Test program of steel beam-to-column connections, July 1971.

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Beam-to-Column Connections

TEST PROGRAM OF STEEL BEAM-TO-COLUMN CONNECTIONS

by

Joseph S. Huang
Wai-Fah Chen
John E. Regec

Fritz Engineering Laboratory Report No. 333.15
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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.

Department of Civil Engineering

Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania

July 1971

Fritz Engineering Laboratory Report No. 333.15
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1. INTRODUCTION

One of the most influential elements in the cost of steel multi-story building frames is the moment-resisting beam-to-column connection. Should the engineer use a bolted joint, a welded joint, or a combination of both? Selection of connections is often based upon the consideration that some particular connection assembly used is more economical and better than any other fabrication. In general, the cooperation of both the consulting engineers and the fabricators is essential in order to attain the most efficient and economical design for beam-to-column connections.

The various types of connections which have been used extensively in steel building frames and are of particular interest to designers are summarized in Ref. 1. It is an interim report prepared to indicate the basic problems pertaining to connection study and to suggest some possible areas of needed research work. Reference 2 contains detailed evaluation of different types of connections from the viewpoint of a structural engineer.

1.1 PREVIOUS RESEARCH

Previous investigations on moment-resisting beam-to-column connections conducted at Cambridge University, Cornell University, and Lehigh University are summarized and discussed in Ref. 3. The types of connections studied are: fully welded connections, welded top plate and angle seat connections, bolted top plate and angle seat connections, end plate connections, and T-stub connections. In addition, the behavior of
welded corner connections, bolted lap splices in beams, and end plate type beam splices was discussed. The connecting media for these specimens were welding, riveting, and bolting. Only A325 high-strength bolts were used. The most important result of these tests is that for all properly designed and detailed welded and bolted moment connections the plastic moment of the adjoining member was reached, and the connections were able to develop large plastic rotation capacity. There were no premature failures except those which could have been predicted and prevented (3).

A series of eight tests of full-size steel beam-to-column connections was carried out at the University of California (4). The connections were subjected to cyclic loading simulating earthquake effects on a building frame. Among those connections tested were two fully welded connections, five flange-welded web-bolted connections, and one flange-welded connection. A325 bolts were used in fastening the web shear plates. Beam sections used were W18x50 and W24x76. Column sections were W12x106. The connection specimens were made of ASTM A36 steel. All connections had horizontal stiffeners which were connected to the columns by groove welds. Results of this series of tests show that the hysteresis loops in all cases were stable in shape under repeated loading cycles. The failure of connections was due to either local buckling of beam flanges or weld fracture, and occurred only after many cycles of loading beyond yield.

1.2 STATEMENT OF THE PROBLEM

Currently, a research program on beam-to-column connections is under way at Lehigh University under the guidance of the Welding Research Council. One of the objectives of the investigation is to study those types of beam-to-column connections which have been suggested to be of higher
priority and require immediate attention.

In this test program three types of beam-to-column connections are considered: flange-welded web-bolted connection (Problem 1.6 of Ref.1), flange-welded connection (Problem 1.7) and bolted top and bottom moment plate connection (Problem 1.9B). One of the topics included in this study is the weld requirements for horizontal stiffeners (Problem 1.5B). This is to verify the use of fillet welds for stiffeners in lieu of groove welds.

The current Lehigh program differs from the University of California program (4) in the following points:

1) The connections will be under monotonic loading and loaded up to failure. There is no strain reversal at any location of the beam.

2) Horizontal stiffeners are not used for eight tests. For the four tests requiring horizontal stiffeners, fillet welds are used to connect stiffeners to column flanges.

3) Higher allowable shear values, 30ksi for A325 and 40ksi for A490, are used for bolts in bearing-type connections.

4) Slotted holes are used for bolts in web shear plates designed as bearing-type connections. Oversize holes are used for bolts in moment plates designed as friction-type connections. The use of slotted holes and oversize holes are desirable for erection adjustments.

5) A325 bolts are used only in the shear plates of two tests. The remaining fasteners are A490 bolts.

6) The specimens are made of ASTM A572 Grade 55 steel.
2. DEVELOPMENT OF TEST PROGRAM

2.1 OBJECTIVE OF RESEARCH

The objective of this study is to investigate the performance of those suggested beam-to-column connections that are under symmetrical loading condition. Primary attention is focused upon the moment capacity, rotation capacity, and overall stiffness of those connections (5).

Figure 1 shows the behavior of a beam-to-column connection under symmetrical loads. If a connection is properly proportioned and the weld and bolts are properly designed, it will be able to carry the plastic moment of the beam and allow the beam to rotate inelastically through a very large angle as indicated by Curve A. Curve A also shows that in the elastic range (under working load) the connection exhibits overall stiffness for maintaining the location of beams and columns relative to each other. Curves B, C, and D show that the performance of connection is unsatisfactory, obviously, because some detail has been underdesigned.

