TEST PROGRAM OF
MOMENT-RESISTANT STEEL
BEAM-TO-COLUMN WEB CONNECTIONS

by
Glen P. Rentschler
Wai-Fah Chen
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Glenn P. Rentschler
and
Wai-Fah Chen

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

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TABLE OF CONTENTS

1. INTRODUCTION
   1.1 PURPOSE
   1.2 BACKGROUND
   1.3 PREVIOUS RESEARCH
   1.4 STATEMENT OF THE PROBLEM

2. DEVELOPMENT OF TEST PROGRAM
   2.1 OBJECTIVE OF RESEARCH
   2.2 PILOT TESTS
   2.3 FULL-SCALE TESTS
      2.3.1 Steel Strength
      2.3.2 Member Size and Beam Span
      2.3.3 Fasteners and Holes
      2.3.4 Welds

3. DESCRIPTION
   3.1 DESCRIPTION OF SPECIMENS
      3.1.1 Connection 14-1: Flange-Welded to Both Column Web and Flanges, and Web-Bolted
      3.1.2 Connection 14-2: Flange-Welded to Column Web Only and Web-Bolted
      3.1.3 Connection 14-3: Fully Bolted
      3.1.4 Connection 14-4: Fully Welded (Control Test)
   3.2 TEST SETUP
   3.3 INSTRUMENTATION

4. SUMMARY

5. ACKNOWLEDGMENTS

6. APPENDICES
   6.1 WRC Task Group Roster
   6.2 Beam-to-Column Connection Research Phases
   6.3 Design Procedure of Test 14-1

7. TABLES

8. FIGURES

9. REFERENCES
1. INTRODUCTION

1.1 PURPOSE

The purpose of this report is to present a complete program of tests of moment-resistant steel beam-to-column web connections. The content of this report is based on discussions held at meetings of the members of the Welding Research Council Task Group on Beam-to-Column Connections at Lehigh University on June 15, 1973 (Ref. 1) and June 21, 1974 (Ref. 2).

At the first meeting, initiation of work on beam-to-column connections was begun, with discussion at that meeting centering primarily on the configuration of specimens to be tested. A preliminary report (Ref. 3) was reviewed at this meeting and served as a basis for further action.

At the second meeting, further progress was made on particular details of the connections to be tested. Reference 4 was presented as a draft of this report and again stimulated much discussion and input by Task Group members. Other ideas concerning items such as pilot tests, testing set-up, steel strength, and relevance of testing with column axial load were presented by Task Group members at the meeting.

It is the purpose of this report to consolidate the ideas from these previous two meetings and finalize the testing program for unsymmetrical steel beam-to-column web connections.
1.2 BACKGROUND

One of the most influential elements in the behavior and cost of multi-story building frames is the moment resisting beam-to-column connection. A majority of these connections are column flange connections where the beam frames into the column flange. Considerable research work has been done on this type of connection. However, another type of moment resisting connection commonly found in building frames is the web connection. In this connection the beam is attached to the column web, with the beam tending to bend the column about its weak axis. A study of this type of connection is the objective of the test program described herein. The study will concentrate only on unsymmetrical web connections (beam on one side of column only).

The need for research on web connections was recognized by the Welding Research Council Task Group on Beam-to-Column Connections. A list of the Task Group members is given in Appendix 1. The web connection, referred to as Prob. 2.1 (Phase 14), is listed in Ref. 5 along with other types of connections used extensively in steel building frames and which are of particular interest to designers. Reference 5 is an interim report prepared, through the assistance of the WRC Task Group, to indicate the basic problems pertaining to connection study and to suggest some possible areas of needed research work. As a result of the work of the WRC Task Group, much research, both past and present, has been conducted to solve some of these connection problems. Shown in Appendix 2 is a list of the phases of connection research activity as suggested by the Task Group, some of which have been completed to date.
Although web connections are used quite often today in buildings, a thorough knowledge of their behavior remains relatively unknown. Questions often posed are:

1. What effect will a concentrated shear force have on the column web?

2. What criteria govern the need for horizontal stiffeners on the opposite side of the column from the beam in such connections?

3. In welded web connections, should the beam flange or beam flange connection plates be welded to the column flange or is merely welding to the column web sufficient?

