB
Fig. 17—COMPARISON OF COLUMN AND CONNECTION ROTATION CHARACTERISTICS WITH THAT FOR THE BEAMS: TESTS AA, BB, DD
1. For Test BB the connection involving the 16WF36 beams, welded directly to the column flanges, proved to be rather stiff. Evidently the direct stiffening at the tension flanges and the indirect stiffening 1½" off center at the compression flanges rather effectively prevented the column web from rotating much under the applied moments. The connection involving the 12WF27 beams framing to the seats and top plates was considerably more flexible than an equivalent 12WF27; however, it may be noted that this flexibility did not prevent the connection from fully meeting the established criteria for a satisfactory connection.

2. For Test DD, the connection involving the beams welded directly to the column flanges proved stiffer than the other one. In the plan view of Figure 7 it may be observed that the stiffness of the other connection, that is welded to the split tee stiffeners, is mainly dependent on the thickness of the stem of the tee stiffener, the flanges of the column being too far away to be very effective. On the other hand, the column web is ably assisted in preventing rotation at the connection by the flanges of the split tee stiffeners.

3. For Test AA, the connection involving the beams welded to the column flanges, the column web portion of the connection was actually stiffened by the flanges of the other pair of beams and as a consequence proved to be much stiffer than the welded connection. The welded connection in this case included 3" of the beam, the column flange, and about 1" of the column web (unstiffened).

In comparing the overall unit rotations for Test AA, the
obvious results obtained, namely that the beams directly welded to the column web and subjected to equal opposing moments provided a stiff connection while the other connection, with only partial stiffening provided, showed considerable flexibility. As noted elsewhere, the connection passed the criteria for a satisfactory connection.

4.5 Stress Distribution in Beam Flanges and Column Web

The distribution of the stress in the beam flanges for Test AA was plotted for each beam flange (Figure 18 & 19) and was determined by the use of longitudinal strain gages. The stresses were plotted with working load $V_w$ and $1.5 V_w$ on the beams, and were obtained by multiplying the unit strain by the modulus of elasticity. In the case of inelastic strains the strain plot was first drawn; this was then converted to a stress plot by an appropriate change of scale and by cutting off the strains where they crossed the yield stress of the material. High stress concentrations at the center of the beam tension flanges (Figure 18) were observed at $1.5 V_w$. In the beam compression flanges, Figure 19, the stresses were more uniform, but evidently a small lateral moment existed also, as shown by the higher stresses on one edge of the beam flanges as compared to the opposite edge. In Figure 20 the stress distribution in the column web is shown, but it was impossible to show the stress distribution on the center line of the web all through because of the welded beam webs framing to the column web.

The above procedure was used to plot the stress distribution in the beam flanges of Test DD and the results are given in Figures 21 and 22. These plots do not indicate any high stress concentrations in the center of the beam flanges, but indicate, rather, the existence of a lateral moment as shown in Figure 21 by the higher stresses on one edge of the beam tension flanges.
Fig. 18 - STRESS DISTRIBUTION IN BEAM TENSION FLANGES; TEST AA
Fig. 19 - STRESS DISTRIBUTION IN BEAM COMPRESSION FLANGES: TEST AA
Fig. 20 - STRESS DISTRIBUTION IN COLUMN WEB: TEST AA
Fig. 21—STRESS DISTRIBUTION IN BEAM TENSION FLANGES: TEST DD
Fig. 22 - STRESS DISTRIBUTION IN BEAM COMPRESSION FLANGES; TEST DD
The plot of the stress distribution in the stiffened connection gives an indication of the stress concentrations which must, as a consequence, have existed in the webs of the column and the stem of the split tee stiffener, thus indicating that the major portion of the beam flange thrust or tension is taken by the inner members of the cellular-type connection. This is verified by three series of gages, shown on Figure 23 by Sections a-a, b-b, and c-c, which of necessity were not placed at the levels of the beam flanges, but 1.5" above or below them as the case may be. While there is some scatter of the results, the fact of high stresses on the inner members of the cellular-type connection is clearly indicated.

Following the above procedures for Tests AA and DD, the plots showing stress distribution in the flanges of Test BB are given in Figures 24 and 25. Figure 24 indicates that in the beams framing onto the column flanges there is a scattering of the test results which appears to indicate that the stresses in the flanges of these two beams are influenced by the actions of the other pair of beams, such as a Poisson effect and the effect of any lateral moments accidentally occurring. The results in Figure 25 of the distribution of stress in the compression flanges indicate a fairly uniform distribution of stress.

