PROPOSAL FOR TESTS OF BEAM-TO-COLUMN CONNECTIONS SUBJECTTED TO MOMENT, SHEAR, AND HIGH AXIAL LOAD

By
John W. Peters
George C. Driscoll, Jr.

Fritz Engineering Laboratory Report No. 333.1
Beam-to-Column Connections

PROPOSAL FOR TESTS OF
BEAM-TO-COLUMN CONNECTIONS
SUBJECTED TO MOMENT, SHEAR,
AND HIGH AXIAL LOADS

by

John W. Peters
George C. Driscoll, Jr.

This work has been carried out as
part of an investigation sponsored jointly
by the American Iron and Steel Institute
and the Welding Research Council.

Fritz Engineering Laboratory
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania

September 1967

Fritz Engineering Laboratory Report No. 333.1
# TABLE OF CONTENTS

1. INTRODUCTION 1
   1.1 Past Work 2
   1.2 Data Presently Available 3
   1.3 Preliminary Investigation 4

2. PROPOSED TEST PROGRAM 10
   2.1 Connection Subassemblage 10
   2.2 Test Setup 12
   2.3 Instrumentation 15
   2.4 Testing Procedure 17

3. FINANCES 17

4. PRELIMINARY PLANNING FOR PHASE II 18

5. SUMMARY 20

6. NOMENCLATURE 21

7. TABLES AND FIGURES 22

8. REFERENCES 34
1. INTRODUCTION

In a multi-story frame, methods have been found to predict the behavior of the frame and most of its components.\(^1\) However, the plastic method of analysis and design presently used neglects the combined effect of shear and axial load on the behavior of beam-to-column connections.\(^2\)

During recent tests on multi-story frames at Lehigh University it was observed that high column axial load and shear resulting from beam moments significantly affect the behavior of beam-to-column connections.\(^3\) In some instances the diagonal stiffener, which was used in an exterior connection to resist the shear caused by a large beam moment, actually yielded before the plastic moment was reached in the beam. This behavior was observed in the lower stories where axial load is higher. In one test in which diagonal stiffeners were not used for an exterior connection, the shear deformations was largest in the connection with the highest axial load even though the shear force was the same in all the connections.

As a result of the observations from the frame tests and because of other unanswered questions about beam-to-column connections, a project on Beam-To-Column Connections was initiated at Lehigh University in 1966 by the American Iron and Steel Institute and the Welding Research Council. This proposal will
describe the first phase of experimental and theoretical work on
the new project, which will consist of tests on small joints
salvaged from previous three-story frame tests.

1.1 Past Work

Previous work done in the area of beam-to-column connections
has not considered the combined effect of high axial load and shear
resulting from beam moments. Solutions were developed by Beedle,
et al. for the problem of shear failure of a corner connection
neglecting the effect of axial load.\(^{(4)}\) The additional problem of
local instability or localized failure of a beam-to-column
connection was studied by Jensen, et al.\(^{(5)}\) Solutions which are
presently available provide a satisfactory prediction of beam-to-
column connection behavior with low axial loads in the column.

There is very little literature available which deals with the
combined shear-axial load-moment interaction for connections.
However, in Reference 6 a group from the University of Tokyo has
examined in great detail the elastic solution of the combined
shear-axial load-moment interaction for beam-to-column connections.
The same reference does give some information on the ultimate
strength of a connection subassemblage.

The test series proposed here makes use of many of the testing
procedures described in Reference 5 and many of the findings of
Reference 6.
1.2 Data Presently Available

The first tests proposed for the new project, Welded Beam-to-Column Connections, will be made on connections salvaged from previous multi-story frame tests. Certain connections in the multi-story frame tests sustained loads significantly below the inelastic range of the material because of their particular location within the test frame. Seven of these connections have been salvaged by burning through the beams and columns on the test frame a short distance from the joint. The result is a small connection subassemblage which is ready to be tested as a connection.