4. Should groove or fillet welds be used for such attachments?

5. What effect does column axial load have on web connection performance?

6. Should the connections be welded, bolted, or a combination of welding and bolting?

These questions can only be answered by full scale testing of connection specimens with monitoring under laboratory conditions.

1.3 PREVIOUS RESEARCH

Previous research on web connections in the United States has been confined to static testing of symmetric web connections and testing of unsymmetrical web connections under repeated and reversed loading.

The static tests were conducted at Lehigh University and are reported in Refs. 6 and 7. The tests reported in Ref. 6 deal with
symmetrical web connections with emphasis on shear connections rather than moment connections. Reference 7 research deals with four-way beam-to-column moment connections.

Research on unsymmetrical web connections under repeated and reversed loading is reported in Refs. 8 and 9. Other than the obvious difference of testing under static versus repeated and reversed loading, the current Lehigh program differs from the University of California program (Refs. 8 and 9) in the following points

1. Specimens are made of ASTM A572 Grade 50 steel.
2. Connections are tested under a higher ratio of shear to moment.
3. A higher allowable shear value of 40 ksi for A490 bolts is used for bolts in bearing type connections.

1.4 STATEMENT OF THE PROBLEM

In the test program presented in this report, a two-pronged study of web connections is planned. The first part consists of small pilot tests to be used to assess the effects of applying concentrated flange forces on a column section. Some information on web connection behavior can thus be obtained without the expense of fabricating full-scale connections. This part of the test program is only described briefly herein. Reference 10 presents complete details of the pilot test program.

The second part of the program deals with testing of full-scale beam-to-column web connections under monotonic loading. This procedure provides a complete picture of connections as would occur in field situations.
2. DEVELOPMENT OF TEST PROGRAM

2.1 OBJECTIVE OF RESEARCH

The objective of this study is to investigate the performance of specified beam-to-column web connections under unsymmetrical loading. Although symmetrically loaded web connections are often found in the interior of building frames, this program will concentrate on the more critical case of unsymmetrical loading, the type of loading normally found on the outer column line of buildings. Primary attention is focused upon the moment capacity, rotation capacity, and overall stiffness of web connections.

Figure 1 shows schematically the load-deflection behavior of a beam-to-column web connection under unsymmetrical loading. If a connection is properly proportioned and the welds 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 large angle as indicated by curve A. Curve A also shows that in the elastic range the connection exhibits overall stiffness for maintaining the location of beams and columns relative to each other. Curves B, C, and D show the performance of connections which is unsatisfactory. In this study, the connections are designed to achieve the behavior represented by curve A.

2.2 PILOT TESTS

The objective of the pilot tests, which were recommended by the WRC Task Group (2), is to obtain as much information as possible concerning the behavior of columns in web connections without going
to the expense of fabricating full-scale specimens. In web connections, the column, and method of attachment of the beam to the column, can be very critical. For example, if the beam flanges are attached only to the column web rather than the column flanges, severe distortion of the column cross-section or possible tearing out of the column web could occur. Severe distortions of the column web can also occur due to the application of a concentrated shearing force as would result from a force from the beam web.

To monitor the behavior of the column under the application of a beam bending moment, a series of moment pilot tests are planned. Figure 2 shows the test set-up to be used in conducting two tests simultaneously. The combination of the downward force and upward reaction on each of the column sections provides a couple simulating the action of tensile and compressive beam flange forces on the column. The distance between the plates on each test will simulate normal beam depths. Several different geometries of attaching the plates to the column will be used, because it is felt that column behavior can vary greatly with the way the plates are connected. The moment pilot tests will be conducted on two column sections--W12x106 and W14x184. These sections are made of ASTM A572 Grade 50, the same as planned for the full-scale tests. Complete details of the pilot tests are given in Ref. 10.

Pilot tests simulating the action of beam shear on the column web will not be conducted because it was felt by the WRC Task Group that such tests did not have as much merit as the moment tests for aiding in design of the full-scale specimens.
Since the expenses involved in such testing are minimal once the column sections are obtained, the number of these pilot tests to be performed will be left open. It will not be attempted to necessarily conduct all pilot tests on the sections made of the same steel as will be used in the fully fabricated specimens, but rather to do such tests on sections and steel strengths that are readily available.