The stress distribution in the column web, Test BB, is given in Figure 26. The SR-4 gages in Sections a-a and b-b were placed to map out, if possible, the stress distribution in the column web opposite the beam compression flanges framing onto the column flanges. The results for Section b-b were disappointing in that the SR-4 gages for 1.5 Vw were obviously not functioning right, but for Section a-a, high stresses were found by the gage immediately opposite the beam flange, the yield point being exceeded at 1.5 Vw. The stress plot for Section c-c is not significant. Two of the connecting lines are shown dotted merely to tie together corresponding load points; obviously some
All data plotted are at a level of 1.5 inches away from the beam flanges, except the VΔ which are at the level of beam flanges. The plotted VΔ are best shown here as unit strain x 30,000 ksi rather than as unit stresses.

Fig. 23 - STRESS DISTRIBUTION IN THE STIFFENED CONNECTION; TEST DD
Fig. 24 - STRESS DISTRIBUTION IN BEAM TENSION FLANGES: TEST BB
Fig. 25 - STRESS DISTRIBUTION IN BEAM COMPRESSION FLANGES: TEST BB
Fig. 26 - STRESS DISTRIBUTION IN COLUMN WEB: TEST BB
greater stress existed in the column web between these gages which were impracticable to obtain.
5. THEORETICAL ANALYSIS

The criterion for a satisfactory connection, stated in Progress Report No. 1(13), has been retained, namely "one which is capable of developing the theoretical maximum moment of resistance of the beams (the "plastic moment") when working axial load is on the column. A desirable additional quality of a satisfactory connection is one that maintains its moment capacity for a considerable rotation past the ultimate load."

Two analyses were presented in Progress Report No. 1(13) to the two-way beam-column connections of that report, both of which were put in suitable form for use in design. The first one, called the Modified A.I.S.C. Method acknowledges that the small area of inelastic strains in the column web immediately opposite the beam flange (Figure 27) is not dangerous, that it merely causes a redistribution or spreading out of the stresses as they are transmitted to the column web. The resulting basic formula, a modification of A.I.S.C. Par. 26(h) is as follows:

\[ \sigma = \frac{R}{t(N+4k)} \]  
where \( \sigma \) must not exceed 24 Ksi

(1)

Notation:  
- \( R \) = beam flange area x 20 Ksi  
- \( t \) = column web thickness  
- \( N \) = beam flange thickness  
- \( k \) = distance from outside of column flange to web toe of fillet (value obtained in AISC Manual)

The above formula presumes no column stiffeners and is in fact the formula to determine whether or not they are required. Should stiffeners be present the denominator, which is the effective web area of the column, must be suitably increased to include the effective area of the stiffeners. In Progress Report No. 1 the above formula was placed in form to compute the required thickness of column web, or of the stiffeners, as the case may
FIG. 27 - Analysis of Column Web Stresses by Modified AISC Approach

FIG. 28 - Analysis of Column Web Stresses by Plastic Analysis Approach
These are given here as follows:

For unstiffened columns:
\[ t_c = \frac{R(\text{in kips})}{2h(N+4k)} \]  

(2)

For horizontal plate stiffeners:
\[ t_s = \frac{(R/2h) - tc(N+4k)}{b} \]  

(3)

For vertical plate stiffeners:
\[ t_s = \frac{1}{2} \left[ \frac{R/2h}{N+4k} - t_c \right] \]  

(4)

Notation: 
- \( t_c \) = column web thickness
- \( t_s \) = stiffener plate thickness
- \( b \) = beam flange width

When Formula (1) is applied to the three tests of this four-way series the following results are obtained:

<table>
<thead>
<tr>
<th>Test</th>
<th>R</th>
<th>( t_c )</th>
<th>N</th>
<th>k</th>
<th>( t(N+4k) )</th>
<th>( \sigma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>AA</td>
<td>60.4</td>
<td>0.395</td>
<td>0.426</td>
<td>1.188</td>
<td>2.044</td>
<td>29.5</td>
</tr>
<tr>
<td>DD</td>
<td>58.0</td>
<td>0.317</td>
<td>0.414</td>
<td>1.125</td>
<td>1.557</td>
<td>37.2</td>
</tr>
<tr>
<td>BB</td>
<td>57.8</td>
<td>0.316</td>
<td>0.413</td>
<td>1.125</td>
<td>1.553</td>
<td>37.2</td>
</tr>
</tbody>
</table>

The Modified A.I.S.C. Method, according to these \( \sigma \) values, requires stiffeners for all three tests. Since no stiffeners were provided for Test AA and since the connection met the criteria for a satisfactory connection there is indication that this method is conservative.