In this study, the connections are proportioned to meet the design requirements as illustrated by Curve A in Fig. 1. The results should provide the basis for further modifications in design procedures of safe, efficient, and economical beam-to-column connections.

2.2 THEORETICAL ANALYSIS

Usually the behavior of a beam-to-column connection can be judged
by its moment-rotation relationship. Figure 2 shows the intermediate steps which facilitate the explanation of the method of analysis. The axial load in the column, Fig. 2(a), will introduce a bending moment diagram as depicted in Fig. 2(b). Research on the behavior of beams under moment gradient indicates that, due to the presence of shear force which accelerates the strain-hardening in the yielded parts of the beam, the beam moment will increase beyond the plastic moment value. The moment diagram in the column web panel zone is shown by the dashed line which gives an upper bound estimation.

Utilizing the stress-strain curve from Fig. 2(c), the curvature along the beam can be obtained by the theory of strength of materials taking into consideration the property of member cross-section. The curvature diagram is shown in Fig. 2(d). Finally, the moment-rotation curve can be readily obtained by performing integration of the curvature diagram.

Two prediction curves are shown in Fig. 2(e). Curve A is predicted by using simplified plastic theory and Curve B, by considering strain-hardening effect. In plastic analysis it is assumed that the connection is able to sustain a large amount of rotation near maximum load. It is the primary goal of this test program to examine the design rules of connection details which will assure the adequate performance of beam-to-column connections.

2.3 **DESIGN OF SPECIMENS**

The specimens are designed according to Section 2.8, Connections, of the AISC Specification\(^{(6)}\). The connections are proportioned to resist
the moment and shear generated by the full factored load. Since the loading condition resembles gravity type loading (dead load plus live load), the load factor used is 1.7. The stresses used in proportioning welds, shear plates, and top and bottom moment plates are then equal to 1.7 times those given in Section 1.5 of the AISC Specification\(^6\). For A325 and A490 high-strength bolts in bearing-type connections the design shear stresses used are equal to 1.7 times 30 ksi and 40 ksi, respectively, instead of 22 ksi and 32 ksi suggested in current Specification. The concept for this procedure will be discussed later.

1. Member Size and Beam Span

The connection specimens are chosen to have an appropriate combination of a beam section and a column section which represents the real interior beam-to-column connections in a multi-story frame. Another factor considered in selecting beam size is the way a wide-flange shape resists bending moment and shear force. It is well known that the flanges resist most of the bending moment, and the web almost entirely carries the shear force. The ratio of flange area to web area furnishes an index to the amount of moment carried by the web, which must be transferred to the flanges at the connections (since the shear connections have negligible moment-resisting capacity).

The choice of column size is dictated by the need for horizontal stiffeners. When horizontal stiffeners are not required, the columns selected are the least column size based upon the tension or compression flange criterion.

The specimens are proportioned such that the section at beam-column juncture can resist plastic moment concurrently with the factored shear capacity of single shear bolts in a single line. Beam span, then, is simply the ratio of moment to shear values.
Three different sizes of specimens consisting of W14x74, W24x61, and W27x94 beams connected to W10x60, W14x136, and W14x176 columns, respectively, are used in this test program. The ratios of one flange area to web area, \(A_f/A_w\), for W14x74, W24x61, and W27x94 sections are 1.39, 0.44, and 0.60, respectively. The behavior of these sections should be representative of a wide range of wide-flange sections.

The material of the specimens is ASTM A572 Grade 55 steel.

2. Fasteners and Holes

ASTM A325 and A490 bolts are used to assemble the joint. In bearing-type connections, the allowable shear stresses used in design for A325 and A490 bolts are 30 ksi and 40 ksi, respectively. These values reflect the logical design criterion which would result if an adequate factor of safety was applied against the shear strength of the fasteners. This design criterion is based upon the results of study of A7 and A440 steel lap and butt joints fastened with A325 bolts, and A440 steel joints connected with A490 bolts\(^{7}\). Tests have been subsequently carried out to substantiate the suggested design criterion, especially the use of A490 bolts in A440 and A514 steel joints\(^{8, 9}\).

Since both the oversize holes and slotted holes are desirable to facilitate erection adjustments, and slotted holes may better facilitate the assumed distribution of shear and moment at the connections, experimental justification is required for beam-to-column connections assembled with high-strength bolts with enlarged and with slotted holes. Previous research has indicated that oversize holes, size according to bolt diameter, do not adversely affect the slip behavior of friction-type joints or cause undesirable bolt-tension losses\(^{10}\). It was also observed that slotted holes did not affect the strength of bearing-type joints.
Provisions based upon these findings are now included in the Specifications for Structural Joints Using ASTM A325 or A490 Bolts as approved by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation (endorsed by American Institute of Steel Construction and Industrial Fasteners Institute)\(^{(1)}\).