2.3 FULL-SCALE TESTS

Four full-scale web connections have been designed and are shown in Figs. 4 through 7. The four connections were selected by the WRC Task Group as those web connections commonly used in buildings today on which more knowledge is required. The connections are full-scale, using realistic beam and column sections, unsymmetrically loaded by an increasing monotonic load to simulate static conditions. A general beam and column web connection assemblage is shown in Fig. 3.

The specimens are designed according to the AISC Specification (11). 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 Sec. 1.5 of the AISC Specification (11). For A490 high-strength bolts in bearing-type connections the design shear stresses used are equal to 1.7 times 40 ksi, instead of 32 ksi as suggested in the current Specification. The concept for this procedure will be discussed later.
2.3.1 Steel Strength

Steel: ASTM A572 Grade 50

The steel strength selected for use in fabricating the four full-scale specimens is ASTM A572 Grade 50. This steel was selected for two reasons. First, on a high strength steel such as this, there is a narrower margin between yield and ultimate than for lower strength steels. The plastic range is somewhat less compared to A36 steel. Thus, if the connection behavior is adequate for A572 steel, the results could be assumed to apply to lesser grade steels. Secondly, this steel was selected due to its anticipated availability to avoid long delays in fabrication.

2.3.2 Member Size and Beam Span

Beam: W27x94
Column: W14x176
Beam Span: 48 in. (to critical section)

The beam and column sizes selected were a W27x94 and a W14x176 respectively. The connection specimens are chosen to have an appropriate combination of a beam section and a column section which represents the realistic beam-to-column web 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.
The ratio of one flange area to the web area of the W27x94 beam is 0.60.

The column section was chosen so as to avoid a failure in the column outside the connection. To accomplish this, a column section was selected so that twice the value of the plastic moment $M_p$ of the column about its weak axis was greater than the $M_p$ of the beam, where $M$ is defined as

$$M_p = Z \sigma_y$$  \hspace{1cm} (1)

$Z$ = plastic modulus

$\sigma_y$ = steel yield stress

The specimens are proportioned such that the section at the beam-column juncture can resist $M_p$ and 80.7% of $V_p$ where the shear value producing full yield, $V_p$, is defined as

$$V_p = \frac{\sigma_y w}{\sqrt{3}} d_w$$  \hspace{1cm} (2)

where $\sigma_y$, $w$, and $d_w$ are yield stress level, web thickness and web depth of wide-flange shape respectively. The beam span to the critical section from application of the beam load is then simply the ratio of $M_p$ to 80.7% $V_p$. The critical section for Connections 14-1 and 14-4 is where the beam is connected to the flange and web connection plates. For Connection 14-2, the critical section is at the column web, and for Connection 14-3, it is at the outer row of flange bolts. For the sections used in this program, the critical beam span is 48 inches.
2.3.3 Fasteners and Holes

<table>
<thead>
<tr>
<th>Bolts:</th>
<th>ASTM A490</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type:</td>
<td>Bearing</td>
</tr>
<tr>
<td>Stress (shear):</td>
<td>40 ksi</td>
</tr>
<tr>
<td>Holes:</td>
<td>Regular</td>
</tr>
<tr>
<td>Tightening:</td>
<td>Turn of Nut</td>
</tr>
</tbody>
</table>

In connections where some of the elements are bolted, ASTM A490 bolts are used to assemble the joint. In bearing-type connections, the allowable shear stress used in design for A490 bolts is 40 ksi. This value reflects the logical design criterion which would result if an adequate factor of safety were 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 (12). Tests have been subsequently carried out to substantiate the suggested design criterion, especially the use of A490 bolts in A440 and A514 steel joints (13,14). Also, the higher allowable shear stresses have proved satisfactory when used in Phases 10 and 11 beam-to-column connection tests (15).

Although both oversize holes and slotted holes are desirable to facilitate erection adjustments, the merit of using holes cut in this manner has already been proven in Phases 10 and 11 to work very well and further experimental justification is not required. It was observed that slotted holes did not affect the strength of bearing-type joints (15). For this reason, it was decided to use round holes instead of slotted holes to assess their effects on the behavior of bolted beam-to-column connections.
Provisions based upon the excellent results obtained by using slotted and oversize holes 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) (16).