Data from the multi-story frame tests has been preserved and will be presented for comparison with results from the proposed investigation. Seventeen of the beam-to-column connections in the frame were gaged. Rotations of the beams and columns were measured for each loading increment.

The configuration of the connections gaged in the multi-story frame test was varied. Most of the connections had diagonal stiffening which acted in tension, but some had their diagonal stiffening acting in compression. (See Figure 1). There were some additional connections from interior regions of the frame tested which were unstiffened. It is hoped that data from the test frames described above will supply experimental information on light column connections with relatively low axial load and high beam moment. In the future, tests will be designed to examine the connection
member size effect on beam-to-column connection behavior, with particular emphasis on those connections in which there is a disparity between the thicknesses of connecting elements.

1.3 Preliminary Investigation

In order to help formulate a test program and provide advance estimates of test behavior, a preliminary theoretical investigation was attempted. The preliminary investigation has taken the same type approach as that used to determine the interaction of thrust, shear, and moment in beams. This type of a solution is a lower bound approach. Using a lower bound approach it is necessary to begin the analysis with a description of the force distribution applied to the connection. This is a complex problem for a solution in the elastic or plastic range. The problem is further complicated by the residual stress pattern due to welding in the fabrication process. As a result, the authors have attempted to formulate an equilibrium solution for the strength of the connection which will reflect the effect of normal force in the column on the shear capacity of a connection web panel.

The following is a list of the assumptions made in developing the lower bound solution described above:

1. All shear in the column is distributed uniformly across its web.

2. All bending moment in the beam is taken by the beam flanges.
3. Axial load in the beam is neglected.

4. All normal force in the beam flange is transmitted directly to the horizontal stiffener.

5. No specific steps have been taken to account for residual stresses.

6. A connection will fail when its cross section becomes fully yielded.

7. The yield surface of the connection is defined by the von Mises Yield Theory.

8. Strain-hardening is not taken into account.

9. No account is made for any discrepancies in the directions of shears along the edge of a web panel at the corners of a connection. (It is believed that this problem can be corrected once a better knowledge is gained of the shear distribution in the plastic range).

10. The connection is considered to be made of an elastic, perfectly plastic material.

The following relationships are those developed in the preliminary theoretical analysis to predict beam-to-column behavior with the neutral axis of the column in the web and in the flange respectively. The equations presented are shortened using the terms defined by equations (1.4) through (1.7). Complete nomenclature is listed in Section 6 of this report.
The lower bound solution developed relates the reduced plastic moment in the column to the size of the members in the connection, to the frame dimensions, to the beam load, and to the axial load in the column of the connection. Equation (1.1) is the relationship developed relating reduced column moment, $M_{pm}$, to thrust, $T$, when the neutral axis of the column is in the web. The relationship has been non-dimensionalized by dividing by the plastic moment of the column, $M_p$, and the yield thrust of the column, $T_y$.

$$\frac{M_{pm}}{M_p} = 1 - \frac{A_w d_w}{4Z} \left[ 1 - \sqrt{1 - \frac{0.75}{w^2 L^2} \left[ \frac{A_f - B A_w}{w L} \right]^2} \right] - \frac{A^2 \left( \frac{T}{T_y} \right)^2}{4Z w \left( 1 - \frac{0.75}{w^2 L^2} \left[ \frac{A_f - B A_w}{w L} \right]^2 \right)}$$

Equation (1.2) is the non-dimensionalized relationship developed relating reduced column moment to column thrust when the neutral axis of the column is in its flange. This equation is influenced by the amount of yielding in the column flange, $\eta_o$. Yielding in the column flange is affected by the amount of yielding in the beam flange. The results presented in this paper assume the beam flange is fully yielded.