In the test program, 1 1/4 in. round holes are used in top and bottom moment plates fastened with 1 in. diameter A490 bolts and designed as friction-type connection (Test C8 in Fig. 10). The use of 1/4 in. oversize holes is the maximum size permitted in current Specification\(^{(1)}\). Slotted holes are used in one-sided shear plates fastened with either A490-X bolts (C3 in Fig. 5) or A325-X bolts (C8 and C9). The remaining joints have round holes 1/16 in. larger than the nominal diameter of the bolt. Holes are to be punched, sub-punched and reamed, or drilled, as required by the AISC Spec.\(^{(6)}\).

Both A325 and A490 bolts will be installed by the turn-of-nut method. Washers are not used for A325 bolts. In bearing-type connections, A490 bolts will have a hardened washer under the element (nut or bolt head) turned in tightening. In friction-type connection (C8), a hardened washer will be inserted under both the head and nut. Joint surfaces and nut rotation from snug tight condition conform to appropriate provisions in the Specification\(^{(1)}\). All bolts will be calibrated and installed in the Fritz Laboratory.

3. Welds

The connection specimens will be welded according to the AWS Building Code\(^{(12)}\). Weld electrodes are E70XX low hydrogen. In determining the size of fillet weld, the allowable shear stress on effective throat is 21 ksi.
Nondestructive testing methods will be employed to inspect the welds before testing of the specimen. Groove welds will be inspected by ultrasonic testing and fillet welds by magnetic particle.
3. DESCRIPTION OF TEST PROGRAM

The current test program, designated as C-series, is summarized in Table 1. This series of twelve tests is designed to investigate those problems suggested in Phases 10 and 11 of the research project. Comments of the WRC Task Group members (Appendix 2) have been incorporated. Detailed discussion of the test program is given as follows.

3.1 DESCRIPTION OF SPECIMENS

Figure 3 shows the joint detail of Test C1. Beam flanges are directly welded to the column flanges providing for plastic moment capacity. A one-sided shear plate bolted with three 1 in. diameter A490-X bolts is used to resist vertical shear. Horizontal stiffeners are designed according to Section 1.15.5 of the AISC Specification\(^6\). Since the connection panel zone is under symmetrical loads, a clearance of 1/2 in. is provided between horizontal stiffeners and column web. The size of fillet welds for horizontal stiffeners is determined by computing the force taken by stiffeners when plastic moment is attained at beam-column juncture. An illustrative design procedure of Test C1 is given in Appendix 3.

Test C2 is shown in Fig. 4. Its connection type is similar to Test C1, the only difference being that horizontal stiffeners are not required.

Test C3 is identical with Test C2 except that the one-sided shear plate of Test C3, shown in Fig. 5, has slotted holes. The dimensions of these slots conform to the provision in current Specification\(^11\). A
continuous bar with 5/16 inch in thickness and having a width equal to the length of the slot is attached on the side of the slotted shear plate. (The addition of continuous bars for single shear connections was approved by the Research Council on Riveted and Bolted Structural Joints at its annual meeting on May 12, 1971.)

Figure 6 shows the joint detail of Test C4. Moment capacity is provided through direct butt welding of the beam flanges to the column flanges. Vertical shear is resisted by a two-plate welded stiffener seat which was designed according to Table VIII of the AISC Manual (6). The strength of this connection should be greater than that of Test C5 shown in Fig. 7.

In the case of Test C5, both the moment and shear are resisted by the butt welds. Test results of this connection should provide a basis for evaluating connection design procedures. An analytical study will be attempted to predict the behavior of this type of connection.

Figure 8 shows bolted top and bottom moment plate connection, Test C6. The plastic moment is carried by flange plates which are fastened with 1 in. diameter A490-X bolts. The design procedure follows the example given on page 4 - 92 of the AISC Manual (6). The bracket stiffeners are designed with the aid of Table VIII. Design alternatives for stiffener detail can be found in Ref. 13.

Test C7 is shown in Fig. 9. The vertical shear is supplied by a one-sided shear plate connected to the beam web by three 1 in. diameter A490-X bolts. Tests C1, C6, and C7 are designed for the same amount of moment
and shear, and, therefore, their behavior should be comparable.