In the test program, 1-1/16" round holes are used in the top and bottom moment plates fastened with 1 inch diameter A490 bolts in Specimen 14-3 which is designed as a bearing-type connection. Round holes 1/16" greater in diameter than the bolt size are used in the one-sided shear plate of specimen 14-1, 14-2, and 14-3. Holes are to be punched, sub-punched and reamed, or drilled as required by the AISC Spec. (11).

The A490 bolts will be installed by the turn-of-nut method. In bearing-type connections, A490 bolts will have a hardened washer under the element (nut or bolt head) turned in tightening. Joint surfaces and nut rotation from snug tight condition conform to appropriate provisions in the Specification (16). All bolts will be calibrated and installed in the Fritz Laboratory.

2.3.4 Welds

The connection specimens will be welded according to the AWS Building Code (17). Weld electrodes are E70XX low hydrogen. In determining the size of fillet weld, the design shear stress on effective throat is 1.7 times 21 ksi. Ultrasonic tests will be conducted on the welds to check for defects.
3. DESCRIPTION OF TEST PROGRAM

Since the pilot tests do not constitute a part of the formal program of web connections, detailed discussion of the geometrical configuration of these specimens to be tested will not be presented here. It should be pointed out that the configuration of these specimens will closely approximate those of the full scale tests or other web connections in Ref. 3. See Ref. 10 for specific pilot test details.

The four full-scale specimens designed for study in this program are shown in Figs. 4 through 7. This series of tests is designed to investigate the topic of web connections in Phase 14 of the research described in Ref. 5. Detailed discussion of the full-scale test program is given as follows.

3.1 DESCRIPTION OF SPECIMENS

The four web connections described below were selected by the WRC Task Group as those connections from Ref. 3 requiring further immediate investigation. Since these web connections deal with research activity under phase 14 of the program of overall connection studies, the four connections were numbered 14-1 through 14-4. Of the four connections, 14-1 received the most support for further study and 14-4 the least, thereby dictating the sequence of numbering within the group. Because Connection 14-4 is fully welded, this connection will be considered the control test.

Each of the web connections has an equivalent (geometrical configuration) column flange connection in the recently completed
program of connection study classified as Phases 10 and 11. Table 1 compares each web connection with its equivalent in Phases 10 and 11.

As an aid for ready reference, the four web connections are described briefly in Table 2.

Presented in Appendix 3, as an example, is the detailed design of Connection 14-1.

3.1.1 **Connection 14-1: Flange-Welded to Both Column Web and Flanges, and Web-Bolted**

Figure 4 shows the joint detail for test 14-1. Beam flanges are groove welded to the flange moment plates which in turn are fillet welded to the column web and flanges. A one-sided shear plate bolted with seven 7/8-inch diameter A490 high strength bolts is used to resist vertical shear. The shear plate is fillet welded to both the column web and flange moment plates. Round holes 1/16" greater than the bolt diameter are used in the web plate and beam web. The flange moment plates are 3/4" thick which is the thickness of the beam flanges, and the web plate is 1/2" thick, which is the beam web thickness.

3.1.2 **Connection 14-2: Flange-Welded to Column Web Only and Web-Bolted**

Connection 14-2 shown in Fig. 5 is a connection similar to connection 14-1, except in this connection no intermediate flange moment plates are used. The flanges of the beam are butt welded directly to the column web. Vertical shear is resisted by a shear plate fillet welded directly to the column web and bolted with seven 7/8-inch diameter A490 high strength bolts in 15/16" diameter holes to the beam web. The
web shear plate is again 1/2" thick. In this connection as in connections 14-1 and 14-3, the web shear plates which are attached to the column web are welded 1/2" off of the transverse web centerline so that the web of the beam frames into the column exactly on this transverse web centerline.