$$\frac{M_{pm}}{M_p} = \frac{2A_f}{A_w} \left[ \frac{d}{d_w} - \eta_o - \frac{t}{d_w} \eta_o \right] \frac{1 + 2 \frac{A_f}{A_w} \left[ \frac{d}{d_w} \right]}{\left( \frac{d}{d_w} \right)}$$
Equation (1.3) expressed column flange yielding as a function of connection and frame geometry and sizes. Axial load is introduced into the equation by parameter $\alpha$.

$$
\eta = 1 - \frac{A_w}{A_f} \theta - \frac{2\sqrt{3}}{3} \frac{w}{\theta} \sqrt{\beta - (\alpha - \beta)^2 (1 - \frac{4}{3} \lambda^2) + \frac{4}{3} \lambda (\alpha - \beta) \sqrt{\beta - (\alpha - \beta)^2}}
$$

(1.3)

The following relationships have been used to shorten equations (1.1) through (1.3). Both $\lambda$ and $\beta$ are functions of the connection member sizes. The relative member sizes and the frame geometry are reflected in $\beta$. Axial load and column size are reflected by $\alpha$.

$$
\alpha = \frac{A_f}{A_w} - \left[ 1 + \frac{A_f}{A_w} \right] \frac{T}{T_y}
$$

(1.4)

$$
\beta = \frac{d_w A_f}{2 d_w L A_w}
$$

(1.5)

$$
\lambda = \frac{L}{d_w}
$$

(1.6)

$$
\theta = 1 + \frac{4}{3} \lambda^2
$$

(1.7)

Figure 2 shows a typical interaction curve relating reduced column moment to thrust for a given beam shear, member size, and frame geometry.

Data from tests described in Section 1.2 is also presented in Figure 2.
Due to violations of equilibrium conditions and plasticity conditions in some of the assumptions made at the beginning of the preliminary analysis outlined above, it is expected that possibly there may be a better lower bound solution. Work is presently underway for applying the information gained from two pilot tests to the problem of finding a better lower bound solution. An overall yielding pattern over a large portion of the connection web panel is the objective of the lower bound solution presently underway.

A series of upper bound solutions is also presently being attempted which will incorporate overall connection failure with the combined shear, moment, axial load interaction. An upper bound solution is being undertaken because the pilot tests conducted have indicated that the lower bound solution developed in the preliminary theoretical investigation is highly conservative. The result of this study should be several upper bound solutions which each are applicable depending upon the relative connection member sizes, frame dimensions, and relative load magnitudes. As a result of the preliminary lower bound theoretical investigation and the pilot study on connections with unstiffened web panels, it is expected that an upper bound solution may supply a more realistic answer to a complete connection failure than is now achieved in the lower bound solution. For an upper bound solution, it is necessary to assume a failure mechanism. Therefore, by observing how the test connections in the proposed test series fail it should be
possible to develop a series of upper bound solutions which closely correspond to actual beam-to-column connection behavior.

Since very little is known about the shear, moment, axial load interaction in beam-to-column connections a test series is proposed in this paper in an effort to gain a better understanding of beam-to-column connection behavior. The test series will provide information which will permit a more logical upper and lower bound solution to be formulated and checked.
2. PROPOSED TEST PROGRAM

The remainder of this paper is the presentation of a proposal for a preliminary study of welded beam-to-column connections. The final objective of the proposed test series is a better understanding of the shear, axial-load, moment interaction in a beam-to-column connection which is loaded beyond first yield.

2.1 Connection Subassemblage

The connection subassemblages proposed to be tested in this series are exterior connections of a frame as is shown in Figure 3. The test specimens were all cut from the frames of Fritz Laboratory Project 273, Plastic Design of Multi-Story Frames. Seven connection subassemblages have been saved. Figure 3 shows the proposed connection subassemblage test setup. The columns are 8 feet 4 inches in height (base to base). The beams are about 2 feet 6 inches long and are loaded vertically 2 feet from the inside column face. This simulates a beam-to-column connection for a multi-story frame in the inverted position. The subassemblage dimensions simulate the dimensions of a frame with columns spaced every 12 feet and a story height of 8 feet 4 inches.