Test C8 in Fig. 10 is designed as a friction-type connection having oversize holes in moment plates. The use of 1 1/4 in. round holes for 1 in. diameter A490-F bolts is permitted by the Specification (11). There is no reduction in slip resistance of the joint. The one-sided shear plates have slotted holes and are designed as bearing-type connections. A continuous bar is also attached on the side of the shear plate as shown in Fig. 10.

Test C9 is similar to Test C8. For the purpose of comparison, the moment plates of Test C9 are designed as bearing-type connections having round holes 1/16 in. in excess of the nominal bolt diameter.

Provisions concerning the use of oversize and slotted holes in current Specification (11) are based upon the results of tests conducted on lap joints (10). Test results of C3, C8, and C9 should provide justification for this provision when the location of the joint occurs where a plastic hinge forms and is subjected to substantial inelastic deformation.

For the purpose of rating the performance of those previously described connections, Tests C1 through C9, three control tests are included in this test program. The details of those control tests are shown in Figs. 12, 13, and 14. Beam flanges and beam web are connected to the column flanges by groove welds. An erection plate is tack welded to the column flange, and is to be used as the backing strip for the beam web groove weld.
As an aid for ready reference, section properties of beams and columns are shown in Table 2. The dimensions listed are those from the AISC Manual (6).

3.2 TEST SETUP

The test setup is shown in Fig. 15. The axial load in the column will be applied by a 5,000,000 pound-capacity hydraulic universal testing machine. The crosshead of the testing machine is indicated. The beam will be supported by two pedestals resting on the floor. Rollers will be used to simulate simply supported end conditions. In the compression region of beam-column juncture, lateral bracing will be used to provide stability. A member resting against the testing machine column will support the lateral bracing as indicated in Fig. 15.

3.3 INSTRUMENTATION

The specimen will be instrumented with SR-4 strain gages at locations on the beam, the column, and the horizontal stiffeners, as indicated in Fig. 16. Those strain gages mounted at Section A-A will provide data for constructing the moment diagram. Strain distribution in the immediate proximity of beam flange groove welds will be measured by those strain gages at Section B-B. The resultant axial force will be computed by the strain readings taken at Section C-C. The four gages at Section C-C will also be used for achieving the alignment of the column. One pair of strain gages mounted on horizontal stiffeners, shown in Section D-D, will be used to indicate the effectiveness of the stiffeners in resisting the forces coming from beam flanges.
In addition to SR-4 gages, strain rosettes will be added near shear plate welds at appropriate locations.

The absolute rotations of beam ends and beam-column juncture will be measured with level bars attached to the member webs directly inside the flanges. The locations of the level bars are shown in Fig. 16.

Finally, the over-all deflection of the connection will be monitored by a dial gage placed directly under the column.
Steel framing costs can be reduced if proper attention is given to moment-resisting beam-to-column connections. Realistic design rules for connections should consider not only strength and rigidity but also economical fabrication and erection. With the advent of high-strength steels being used in tall steel buildings, there is a need to evaluate the connection design procedures, taking into account the properties of connecting media (welds and bolts) and the details of connection components (stiffeners and plates).

In this test program a series of twelve tests is to be carried out on full-size beam-to-column connections made of ASTM A572 Grade 55 steel, fastened with A325 and A490 bolts utilizing higher allowable shear stresses. The results of these tests will be utilized to formulate design procedures that will facilitate more efficient, and therefore more economical, connections.
5. ACKNOWLEDGEMENTS

The project is sponsored jointly by the American Iron and Steel Institute and the Welding Research Council. Research work is carried out under the technical advice of the WRC Task Group, of which Mr. John A. Gilligan is Chairman. (See Appendix 1 for Task Group Roster).

This test program was developed under the supervision of Dr. Lynn S. Beedle, Project Director. Other staff members are Dr. George C. Driscoll, Jr. and Mr. David J. Fielding.

This test program has been reviewed by Dr. John W. Fisher. Members of the WRC Task Group have contributed valuable suggestions and comments. (See Appendix 2). Mr. W. E. Edwards has reviewed the manuscript of this report.

This report was typed by Mrs. Jean Neiser.

The authors are thankful to all of these.
6. APPENDICES

APPENDIX 1: WELDING RESEARCH COUNCIL
TASK GROUP ON BEAM-TO-COLUMN CONNECTIONS

(AISI Project 137, Lehigh University Project 333)