3.1.3 **Connection 14-3: Fully Bolted**

Test 14-3 is shown in Fig. 6. It is a bolted top and bottom moment plate connection. The top and bottom moment plates are bolted to the beam flange by ten one-inch diameter A490 high strength bolts in 1-1/16" round holes. These plates are then fillet welded to the column web and flanges. This bolted flange connection is designed as a bearing connection. The beam shear is transferred to a one-sided web shear plate by means of seven 7/8-inch diameter A490 high strength bolts in 15/16" diameter holes. The web shear plate is fillet welded to the column web and to the top and bottom moment plates. The web shear plate is again 1/2" thick.

3.1.4 **Connection 14-4: Fully Welded (Control Test)**

Figure 7 shows the joint detail of connection 14-4. This connection is similar to 14-1 in that the beam flanges are butt welded to the flange moment plates which are in turn fillet welded to the column web and flanges. However, in this connection the beam web is groove welded to the shear plate to transfer the beam shear. The web shear plate is again fillet welded to both the column web and flange moment plates. The thicknesses of the flange moment plates and web shear plates are similar to those of connection 14-1. This fully welded control test will serve as a basis of comparison as Connection C12 did in Phases 10 and 11 (Ref. 19).
3.2 TEST SETUP

The proposed test setup for the full-scale tests is shown in Fig. 8. Because tests (Ref. 18) indicated that axial load has some effect on yielding and deformation of connections, a constant axial load will be applied to the column by a 5,000,000 pound capacity hydraulic universal testing machine. The load on the column will be such that the combination of the constant column axial load and the beam load at maximum (assumed to occur when beam reaches $M_p$) will be $0.5 \frac{P}{P_y}$ of column. This value was selected from Fig. 9 as being the axial load at which the probability of failure of the beam and column are equalized.

The crosshead and bracing for the crosshead are indicated in Fig. 8. The column will have fixed ends with a length of column between fixed ends of fifteen feet. This length was selected so that the distance between inflection points in the column is ten feet which simulates a distance between inflection points commonly found in buildings.

To provide stability, lateral bracing will be provided at the beam-column junction. A member resting against the universal testing machine column will support the lateral bracing.

The load to the beam will be applied by means of a portable hydraulic jacks bolted to the floor and pushing upward on the beam in small increments to simulate static loading.

3.3 INSTRUMENTATION

The specimens will be instrumented at locations on the beam, the column, and the connection plates with electrical resistance strain
gages, as indicated in Fig. 10. The linear gages at Sec. A will provide
information for constructing the beam moment diagram. The strain
rosettes at this section will furnish the information on shear distri­
bution. The gages at Sec. B will be used to monitor the axial load,
as well as any bending stresses, in the column. Linear gages will be
applied to the flange moment plates (Sec. C) and rosettes to the
web shear plates of specimens 14-1, 14-3, 14-4 to determine the
distribution of linear and shear stresses within the connection region.
Linear gages and rosettes will also be placed on the column web in the
panel zone (view D).

The rotations of the column ends and the beam at the beam-
column junction will be measured with level bars attached to the
member webs directly inside the flanges. The location of the level
bars is shown in Fig. 10.

Finally, deflections will be measured by dial gages as shown
in the figure.
4. **SUMMARY**

Although there has been extensive research conducted on many topics related to building design, one of the most over-looked areas in steel building frames has been the beam-to-column connection. This is especially true of web connections. The use in buildings today of such connections has been based mainly on the fact that they have performed well in the past. There are no studies available that accurately predict their behavior. Also, information is not available on proper design concepts. This design information should not only be concerned with strength, but also with rigidity and economy of fabrication and erection.

In this test program a series of four tests is to be conducted on symmetrical full size beam-to-column web connections made of ASTM A572 Grade 50 steel loaded monotonically. The column section is a W14x176 and the beam section is a W27x94. Also planned prior to the full scale testing are several pilot tests to determine column behavior without the testing of full-scale connections. The full-scale specimens are to be fabricated by bolting, welding, and a combination of bolting and welding. The results of these tests will be utilized to accurately predict web connection behavior and formulate design procedures that will facilitate more efficient connections.
5. ACKNOWLEDGMENTS

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 for Task Group Roster).

This test program was developed under the supervision of Dr. Lynn S. Beedle, Project Director. Dr. George C. Driscoll, Jr. is also a staff member.