The dimensions of the subassemblage were chosen in order to prevent instability in any of its members. Columns are made from either a 6WF20 \( (L/r_y = 66.6) \) or a 6WF25 \( (L/r_y = 65.8) \) depending upon the portion of the original frame from which they were taken.
All beams are made from 12B16.5 (L/r_y = 32) sections. ASTM-A36 steel is used for both beams and columns.

The subassemblies which are proposed to be retested have two basic types of stiffening. Some connections have no web stiffening and some have diagonal stiffening. Specimens with diagonal stiffening can be tested in such a manner that the web stiffening is subjected to either tension or compression. It is proposed that of the five connections with diagonal stiffening, two be tested with the stiffener acting in compression and two with the stiffener acting in tension. The fifth specimen would then be tested with the stiffener acting in either tension or compression in order to supplement any of the previous test data.

The unstiffened test specimens would be used to determine the force description in a web panel of a connection as yielding progresses throughout the section. These unstiffened test specimens would also supply valuable test data which could be used to determine if the present plastic design connection criterion can be applied not only to connections of low axial load values but to those with relatively high values of thrust in the column.

The connections would be loaded in such a manner that the present plastic design method would predict that a shear failure would not occur in the connection web panel. However, theory indicates that the combined effect of high axial load and high shear will reduce the carrying capacity of such a connection under high axial load.
The general purpose of the proposed test series is three fold:

1. To help determine the stress distribution on a connection web panel loaded into the plastic range.

2. To determine the effect of various types of web stiffening on both the strength and stress distribution of a connection.

3. To determine if the method presently used in plastic design for designing beam-to-column connections can be applied to cases of high axial load and beam moment.

2.2 Test Setup

The proposed test setup is shown in Fig. 3. Column loads will be applied by an 800 kip screw-type universal testing machine with a poise-and-lever-type weighing system.

Two types of end connections for the columns have been considered. Both pinned and fixed-end columns have been studied. The most desirable end condition would be pinned, because this simulates the actual behavior of a column in a real frame. However, due to the high axial load and end shear in the column a very large pin is required. The cost of a machined end fixture which would supply the required pin action was considered to be excessive for the pilot tests.
Fixing the column end against rotation can be accomplished by a much simpler test setup. End fixity is developed by bolting the column base plates directly to the load applicator. A setup of this type no longer simulates usual column behavior in a frame, because the shear force resulting from the column bending causes too great a reduction in the shear entering the connection from the beam flange. However, for tests whose principal goal is to determine the behavior of a connection under a given set of force boundary conditions, a fixed end condition for the columns would be satisfactory.

Two pilot tests have been conducted on unstiffened connection subassemblages which have very short columns due to the previously described salvage operations. The specimens used for tests on these connections had full end-fixity (see figure 4). The purposes of the pilot tests were:

1. Check the proposed testing procedure.

2. Check the proposed instrumentation.

3. Obtain a better description of the actual force distribution in an unstiffened beam-to-column connection loaded to failure.

4. To make use of very short specimens inexpensively and still obtain valuable test information to supplement later test data.
The beam load versus beam deflection curve for pilot test 333.A2 is shown in Figure 5. This figure shows how much strength the connection region of a frame has beyond first yield.

The remainder of the specimens to be tested which were salvaged from the previously tested frames have longer columns and are stiffened. It is proposed to test them in a manner which simulates frame action. This would be accomplished by simulating pin action at the top and bottom of the specimen column. The pins would correspond to the inflection points in the columns above and below a floor level of an actual structure. In order to obtain an inexpensive pin-end condition use will be made of the 2,000 kip capacity column pin-end fixtures available at Fritz Laboratory.(9)