The second analysis presented in Progress Report No. 1 is named the Plastic Analysis Method. This approach is shown in Figure 28. It assumes a stress distribution in the beam, loaded to its capacity, \( M_p \), as shown in Section a-a, and a corresponding stress distribution in the column web at the end of the flange-to-web fillet as shown by Section b-b. The following formulas result:
For unstiffened columns

\[ t_c = \frac{A_b}{d_b+2k} \]  

(5)

For horizontal plate stiffeners

\[ t_s = \frac{1}{2b} \left[ A_b - (d_b+2k)t_c \right] \]  

(6)

For vertical plate stiffeners

\[ t_s = \frac{1}{4} \left[ \frac{A_b-t_c(d_b+2k)}{N+2k} \right] \]  

(7)

Notation:  \( A_b \) = area of beam section  
\( d_b \) = depth of beam

The above two methods of analysis have been applied to the four-way tests by proper substitution of values in formulas (2) to (7), and the required thicknesses determined, either of the column web in the unstiffened connection of Test AA, or of the stiffeners in the case of Tests BB and DD. The results are given below in Table III.

Table III. Comparison of Analytical Methods with Actual Results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Req'd: ( t_c ) or ( t_s )</th>
<th>Actual</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mod. A.I.S.C. Plastic Analysis</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AA</td>
<td>( 0.49'' (t_c) ) 0.56'' (t_c)</td>
<td>0.39''(t_c)</td>
<td>Test passed criteria</td>
</tr>
<tr>
<td>BB</td>
<td>( 0.25'' (t_s)* ) 0.25'' (t_s)</td>
<td>0.5'' (Top plate opposite beam tension flange). 0.5'' (Seat at 4'' above beam compression flange).</td>
<td>Test passed criteria. No sign of overstress in any stiffeners.</td>
</tr>
<tr>
<td>DD</td>
<td>( 0.25'' (t_s)* ) 0.46'' (t_s)</td>
<td>0.60</td>
<td>Test passed criteria.</td>
</tr>
</tbody>
</table>

* Thickness governed by AISC Par. 18 (c) that thickness shall be at least \( 1/40 \) of clear distance between column flanges.
A study of the values in Table III show that for Test AA both methods appear to be conservative. Evidently the beam flanges that were directly welded to the column web offered considerable stiffening even though they did not extend the full distance between column flanges. For Tests BB and DD no comparisons are afforded in that the stiffeners were thicker than required and in the tests showed no sign of overstress. In Test BB neither method has provision for the case where the column stiffeners are not positioned directly opposite the beam compression flanges. From a study of the stress distribution in the unstiffened column web opposite the beam flanges, which was developed in Progress Report No. 1, employing the Boussinesq formula for a semi-infinite mass subjected to a strip loading, it would appear that a horizontal stiffener to be 100% effective, should be directly opposite the beam flange. It loses effectiveness then dependent on its distance from this optimum position. It is recalled from Part I that crippling of the column web due to the stress concentrations is not the only mode of failure but that the entire column web between the compression flanges may eventually buckle. A stiffener 1/4" off center of pressure serves two purposes in that it assists in carrying the load concentrations through a re-distribution of the stress, and it materially resists any tendency of the column web to buckle.

Admittedly the present analyses do not cover the situation of stiffeners positioned eccentrically with respect to the beam flanges they are supposed to service. Further tests are planned for this purpose.
6. **DISCUSSION**

All three specimens passed the criteria by (1) possessing the strength to develop the theoretical plastic beam moment, and (2) showing considerable rotation capacity at peak loads. Test AA, as shown in Figure 29, was stronger than its two-way counterpart, Test A-4. This evidently shows that the gain obtained by the stiffening action provided by the two beams framing onto the column web outweighed the effects of the triaxial stresses which obtained in the four-way tests.

The comparison of Test AA with its counterpart the A-4 test indicates, as a first impression, that no stiffening of the column in Test AA is required. This is true only under the special conditions, namely using beams of equal depth and subjecting the beams to equal loads. In actual buildings, however, the case is quite different, the opposing moments being unequal. The column web is unfavorably situated to transmit localized normal thrusts without the aid of diaphragms. This becomes accentuated when the beams framing onto the column web are deeper or shallower than the flange beams.