The authors are grateful to Dr. Roger G. Slutter who made valuable suggestions and comments on the testing program, to Mr. E. A. Burroughs for his help in designing the connections, to Mr. W. E. Edwards for reviewing the test program, and to Miss Shirley Matlock who typed this proposed test program.
6. APPENDIX I

WELDING RESEARCH COUNCIL - AISI
TASK GROUP ON BEAM-TO-COLUMN CONNECTIONS

John A. Gilligan (Chairman), U.S. Steel Corporation
Victor V. Bertero, University of California, Berkeley
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-Steveson-Value-Knecht, Consulting Engineers, N. Y.
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, Los Angeles
Archibald N. Sherbourne, University of Waterloo
Vincent R. Cartelli, Severud-Perrone-Sturm-Bandel, Consulting Engineers, N. Y.
Charles F. Larson, Secretary, Welding Research Council

Current coordination with Dr. H. S. Lew of the National Bureau of Standards.
APPENDIX 2

BEAM-TO-COLUMN CONNECTION RESEARCH PHASES

1. Pilot tests on small welded (stiffened) connections cut from old test frames. (complete, Report 333.2)

2. Theoretical analysis of column load-moment interaction. (deferred)


4. Preparation of design table for connection shear stiffening requirements using results of (2) above. (deferred)

5. Major test on a large connection with no shear stiffening. (complete) 333.4, 333.9

6. Analysis of effect on structure of omitting shear stiffening. (complete) 333.16, 333.16A

7. Tests to check column web buckling formula especially for higher strength steels. (complete) 333.8, 333.10, 333.11, 333.14

8. Study of stiffening requirements for beams of different depth on opposite sides of interior connection. (deferred)

9. Basic study of plastic deformation and strain hardening in a thick plate subject to high shear stress. (current)

10. Flange-welded web-bolted connections. (current) 333.12, 333.15, 333.17, 333.20, 333.21, 333.23, 333.30

11. Bolted moment plate connections. (current) 333.12, 333.15, 333.17, 333.23, 333.29, 333.31

12. Study of web stiffeners. (current)

13. Study of connection behavior in related tests. (current)


15. Panel zone stiffening by adding a web doubler plate for beam-to-column flange connections. (future)
APPENDIX 3

DESIGN PROCEDURE OF TEST 14-1
(W27x94 Beam and W14x176 Column)

1. General Considerations

a. Connections will be designed to carry the full plastic moment $M_p$ and a shear of $0.807V_p$, where $V_p$ is the shear causing yielding on the beam web.

b. Beam flanges are assumed to resist the total bending moment.

c. Beam web is assumed to carry all shear.

d. Yield stress for all material is 50 ksi. Assumed minimum ultimate is 65 ksi.

e. Allowable shear stress for A490 bearing bolts is 40 ksi.

f. Allowable shear stress for fillet welds is 21 ksi.

g. Load factor $= 1.7$ on all allowable stresses.

2. Determine Strength of Sections

$M_p$ of W27x94 $= \sigma_y \frac{Z}{x} = 50(278) = 13900 \text{ in-K (beam)}$

$Z_x = 2\{12.624(0.41)(0.205) + 2(1.313)(7.82)(3.91)\} = 162.71 \text{ in}^3 \text{ (column)}$

$M_p = \sigma_y \frac{Z_x}{x} = 50(162.71) = 8135 \text{ in-K (column)}$

$2M_p$ of column $> M_p$ of beam

$2(8135) = 16270 > 13900 \quad \text{O.K.}$
3. Beam Span

\[ V_p \text{ of W27x94} = \frac{\sigma_y}{\sqrt{3}} \frac{d_w}{t_w} = \frac{50}{\sqrt{3}} (25.416)(.490) = 359.51 \text{K} \]

\[ \frac{V_{\text{DESIGN}}}{V_p} = 0.807 \quad V_{\text{DESIGN}} = 0.807(359.51 \text{K}) = 290 \text{K} \]

\[ \text{Span} = \frac{M_p}{V_{\text{DESIGN}}} = \frac{13900}{290} = 48 \text{ in} \]

4. Design of Shear Plate and Bolts

a. Design of bolts

Pitch of bolts = 3"