John A. Gilligan, Chairman, U. S. Steel Corporation
Victor V. Bertero, University of California
Omer W. Blodgett, Lincoln Electric Company
Hubert C. Crick, Mosher Steel Company
Carson F. Diefenderfer, Bethlehem Steel Corporation
Norman W. Edwards, Pittsburgh-Des Moines Steel Company
William E. Edwards, Bethlehem Steel Corporation
Harold J. Engstrom, Jr., AFCO Steel Company
Theodore R. Higgins, American Institute of Steel Construction
Ira M. Hooper, Seelye-Stevenson-Value-Knecht
Carl. L. Kreidler, Bethlehem Fabricators, Inc.
Hugh A. Krentz, Canadian Institute of Steel Construction
William A. Milek, Jr., American Institute of Steel Construction
Clarkson W. Pinkham, S. B. Barnes and Associates
Arch N. Sherbourne, University of Waterloo
Charles F. Larson, Secretary, Welding Research Council
APPENDIX 2: SUMMARY OF COMMENTS

Comments of Task Group members on Fritz Lab Report 333.12, "Proposal for Tests of Moment-Resistant Beam-to-Column Connections", are summarized in the following tabulation. Action taken on these comments by the Lehigh Project Staff is also indicated.

<table>
<thead>
<tr>
<th>Name</th>
<th>Date</th>
<th>Comments</th>
<th>Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gilligan</td>
<td>4/1/71</td>
<td>1. A column lighter than the W14x314 should be used with the W27x94 beam.</td>
<td>A W14x176 section was used as a replacement for the W14x314. This section is the least column size that can be used without requiring stiffeners.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Strain gages should be added near shear plate welds.</td>
<td>Incorporated</td>
</tr>
<tr>
<td></td>
<td>6/24/71</td>
<td>The release of the revised test program was approved.</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>7/6/71</td>
<td>If A572 Grade 55 steel plates are not available, a substitute steel can be used for shear plates, moment plates, and stiffeners, provided the required strength can be met. Mill report is required.</td>
<td>Incorporated</td>
</tr>
<tr>
<td>Blodgett</td>
<td>4/12/71</td>
<td>1. Web shear plate can be designed only for vertical shear.</td>
<td>Incorporated</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Design procedure for at least one connection should be included in the report.</td>
<td>Incorporated (See Appendix 3)</td>
</tr>
<tr>
<td>Edwards, W.E.</td>
<td>4/13/71</td>
<td>1. Control specimens should be added to the program for performance ratings.</td>
<td>Tests C10, C11, and C12 are control tests for this.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Beam span criterion is to be based upon ( M ) being reached concurrently with the factored capacity of single</td>
<td>Incorporated</td>
</tr>
<tr>
<td>Name</td>
<td>Date</td>
<td>Comments</td>
<td>Action</td>
</tr>
<tr>
<td>---------------</td>
<td>---------</td>
<td>--------------------------------------------------------------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>Edwards, W. E.</td>
<td>4/26/71</td>
<td>shear bolts in a single line.</td>
<td>Incorporated</td>
</tr>
<tr>
<td>(cont'd)</td>
<td></td>
<td>3. The lightest column size for which stiffeners are not required should be used.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4/26/71</td>
<td>An alternate test program was suggested.</td>
<td></td>
</tr>
<tr>
<td>Higgins</td>
<td>6/14/71</td>
<td>Agreed with the revised program.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>An upper bound moment diagram for the panel zone is shown in Fig. 2.</td>
<td></td>
</tr>
<tr>
<td>Kreidler</td>
<td>4/7/71</td>
<td>Edge distances conform to Table 1.16.5 of the AISC Specification.</td>
<td>Incorporated</td>
</tr>
<tr>
<td>Krentz</td>
<td>6/22/71</td>
<td>Test C5 should have an erection seat or an erection clip to simulate actual field conditions.</td>
<td>Test C5 has neither an erection seat nor an erection clip. The purpose is to determine the actual capacity of the beam flange groove welds.</td>
</tr>
<tr>
<td>Milek</td>
<td>4/1/71</td>
<td>The moment diagram in the column web panel zone should be shown in the report.</td>
<td>The fabricator will inspect welds by ultrasonic testing.</td>
</tr>
<tr>
<td>Pinkham</td>
<td>6/15/71</td>
<td>1. Is Lehigh or the fabricator going to inspect specimens ultrasonically before testing?</td>
<td>Practice varies. The groove weld indicated is to be made from the bottom because in the shop it can be done with the column resting on the floor.</td>
</tr>
</tbody>
</table>
APPENDIX 3: DESIGN PROCEDURE OF TEST C1
(W14x74 Beam and W10x60 Column)

1. Determine beam span (L).

Plastic Moment

\[ M_p = F \frac{Z_y}{x} = (55)(126) = 6930 \text{ kip-in} \]

Design Ultimate Shear

Assuming three 1" diam. A490 bearing type bolts with threads excluded from shear plane:

\[ V = 3 \frac{(0.7854)(40)(1.7)}{9} = 160 \text{ kips} \]

Beam Span

\[ L = \frac{M_p}{V} = 43.3 \text{ in.} \]

Use: \( L = 43 \) in.