Test DD is compared in Figure 29 with its two-way counterpart D-12. In both tests the split beam stiffeners effectively prevented any buckling or crippling of the connection. Test BB had no two-way counterpart.

Not mentioned under THEORETICAL ANALYSIS is an additional item required in the design of the split-beam type stiffeners. As shown in Figure 30 the design moment, \( M \), is to be resisted by the couple set up by the two pairs of fillet welds connecting the stem of the stiffener to the column web.

\[
M = T_f \text{ or } C_f
\]

whence the welds are easily designed. Having designed the weld sizes the thickness of the stem of the stiffener should be checked. It should be approximately equal to the leg size of the welds.
Fig. 29 - SUMMARY OF TEST RESULTS: BEAM LOAD VS. BEAM DEFLECTIONS
For convenience in testing, the beam moments have always been equal (for opposite pairs) and of opposite sense. The consequence is that the column web at the connection has not been subjected to substantial shearing stresses. As far as a study of column stiffening is concerned, to carry the thrusts and tensions from the beam flanges, the present tests are believed to be correctly conceived; however, when the opposing beams have moments of the same sense (wind moments), shears result in the column web which may require special stiffening such as a pair of diagonal stiffeners or a doubler plate. This subject has been partially treated by L.S. Beedle in his doctoral thesis (11) and it is expected that it will be further treated in the next program of tests.

Also reserved for future tests is the subject of the stiffening procedure when only one beam frames to the column web, with presumably a pair of beams framing to the column flanges. A similar situation to the above exists in a four-way connection if the beams framing to the column web are of unequal depth and in turn of different depth from the flange beams.
7. **SUMMARY**

a. Specimen AA, an unstiffened connection, in which the column (12WF65) had a web 0.0390" thick, should, according to the methods of analysis, have been stiffened; yet it passed the criteria for a satisfactory connection by developing the theoretical plastic beam moments (see Figure 10) while carrying an axial stress equal to the usual working stress found in practice (see THEORETICAL ANALYSIS for complete statement of the criteria). This indicates that the methods of analysis are conservative in some cases.

b. Specimen DD, with its split beam-type stiffeners (ST6WF32.5) on a column (12WF40), which obviously required stiffeners, proved to be more than adequately designed (see Figure 11) and the connection itself showed no signs of overstress. Butt welds failed, one beginning at practically peak load and progressing as the beam was deformed, the others failing just as the test was about to be discontinued, the prescribed fall-off of the beam loads having been reached. There was evidence of an accidental lateral moment as shown in Figure 21 by the higher tension stresses obtaining in one edge of the beam tension flanges than in the other edge.

c. Specimen BB, with 16WF36 beams framing onto the column flanges and 12WF27 beams framing to tee seats welded to the column web and to top plates which also served as stiffeners, tested very well, fully meeting the criteria as shown in Figure 15. The horizontal plate of the tee seat also served satisfactorily as a stiffener even though it was 4" above the compression flanges of the 16WF36 beams framing on the column flanges. More tests will be required to establish the effectiveness of stiffeners when placed eccentric to the beam flanges.
Acknowledgements

The entire program was carried out at the Fritz Engineering Laboratory of Lehigh University, of which Professor W.J. Eney is Director, with funds supplied by the American Institute of Steel Construction. The authors are indebted to members of the Research Committee on Welded Beam-Column Connections who gave invaluable advice, technical and otherwise, in the organization and execution of this project. This committee included Messrs. F.H. Dill, Chairman; L.S. Beedle; E.R. Estes, Jr.; T.R. Higgins; C.L. Kreidler and H.W. Lawson.

The authors also wish to acknowledge the assistance given them by members of the research staff of the Fritz Laboratory, particularly to Dr. Lynn S. Beedle, Chairman of the Structural Metals Division, to Mr. I.J. Taylor and Mr. R. Clark who were responsible for the instrumentation, to Mr. K. Harpel and his assistants who helped organize the actual tests, and to all those members responsible for the preparation of this manuscript.
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# APPENDIX