End Distance = 2"

Total Plate Depth = 3"x6 + 2x2" = 22"

Holes Are 1/16" Oversize

Assume 7/8" \( \phi \) A490 Bolts

Number = \( \frac{290}{(.6013)(40)(1.7)} \) = 7.09 Use 7-7/8" \( \phi \) A490 bolts

b. Plate thickness

\[ t = \frac{V_{\text{DESIGN}}}{\sigma_y} = \frac{290}{(22)(\frac{50}{\sqrt{3}})} = 0.457" \]

Check based on 1.7 times shear allowable

\[ t = \frac{V}{(t)(1.7)(.4)(\sigma_y)} = \frac{290}{(22)(1.7)(.4)(50)} = 0.387 \text{ in} \]

Use 1/2" thick Plate

Check Plate Thickness Based on Bolt Bearing (Ref. 20)

\[ \sigma_{\text{bearing}} = \frac{290}{(7)(.50)(.875)} = 94.7 \text{ ksi} \]
\[ \frac{\sigma_b}{\sigma_u} = \frac{94.7}{65} = 1.457 \]

\[ \frac{L}{d} = 0.5 + 0.715 \frac{\sigma_b}{\sigma_u} = 0.5 + 0.715(1.457) = 1.54 \]

with \( d = \frac{7}{8} \)

\[ L = 1.54(\frac{7}{8}) = 1.35 < 2.00 \text{ in} \quad \text{OK} \]

5. Design of Flange Plates

From the comments of the WRC Task Group, it is preferred that the flange connection plates be the same thickness as the beam flange. Since we are assuming that the beam flanges alone carry the full plastic moment and that the beam flange material strain hardens to accomplish this, using a connection plate of equal thickness as the beam flange means the connection plate will also strain harden.

\[ d = 2t_f \] of W14x176 = 15-1/4 - 2-5/8 = 12-5/8" \quad T_{\text{dist.}} = 11-1/4" \]

\[ b_f \] of W27x94 = 10.0" \quad t_f = .75" \]

\[ T_{\text{FORC}E} = \frac{M_p}{d - t_f} = \frac{13900}{26.163} = 531.3 \text{K/flare} \]

Stress in connection plate = \( \frac{531.3}{(12.625)(3/4)} = 56.1 \text{ ksi} > 50 \text{ ksi} \)

Therefore plates must strain harden

Use 12-5/8x3/4" Flange Connection Plates
6. Design of Welds

a. Flange Connection Plate Welds

Length of Weld A = \( \frac{15.64 - 0.82}{2} - 1.0 = 6.41 \) in.

Try 7/16\(^\text{th}\) fillet weld both sides at "A"

Load Carried is \((4)(6.41)(.707)(21)(1.7)(.4375) = 283.1^K\)

Load to be carried by weld "B" = 331.3 - 283.1 - 248.2^K

Try 1/2\(^\text{nd}\) fillet weld both sides at "B"

Maximum length of weld "B" = 15.25 - 2(1.313) - 2.0 = 10.62

Force carried by weld "B"

\[ F = (2)(10.62)(.707)(21)(1.7)(.5) = 268.0^K > 248.2^K \quad \text{O.K.} \]

Check shear in connection plate along plane parallel to "A" welds

\( (6.41)(2)(.75)\frac{50}{\sqrt{3}} = 277.6^K > \frac{283.1}{2} = 142.6^K \quad \text{O.K.} \)

Therefore plate will not yield in shear before transferring force to welds

Use 7/16\(^\text{th}\) Fillet Weld Both Sides at "A"

Use one continuous fillet weld at "B" of length 10-1/2"--

Size of weld B = 1/2" both sides.
b. Design of Welds Connecting Connection Plates

\[ x = \frac{15.64 - .82}{2} + 0.5 + 1.5 + 1.75 = 11.16 \text{ in.} \]

Force in Weld "C" = \( \frac{11.16(290)}{25.416} = 127.33^K \)

Size of weld "C" = \( \frac{127.33}{(.707)(21)(6.41)(1.7)(2)} = 0.394" \)

Use 7/16" fillet weld on each side for weld "C"