2. Determine size of shear plate and weld.

Shear Plate

Pitch for 1" diam. bolts is 3 in.

Edge distance from Table 1.16.5 of AISC Spec. is 1 3/4 in.

Length of shear plate is 2 @ 3" + 2 @ 1 3/4" = 9.5 in.

Allowable shear stress: \( F_y = 0.40F_y = 0.40(55) = 22 \text{ ksi} \)

Required thickness of shear plate:

\[ t = \frac{160}{(22)(9.5)(1.7)} = 0.45 \text{ in.} \]

Use: 1/2" x 5 1/4" x 9 1/2" shear plate, \( F_y = 55 \text{ ksi} \).

Fillet Weld

Allowable shear stress of weld using F70XX electrodes is 21 ksi.

Size of fillet weld for shear:

\[ w = \frac{160}{(21)(1.7)(2)(9.5)(0.707)} = 0.334 \text{ in.} \]

Size of fillet weld required for developing shear yield strength of web plate:

\[ w = \frac{(0.5)(22)}{(2)(0.707)(21)} = 0.371 \text{ in.} \]

Minimum size of fillet weld given by Table 1.17.5 of the AISC Specification, based upon the flange thickness of 0.683 in. for W10x60 column, is 0.25 in.

Use: 3/8 in. fillet weld, 9.5 in. each side.

Note: The bearing stresses on shear plate and beam web are within allowable limits. The design concept will be treated elsewhere.
3. Design horizontal stiffeners.

Horizontal stiffeners opposite the compression flange:
Determined by AISC Specification:
\[
t < \frac{C_1 A_f}{t_b + 5k} = \frac{(1)(10.072)(0.783)}{0.783 + 5(1.25)} = 1.12 \text{ in.}
\]
\(t\) for W10x60 column is 0.415 in.; hence stiffeners are required.

\[
t \leq \frac{d_c \sqrt{F_y}}{180} = \frac{7.75 \sqrt{55}}{180} = 0.319 \text{ in.}
\]
Determined by Fritz Lab Report 333.14 "Strength and Stability of Column Web in Welded Beam-to-Column Connections" by Newlin and Chen:
\[
t \leq \frac{d_c \sqrt{F_y}}{180} + 180 C_1 A_f \frac{4 \sqrt{F_y}}{125 d_c}
\]
\[= \frac{(7.75)^2 \sqrt{55} + 180 (1) (10.072)(0.783)}{125 (7.75) \sqrt{55}} = 0.708 \text{ in.}
\]
t provided is 0.415 in.; hence stiffeners are required.

Horizontal stiffeners opposite the tension flange:
\[
t_f < 0.4 \sqrt{C_1 A_f} = 0.4 \sqrt{(1)(10.072)(0.783)} = 1.122 \text{ in.}
\]
\(t_f\) for W10x60 column is 0.683 in.; hence stiffeners are required.

Size of horizontal stiffeners:
\[
A_s \geq \left[ C_1 A_f - t (t_b + 5k) \right] C_2
\]
\[= (10.072)(0.783) - (0.415)(0.783 + 5 \times 1.25) = 4.96 \text{ in.}^2
\]
Try a pair of 3/4" by 4" stiffeners
\[
A_s = 2 (0.75)(4) = 6 \text{ in.}^2 \quad \text{o.k.}
\]
Check length-thickness ratio (see p. 21 of Ref. 14):
As indicated in the figure, consider the stiffener simply supported along the edges welded to the column flanges.
\[
\sigma_{cr} = \frac{\pi^2 E}{12(1-\nu^2)} \left( \frac{t_s}{d_w} \right)^2
\]
For \(\sigma_{cr} = \sigma_y = 55 \text{ ksi}, \nu = 0.3, E = 29000 \text{ ksi}, \frac{d_w}{t_s} = 21.8
Length of stiffeners = 10 1/4 - 2(11/16) = 8 7/8 in. \( d_w/t_s = 11.8 < 21.8 \) o.k.

Use: 2 plates 3/4" x 4" x 8 7/8" at both tension and compression flanges, \( F_y = 55 \) ksi.

4. Determine size of fillet welds on the horizontal stiffeners.

Force in beam flanges:

\[
T = \frac{M_p}{d_b} = \frac{6930}{14.19} = 488 \text{ kips}
\]

There are two methods for computing force in horizontal stiffeners.