Columns of the subassemblage will be welded to reusable base plates. The base plates will be bolted to the fixture plate of the 2,000 kip capacity column end fixtures. A slight modification has been made to the column end fixture described by Huber in Reference 9. A shear plate has been designed which can be bolted to the fixed base of the fixture. The shear plate will provide significant horizontal restraint to the column and still allow pin-end rotation. The shear plate has a hole in it large enough to pass a 1 inch diameter bolt through it. The bolt will be bolted to the rotating pin, sandwiching the shear plate between the pin and the bolt head. If two shear plates are used on each end fixture (see figure 6), the fixtures can be converted from a roller resisting only vertical motion to a pin resisting vertical and horizontal motion, but still allowing rotation.
Beam loads will be manually applied to a point on the beam 2 feet from the column face, using a 35 ton mechanical jack. The beam loads will be measured using a calibrated aluminum dynamometer placed under the jack.

2.3 Instrumentation

Each connection will be instrumented with electrical strain gages at selected locations on the column flanges, stiffeners, and web panel. The gages on the web panel will be rosettes. These gages will indicate quantitatively the stress distribution around the connection in the elastic range. A preliminary study on a gaged beam-to-column connection has indicated that it is possible to get strain readings from relatively inexpensive gages well into the plastic range. See Figure 7 for an indication of the behavior of a SR-4 gage mounted at the exact center of a beam-to-column connection. Despite the fact that gage readings are obtainable in the plastic range, it is doubtful that any more than a qualitative value should be placed on the high strains due to the localized effect of a yield line passing through a gage or the change in gage resistance at such a high strain. However, using such a plot as shown, in Figure 7 and knowing the yield strain of the material in question it is certainly possible to obtain a very good idea of the manner in which a connection progresses from the elastic range toward failure.
The beam and columns of each test will be instrumented with electrical strain gages in such a manner that moments and axial loads may be calculated for each load increment.

Rotation gages will be attached to all connections giving the relative rotation of the top and bottom columns with respect to each other and the relative rotation of the beam with respect to the column. Figure 8 shows in detail how the rotation gage is to be mounted on the web of the connection. The gage will be clamped to rods which are spot welded to the member web a short distance from the joint. The rotation gage is made of a series of clamped rods whose relative movements are measured by dial gages. Figure 9 shows a mounted rotation gage on one of the pilot tests.

Rotations of the column ends will be measured using 20 inch level bars. This will give an absolute value for end rotation. The pedestal will also be gaged in order to determine its absolute rotation.

Beam deflection will be measured using a mechanical dial gage which will be fixed to the base of the testing machine at one end and the beam on the other end. This measurement will serve as a criterion measurement by which it is possible to determine if the connection is in an equilibrium state in the plastic range. This measurement was chosen to be the criterion measurement because it is the most direct deformation reading obtainable and reflects the entire behavior of the subassemblage.
2.4 Testing Procedure

Figure 3 shows a subassemblage ready to test. Testing will begin by aligning the subassemblage column until strain gages on the top and bottom column located at the four corners of the column read within 5% of each other at a given cross section. When the column is aligned a zero datum set of readings will be taken.

The actual subassemblage test will begin by incrementally building up the column axial load to the pre-determined axial load for the test. Beam load will then be applied incrementally using a mechanical jack and calibrated dynamometer. After an incremental beam load has been applied, the column load will be finely adjusted back to the desired axial load for the test. The beam deflection will now be observed and when it becomes steady all gages will be read. This procedure will be repeated for each beam load increment.

Each subassemblage will be tested in the same manner. The only variable (with the exception of stiffening) will be the magnitude of the column working load. Table 1 gives a summary of the loads for the proposed test series.

3. FINANCES

Tests will be conducted using the regularly contracted funds furnished for the Beam-to-Column Connections project by American Iron and Steel Institute through the Welding Research Council.
4. PRELIMINARY PLANNING FOR PHASE II

Future tests should concern themselves with the effect of relative member size on the behavior of beam-to-column connections. The tests proposed in this paper are on members that are small and all of about the same size. Once a workable relationship has been developed which will accurately predict the behavior of beam-to-column connections, it will be desirable to expand the investigation to larger beams and columns. Connections with columns made from 14-inch wide-flange sections and beams of 24- or 36-inch sections should be investigated.