## Summary of Coupon Test Results

<table>
<thead>
<tr>
<th>Section</th>
<th>Mark</th>
<th>E (Ksi)</th>
<th>uy (Ksi)</th>
<th>yl (Ksi)</th>
<th>st y (Ksi)</th>
<th>ult (Ksi)</th>
<th>st (in./in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4&quot; Plate</td>
<td>233/P</td>
<td>30,900</td>
<td>40.65</td>
<td>39.87</td>
<td>62.00</td>
<td>61.54</td>
<td>0.0725</td>
</tr>
<tr>
<td></td>
<td></td>
<td>25,100</td>
<td>41.02</td>
<td>39.74</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12 WF 65</td>
<td>233/W1</td>
<td>30,400</td>
<td>42.57</td>
<td>41.81</td>
<td>67.74</td>
<td>67.74</td>
<td>0.015</td>
</tr>
<tr>
<td></td>
<td></td>
<td>29,700</td>
<td>38.37</td>
<td>38.54</td>
<td>67.74</td>
<td>67.74</td>
<td>0.00675</td>
</tr>
<tr>
<td></td>
<td>233/F1</td>
<td>30,100</td>
<td>40.63</td>
<td>40.07</td>
<td>65.57</td>
<td>65.57</td>
<td>0.01875</td>
</tr>
<tr>
<td></td>
<td>233/F1</td>
<td>30,600</td>
<td>44.28</td>
<td>40.46</td>
<td></td>
<td></td>
<td>0.200</td>
</tr>
<tr>
<td>12 WF 40</td>
<td>233/W2</td>
<td>31,200</td>
<td>47.17</td>
<td>44.16</td>
<td>39.85</td>
<td>68.93</td>
<td>0.021</td>
</tr>
<tr>
<td></td>
<td>233/W2</td>
<td>30,700</td>
<td>50.00</td>
<td>48.86</td>
<td>43.60</td>
<td>70.87</td>
<td></td>
</tr>
<tr>
<td></td>
<td>233/F2</td>
<td>31,300</td>
<td>43.47</td>
<td>41.77</td>
<td>37.86</td>
<td>68.00</td>
<td>0.0175</td>
</tr>
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<td></td>
<td>233/F2</td>
<td>29,400</td>
<td>42.90</td>
<td>41.51</td>
<td>37.67</td>
<td>68.15</td>
<td>0.01875</td>
</tr>
<tr>
<td>16 WF 36</td>
<td>233/W3</td>
<td>29,500</td>
<td>50.58</td>
<td>48.95</td>
<td>63.63</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>233/W3</td>
<td>30,600</td>
<td>47.00</td>
<td>45.66</td>
<td>61.61</td>
<td></td>
<td></td>
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<td></td>
<td>233/F3</td>
<td>30,400</td>
<td>41.86</td>
<td>40.25</td>
<td>61.18</td>
<td>61.18</td>
<td>0.0185</td>
</tr>
<tr>
<td></td>
<td>233/F3</td>
<td>30,200</td>
<td>40.58</td>
<td>38.98</td>
<td></td>
<td></td>
<td>59.99</td>
</tr>
<tr>
<td>12 WF 27</td>
<td>233/W4</td>
<td>31,200</td>
<td>43.70</td>
<td>43.70</td>
<td>38.81</td>
<td>61.62</td>
<td></td>
</tr>
<tr>
<td></td>
<td>233/W4</td>
<td>31,100</td>
<td>45.11</td>
<td>41.89</td>
<td>37.83</td>
<td>61.02</td>
<td></td>
</tr>
<tr>
<td></td>
<td>233/F4</td>
<td>31,100</td>
<td>40.36</td>
<td>38.65</td>
<td>34.74</td>
<td>61.24</td>
<td>0.0175</td>
</tr>
<tr>
<td></td>
<td>233/F4</td>
<td>29,800</td>
<td>39.36</td>
<td>38.17</td>
<td>33.79</td>
<td>60.03</td>
<td>0.02075</td>
</tr>
</tbody>
</table>

**Notes:**
- W = Web
- F = Flange
- st = Strain at strain hardening
- uy = Upper yield pt.
- yl = Yield level
- st = Static yield
- ult = Ultimate strength
APPENDIX

Calculations for Design of Specimen

Columns

Working stress on columns = 14.5 ksi
derived from AISC eqn. \( P = \frac{17000 - 0.485 (L)^2}{A} \)

where \( L = 72 \)

Structural Section Details

<table>
<thead>
<tr>
<th>Test</th>
<th>Column Size</th>
<th>Area in(^2) *</th>
<th>Measured Area</th>
<th>( P_w )</th>
<th>1.65( P_w )</th>
<th>2( P_w )</th>
</tr>
</thead>
<tbody>
<tr>
<td>AA</td>
<td>12WF65</td>
<td>19.11</td>
<td>19.00</td>
<td>276</td>
<td>455</td>
<td>552</td>
</tr>
<tr>
<td>BB</td>
<td>12WF40</td>
<td>11.77</td>
<td>11.70</td>
<td>170</td>
<td>280</td>
<td>380</td>
</tr>
<tr>
<td>DD</td>
<td>12WF40</td>
<td>11.77</td>
<td>11.49</td>
<td>167</td>
<td>276</td>
<td>376</td>
</tr>
</tbody>
</table>