Method 1:
Force in column web:

\[
T_w = F_y (t_b + 5k) t = (55)(0.783 + 5 \times 1.25)(0.415) = 160 \text{ kips}
\]

Force left for horizontal stiffeners:

\[
T_s = T - T_w = 328 \text{ kips}
\]

Method 2:
Force in horizontal stiffeners:

\[
T_s = T \left( A_s \left( A_w/A_s \right) \right)
\]

Substituting \( A_s = 6 \) sq. in. and \( A_w = (0.783 + 5 \times 1.25)(0.415) = 2.92 \) sq. in., \( T_s = 328 \) kips.
Force causing yielding in horizontal stiffeners is \( T_s = F \cdot A_y \cdot y_s = (55)(6) = 330 \) kips.

The fillet welds will be sized for transmitting the force of \( T_s = 328 \) kips.

\[
\frac{328}{(21)(1.7)(2)(8)(0.707)} = 0.812 \text{ in.}
\]

Minimum size of fillet weld given by Table 1.17.5 of the AISC Specification based upon \( t_s = 0.75 \) in. is 0.25 in.

Use: 3/4 in. fillet weld.

Note: It has been known that fillet welds when pulled transverse to their axis were at least 1/3 stronger than when pulled parallel. Hence, the use of 3/4 in. fillet weld is reasonable.
<table>
<thead>
<tr>
<th>PHASE</th>
<th>TEST</th>
<th>BEAM SIZE</th>
<th>COLUMN SIZE</th>
<th>PROB. IN 333.7</th>
<th>FACTORED LOAD</th>
<th>STIFFENING (Prob.1.5B)</th>
<th>BEAM SPAN</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MOMENT</td>
<td>SHEAR</td>
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<td></td>
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<td>V=160K(88.5%V) Shear Plate3-1&quot; A490-Xin 1 1/16&quot; Round Holes</td>
<td>Horizontal Stiffeners 3/4&quot;x4&quot;x 8 7/8&quot;</td>
<td>3'-7&quot;</td>
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<td>V=374K(94.7%V) Shear Plate 7-1&quot;)A490-X in 1 1/16&quot;Rnd.Holes</td>
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<td>3'-5&quot;</td>
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<td></td>
<td>V=160K(88.5%V) Shear Plate1&quot;x5&quot;x11&quot;</td>
<td>-</td>
<td>3'-5&quot;</td>
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<td>V=160K(88.5%V) Shear Plate 3-1&quot;)A490-Xin 1 1/16&quot; Round Holes</td>
<td>DO</td>
<td>3'-7&quot;</td>
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<tr>
<td></td>
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<td></td>
<td></td>
<td>V=157.5K(52.5%V) Shear Plate 7-3/4&quot;)A325-Xin Slotted Holes</td>
<td>-</td>
<td>4'-5&quot;</td>
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<tr>
<td></td>
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<td></td>
<td>DO</td>
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<td>4'-5&quot;</td>
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<td>V=160K(88.5%V) Shear Plate3-1&quot;)A490-Xin 1 1/16&quot; Round Holes</td>
<td>Horizontal Stiffeners 3/4&quot;x4&quot;x 8 7/8&quot;</td>
<td>3'-7&quot;</td>
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<td>V=157.5K(52.5%V) Shear Plate 7-3/4&quot;)A325-Xin Slotted Holes</td>
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<td>4'-5&quot;</td>
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<td></td>
<td></td>
<td>DO</td>
<td>-</td>
<td>3'-5&quot;</td>
</tr>
</tbody>
</table>

**Notes:**

a. All specimens are made of ASTM A572 Grade 55 steel.

b. \( h_a \) for all specimens is 2'-6". \( h_b \) is 1'-3".

---

**TABLE 1 TEST PROGRAM OF C-SERIES**
### TABLE 2  SECTION PROPERTIES FOR DESIGNING

<table>
<thead>
<tr>
<th>SECTION</th>
<th>AREA $A$ (In.$^2$)</th>
<th>DEPTH $d$ (In.)</th>
<th>WIDTH $b_f$ (In.)</th>
<th>THICKNESS $t_f$ (In.)</th>
<th>WEB THICKNESS $t_w$ (In.)</th>
<th>$I_x$ (In.$^4$)</th>
<th>$S_x$ (In.$^3$)</th>
<th>$Z_x$ (In.$^3$)</th>
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</thead>
<tbody>
<tr>
<td>BEAM</td>
<td></td>
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<td></td>
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<tr>
<td>W14x74</td>
<td>21.8</td>
<td>14.19</td>
<td>10.072</td>
<td>0.783</td>
<td>0.450</td>
<td>797</td>
<td>112</td>
<td>126</td>
</tr>
<tr>
<td>W24x61</td>
<td>18.0</td>
<td>23.72</td>
<td>7.023</td>
<td>0.591</td>
<td>0.419</td>
<td>1540</td>
<td>130</td>
<td>152</td>
</tr>
<tr>
<td>W27x94</td>
<td>27.7</td>
<td>26.91</td>
<td>9.990</td>
<td>0.747</td>
<td>0.490</td>
<td>3270</td>
<td>243</td>
<td>278</td>
</tr>
<tr>
<td>COLUMN</td>
<td></td>
<td></td>
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<td>W10x60</td>
<td>17.7</td>
<td>10.25</td>
<td>10.075</td>
<td>0.683</td>
<td>0.415</td>
<td>344</td>
<td>67.1</td>
<td>75.0</td>
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<tr>
<td>W14x136</td>
<td>40.0</td>
<td>14.75</td>
<td>14.740</td>
<td>1.063</td>
<td>0.660</td>
<td>1590</td>
<td>216</td>
<td>243</td>
</tr>
<tr>
<td>W14x176</td>
<td>51.7</td>
<td>15.25</td>
<td>15.640</td>
<td>1.313</td>
<td>0.820</td>
<td>2150</td>
<td>282</td>
<td>321</td>
</tr>
</tbody>
</table>