The question of the practical necessity of doubler plates or diagonal stiffening, will be examined. Comments from practicing engineers point out how difficult it is to frame into a structure with diagonal stiffening. Therefore, tests could be made to determine how critical the condition is when there is no diagonal stiffening present.

It may be well to re-evaluate the seriousness of shear failure of a connection web panel on the behavior of a total structure. The tests and studies which have previously shown the alarming amount of deformation which results from shear failure of a connection were done on connections in determinate assemblages. The interpretation of previous tests did not reflect the idea that the "failed" connection actually rotated considerably after initial shear yielding as well as finally reaching a moment near in
magnitude to the plastic moment of the member used for the connection. The increase in moment after shear yielding of the connection panel was undoubtedly due to strain hardening. Strain-hardening is always present in regions of high shear stress and therefore it seems reasonable to try to better understand it and use its benefits. Figure 5 shows how much deflection can be expected beyond first shear yield. Figure 10 is a photograph showing the extreme deformation and yielding which can occur in a connection and still provide load-carrying capabilities.

Tests on larger subassemblages will be designed in order to determine the effect of a "failed" connection on the rest of the subassemblage. From these tests, possibly a method could be developed to use the "under strength" of the connection as a reduced plastic hinge moment rather than the nominal $M_p$ of the beam or column. The method would then take into account this plastic hinge behavior on the rest of the structure. If this method requires a small increase in member sizes, but saves the fabrication cost of details and permits easy framing of perpendicular floor members into the column web, designers might be able to provide substantial savings in material. These savings could be achieved under the condition that this new form of plastic hinge would give the required rotation capacity to permit the necessary redistribution of moment in the structure.
Seven tests on previously fabricated beam-to-column connections are proposed. The connections will be subjected to the combined effect of high axial and shear loads as is shown in Figure 3.

The seven tests will provide an insight into the stress-distribution and ultimate carrying capacity of various types of beam-to-column connections subjected to high thrust and shear loading.

It is expected that the results of these tests will be of value in the further refinement of an interaction relationship between shear and axial load for beam-to-column connections which may be used eventually for a better structural analysis and design.
6. NOMENCLATURE

A = Area of column

$A_f$ = Area of column flanges (2)

$A_w$ = Area of column web

a = Area of beam

$a_f$ = Area of beam flanges (2)

$a_w$ = Area of beam web

d = Depth of column from $Q_L$ Flange to $Q_R$ Flange

$d_B$ = Depth of beam

$d_{Bw}$ = Depth of beam web

$d_c$ = Depth of column

$d_w$ = Depth of column web

L = Length of beam to point of zero moment

l = Length of column to point of zero moment

$\ell$ = Thickness of column flange

w = Thickness of column web

$y_o$ = Distance from column web $Q_L$ to yield interface

Z = Plastic section modulus

$m_o \neq$ Percent of beam flange not yielded by moment

$\eta_o \neq$ Percent of column flange not yielded by moment

$\xi$ = Percent of full yield load in column web which is given by axial load

$M_{pm}$ = Plastic moment due to shear, bending and thrust

T = Thrust in column

$\tau$ = Shear in column web
7. TABLES AND FIGURES
TABLE 1. CONNECTIONS TO BE TESTED