* AISC handbook value

Analysis of Beams and Beam-Column flange welds

All dimensions of sections as measured on specimens
working = 20 ksi

Bending

\[
M_w = \frac{vS}{L} = \frac{v_vL}{L} = \frac{v^2}{L}, \quad \frac{v^2}{L}
\]

The calculations are similar to those in Progress Report No. 1. Lateral Buckling, local buckling, shear, deflections and beam rotations were investigated and calculations are similar to those found in the Appendix of Progress Report No. 1.
Analysis of Welds for Specimen BB

12WF27 Beams:

Using working load and allowable working stresses for the design of welds, seat, stiffener, etc. . . . . ; then check for ultimate load.

\[ V_w = 19 \text{ kips} \quad M_w = 19 \times 36 = 684 \text{ in-kips} \]
\[ T = C = \frac{684}{11.95} = 57.2 \text{ Kips} \]

Spec. section (26h) AISC:

\[ \frac{R}{t(n+k)} = 24 \text{ ksi} \]
\[ 19 = 24 \times 0.24 (N+0.813) \]

\[ N = \frac{19-4.68}{5.75} = 2.5 \text{ inches Required bearing length} \]

From Table 25 in the AISC text of Structural Shop Drafting Vol. 2, the choice is:

1/4" wide seat; 1/4" Fillet Welds; L = 7"

Plate Thickness 1/2"

Top Plate Weld Design:

Required Plate Thickness = \[ \frac{57.2}{20 \times 9.75} = 0.3" \]

At ultimate load the unit stress will be \[ \frac{125}{0.3 \times 9.75} = 42.7 \text{ kis} \]

Use 1/2" Plate

The length of weld available is \[ 9.75 + 2 \times 3.75 = 17.25" \]. Using butt welds on the plate

\[ \frac{125}{17.25} = 7.25 \text{ K/in.} \]

\[ \frac{7250}{1 \times 1/2} = 14500 \text{ psi}. \] This is okay since we are discussing ultimates

Weld connecting Top Plate to Beam Flanges:

The fillet welds are limited to 3/8" size

Working stress for 3/8" fillets is 3600 pounds/in.
Using the factor of safety of 3, we use for design $3 \times 3600 = 10800$ pounds/in

$\frac{125}{10} = 12.5$ inches required length of weld.

Length of weld available 6" overhead fillets

6-1/2" Fillet on top of flange

Check on Tee seat:

From Grover's "Manual of Design for Arc Welded Steel Structures", page 123

$$ R = \frac{23.04 \cdot D \cdot L^2}{\sqrt{L^2 + 16e^2}} $$

where $D = \frac{3}{16}$ \hspace{1cm} $L = 8''$ \hspace{1cm} $e = 3.2''$

$$ R = \frac{23.04 \cdot \frac{3}{16} \cdot 6^2}{\sqrt{6^2 + 16(3.2)^2}} = 18.3 \text{ kips} $$

Predicted ultimate $R = 3 \times 18.3 = 54.9 \text{ K}$

This is nicely in excess of 41.4, the predicted ultimate load.
Material Dimensions & Properties

In Figure 29 the average values of all the dimensions of the WF sections used in the tests is shown. The calculations of the section properties are similar to that presented in the Appendix of Progress Report No. 1. In the Table below the different section properties are shown:

<table>
<thead>
<tr>
<th>Test</th>
<th>Beam Size</th>
<th>Area</th>
<th>Section Modulus</th>
<th>Plastic Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>AA</td>
<td>16WF36</td>
<td>10.28</td>
<td>55.59</td>
<td>62.73</td>
</tr>
<tr>
<td>BB</td>
<td>16WF36</td>
<td>10.29</td>
<td>54.20</td>
<td>61.52</td>
</tr>
<tr>
<td></td>
<td>12WF27</td>
<td>7.83</td>
<td>32.60</td>
<td>36.56</td>
</tr>
<tr>
<td>DD</td>
<td>16WF36</td>
<td>10.24</td>
<td>54.06</td>
<td>61.37</td>
</tr>
</tbody>
</table>
WF SECTIONS - Average values

**Fig. 31 - WF SECTIONS, ACTUAL DIMENSIONS**