Try 3/8" fillet weld at"D"on both sides

Length = \( \frac{290}{(.707)(21)(2)(1.7)(.375)} = 15.32 \)

Use 3/8" fillet weld both sides at"D"- 15-1/2" long Centrally Located
### TABLE 1
Comparison of Web Connections (Phase 14) with Equivalent Flange Connection From Phases 10 and 11

<table>
<thead>
<tr>
<th>Type</th>
<th>Phase 14 Web</th>
<th>Phases 10 and 11 Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange-Welded to Both Column Web and Flanges, and Web-Bolted</td>
<td>14-1</td>
<td>C1, C2, C3</td>
</tr>
<tr>
<td>Flange-Welded to Column Web Only and Web Bolted</td>
<td>14-2</td>
<td>C1, C2, C3</td>
</tr>
<tr>
<td>Fully-Bolted</td>
<td>14-3</td>
<td>C7, C8, C9</td>
</tr>
<tr>
<td>Fully Welded (Control)</td>
<td>14-4</td>
<td>C10, C11, C12</td>
</tr>
</tbody>
</table>
### TABLE 2

**TABLE OF TESTS: WEB CONNECTIONS (Phase 14)**

**Material:** ASTM A572 Grade 50  
**Beam:** W27x94  
**Column:** W14x176  
**Column Axial Force:** $P/P_y = 0.5 \text{(max)}$  
**Bolt Holes:** Round--1/16" Oversize

<table>
<thead>
<tr>
<th>Connection Designation</th>
<th>Purpose</th>
<th>Bolts</th>
<th>Beam Span*</th>
</tr>
</thead>
<tbody>
<tr>
<td>14-1 Flange-Welded to Both Web and Flanges of Column and Web-Bolted</td>
<td>Examine Connection Behavior When Beam is Flange Welded and Web Bolted to Column Through Intermediate Connection Plates</td>
<td>7/8&quot; φ A490</td>
<td>4'-8&quot;</td>
</tr>
<tr>
<td>14-2 Flange-Welded to Column Web Only and Web-Bolted</td>
<td>Examine Connection Behavior When Beam is Directly Welded and Bolted to Column Web Through Intermediate Web Connection Plate</td>
<td>7/8&quot; φ A490</td>
<td>4'-0&quot;</td>
</tr>
<tr>
<td>14-3 Fully Bolted</td>
<td>Examine How Behavior of Connection 14-1 is Changed Due to Bolting Beam Flanges as Compared to Welding</td>
<td>1&quot; φ-Flange 7/8&quot; φ-Web A490</td>
<td>5'-10&quot;</td>
</tr>
<tr>
<td>14-4 Fully-Welded</td>
<td>This Specimen Will Be Used As a Control Test</td>
<td>None</td>
<td>4'-8&quot;</td>
</tr>
</tbody>
</table>

*From Application of Load to Centerline of Column. Based on the span length above, when $M$ is attained at the critical section, the shear at the critical section will be $0.81 \frac{V}{P}$. 


Fig. 1 Moment-Rotation Curves
Section A-A

Fig. 2 Moment Pilot Tests
Fig. 3 Web Connection Assemblage
Fig. 4 Flange-Welded to Column Web and Flanges, and Web-Bolted Connection (14-1)
Fig. 5 Flange-Welded to Column Web and Web-Bolted Connection (14-2)
Material: ASTM A572 Grade 50
Electrode: E70
Bolts: ASTM A490-X
Flange = 1" Web = \( \frac{7}{8} " \)

Section A-A

Section B-B

Fig. 6 Fully Bolted Connection (14-3)
Fig. 7 Fully Welded Connection (14-4)
Fig. 8 Test Setup
M_P of Beam (W27x94) = 1160 ft-kips
M_P of Column (W14x176) = 679 ft-kips

\[ M_A = M_B \]
\[ 2M_A = M_{bm} = (M_P)_{bm} \]
\[ M_A = \frac{(M_P)_{bm}}{2} \]

\[ \frac{M_A}{(M_P)_{col}} = \frac{2}{(M_P)_{col}} = \frac{580}{679} = 0.854 \]

Fig. 9 Determination of Column Axial Load
Fig. 10 Instrumentation
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