![Diagram of section properties](image-url)
Fig. 1 Moment-Rotation Curves
Fig. 2 Procedures of Theoretical Analysis
Section A-A

Scale:

0  5  10 in.

Fig. 3 Test Cl
333.15

Fig. 4 Test C2
Plate with 1\(\frac{1}{16}\)" Round Holes (A36)

W14 x 176 (\(F_y = 55\text{ ksi}\))

Shear Plate (\(F_y = 55\text{ ksi}\))

7-1" \(\phi\) A490 - X Bolts in Slotted Holes

3\(\frac{1}{8}\)" x 1" x 12"

Backing Strip (A36)

Typ.

Scale:

0 5 10 in.

Fig. 5 Test C3
Fig. 6 Test C4
Fig. 7 Test C5
Fig. 8 Test C6
8 - 1" φ A490-X Bolts in 1\(\frac{1}{16}\)" Round Holes

Elevation

Section A-A

Fig. 9 Test C7
14 - 1" φ A490-F Bolts in 1¼" Round Holes

3/8" Strip Shims

7 - 3/4" φ A325-X Bolts in Slotted Holes

3/8" x 1" x 13" Backing Strip (A36) (Typ.)

5/16" x 2" x 20½" Plate with 3/16" Round Holes (A36)

W14 x 136 (F_y = 55 ksi)

Elevation

Sym.

3/16" x 11" x 24 3/8" Moment Plate (F_y = 55 ksi)

Plan View

Scale:

Fig. 10 Test C8
A490-X Bolts in \( \frac{3}{8} " \times \frac{1}{2} " \) Round Holes

Back ing Strip (A36) (Typ.)

\( \frac{3}{8} " \times \frac{5}{4} " \times 20\frac{1}{2} " \) Shear (\( F_y = 55 \text{ ksi} \))

\( \frac{3}{8} " \times 1 " \times 13 " \) Back ing Strip (A36) (Typ.)

W24 x 61 (\( F_y = 55 \text{ ksi} \))

\( 3 @ 3 " = 9 " \)

\( d/2 \)

d = 23\frac{3}{4} "

\( \frac{3}{8} " \) Strip Shims

\( \frac{3}{4} " \times 11 " \times 12 \frac{3}{8} " \) Moment Plate (\( F_y = 55 \text{ ksi} \))

\( \frac{3}{8} " \) Gage

Scale:

0 5 10 in.

Fig. 11 Test C9
**Sym. A**

**Sym. I/A**

Clearance

W10 x 60

(F_y = 55 ksi)

13/16" 3/8" 10 1/8

30°

1 1/4" 1/4" 3/8" x 4" x 11 1/4"

Erection R (A36)

d/2
c

d = 14 1/4"

W14 x 74

(F_y = 55 ksi)

1/4"

4 3/8"

2 3/4" 4 3/8" Erection

Bolts in 13/16" Holes

3/8" x 1" x 12"

Backing Strip (A36)

(Typ.)

**Sym.**

1/2" Clearance

2 Tack Welds to Column

3/4" x 4" x 8 7/8"

Stiffener R (F_y = 55 ksi)

(Typ.)

**Section A-A**

**Scale:**

0 \ 5 \ 10 in.

**Fig. 12 Test C10**
Sym.

Elevation

W14 x 136 (Fy = 55 ksi)

(For)

Erection P (A36)

Bolts in 13/16" Holes

2 - 3/4" φ A307 Erection

Backing Strip (A36)

(Typ.)

Sym.

Section A-A

3 Tack Welds to Column

Scale:

0 5 10 in.

Fig. 13 Test C11
Fig. 14 Test C12
Fig. 15 Test Setup
SR-4 Strain Gages

Level Bars for Rotation Measurement

Deflection Dial Gage

Fig. 16 Instrumentation
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