<table>
<thead>
<tr>
<th>Test</th>
<th>Col. Size</th>
<th>Stiffening</th>
<th>L/(r_y)</th>
<th>P/(P_y)</th>
</tr>
</thead>
<tbody>
<tr>
<td>333.A1</td>
<td>6WF25</td>
<td>None</td>
<td>40*</td>
<td>0.60</td>
</tr>
<tr>
<td>333.A2</td>
<td>6WF25</td>
<td>HC</td>
<td>40*</td>
<td>0.80</td>
</tr>
<tr>
<td>333.A3</td>
<td>6WF20</td>
<td>HCT</td>
<td>67</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>333.A4</td>
<td>6WF20</td>
<td>HCT</td>
<td>67</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>333.A5</td>
<td>6WF20</td>
<td>HCT</td>
<td>67</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DC</td>
<td></td>
<td></td>
</tr>
<tr>
<td>333.A6</td>
<td>6WF20</td>
<td>HCT</td>
<td>67</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DC</td>
<td></td>
<td></td>
</tr>
<tr>
<td>333.A7</td>
<td>6WF25</td>
<td>HCT</td>
<td>66</td>
<td>**</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DCorDT</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

HC = Horizontal stiffening in compression
HCT = Horizontal stiffening in both compression and tension
DT = Diagonal stiffening in tension
DC = Diagonal stiffening in compression
* Due to salvage operations these columns are 5 feet in length
**To be determined after other tests are conducted
Fig. 1 Types of Beam-to-Column Connections
Fig. 2 Shear-Moment-Axial Load Interaction Curve Developed From the Preliminary Lower Bound Investigation. Data from previous frame tests is also shown.
Fig. 3 Proposed Test Setup
Fig. 4 Pilot Study Test Setup
Fig. 5 Load-Deflection Curve for Pilot Test 333.A2
Fig. 6 View of End Fixtures
Fig. 7 Strain - Beam Load Curve for Compression Diagonal at Center of Connection Web Panel for Pilot Test 333.1
Fig. 8 Web Panel Rotation Gage
Fig. 9 View of Web Panel Rotation Gage and Beam Deflection Gage on a Pilot Test
Fig. 10 View of Failed Pilot Test Subassemblage
8. REFERENCES

1. Driscoll, G. C.; Beedle, L. S.; Galambos, T. V.;
   Lu, L. W.; Fisher, J. W.; Ostapenko, A. and Daniels, J.H.
   LECTURE NOTES--"PLASTIC DESIGN OF MULTI-STORY
   FRAMES", Fritz Laboratory Report No. 273.20, Lehigh
   University, 1965

2. Fisher, J. W.
   WELDED CONNECTIONS, Structural Steel Design, Chapter
   19, ed. L. Tall, Ronald Press, New York, 1964

3. Yura, J. A.
   THE STRENGTH OF BRACED MULTI-STORY FRAMES, Fritz
   Laboratory Report No. 273.28, Lehigh University, September 1965

4. Beedle, L. S.; Topractsoglou, A. A. and Johnston, B. G.
   CONNECTIONS FOR WELDED CONTINUOUS PORTAL FRAMES,
   Progress Report No. 4, Welding Journal Research
   Supplement, July and August, 1951, November 1952

5. Graham, J. D.; Sherbourne, A. N.; Khabbaz, R. N. and
   Jensen, C. D.
   WELDED INTERIOR BEAM COLUMN CONNECTIONS, WRC
   Bulletin Series No. 63, 1960

   RESEARCH ON THE BEHAVIOR OF STEEL BEAM-TO-COLUMN
   CONNECTIONS, Laboratory for Steel Structure,
   University of Tokyo, 1966

7. Horne, M. R.
   THE FULL PLASTIC MOMENTS OF SECTIONS SUBJECTED TO
   SHEAR FORCE AND AXIAL LOAD, British Welding Research
   Association Report No. FE.1/51/57, March 1967

8. Kusuda, T. and Thurlimann, B.
   STRENGTH OF WIDE FLANGE BEAMS UNDER COMBINED
   INFLUENCE OF MOMENT, AND AXIAL FORCE, PROGRESS
   REPORT NO. 27, Fritz Laboratory Report No. 248.1,
   May 1958

9. Huber, A. W.
   FIXTURES FOR TESTING PIN-ENDED COLUMNS, Fritz
   Laboratory Report No. 220A.24, Lehigh University,
   July 1956