Fracture of Moment Connections

PROPOSED BEAM-TO-COLUMN WEB CONNECTION SPECIMENS

by

George C. Driscoll
Abbas Pourbohloul

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Department of Civil Engineering

Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania 18015

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ABSTRACT

Tests of beam-to-column web moment connections under static and cyclic loading are proposed to confirm the effects of new design recommendations. Featured details of the connection designs are extended beam tension flange connection plates and varied connection plate thicknesses and fillet weld sizes required for different arrangements of stiffening and fillet weld placement.

Extended tension flange connection plates function to place groove weld for the beam flange connection outside the restrained triaxial stress zone between the heavy flanges of the column. Revised connection plate thicknesses and fillet weld sizes are based on better knowledge of force distribution in the connection. Larger plate thicknesses and fillet weld sizes are a trade-off for eliminating stiffeners and even omitting welds between the flange connection plate and column web.

The connections will be part of a beam and column subassemblage of full-size rolled structural shapes. A test setup of loading frame, reaction frame, and dual-acting hydraulic jacks is required to support and load the specimens. The system of specimen, frames, and jacks must be carefully coordinated for both strength and deformation to permit uninterrupted operation during the cyclic portions of the tests.

The test program is needed to determine the success of new connection designs in reducing the tendency toward fracture in joints with high restraint subjected to static and seismic loading.
INTRODUCTION

A program of beam-to-column web moment connection tests subjected to cyclic loading is proposed. These tests are a follow-up of findings obtained from two prior investigations on connections.

Static tests to ultimate load of full-scale beam-to-column web moment connections resulted in some fractures of tension flange connection plates [Rentschler, 1980]. An investigation of material quality and theoretical stress distributions led to the conclusion that the failures were a result of configuration of the tension flange connection plates rather than any deficiencies of materials or welds. A subsequent series of theoretical and experimental investigations resulted in recommendations for revised configurations of tension flange connection plates and for appropriate weld sizes [Pourbohloul, 1983]. Tests are needed to confirm the validity of the previous findings and to gather additional data about the performance of this type of connection under cyclic loading.

Certain conclusions of the prior investigation have led to the experiment designs proposed here:

- There was a definite difference in the post-yield capacity of specimens having the connection plate extended so the groove weld was removed from the region of the column flange tips. All details with extended connection plates showed better performance than those with the groove weld at the region of the column flange tips. The extension of the connection plate tends to alleviate triaxial stress conditions in the vicinity of butt welds.

- Rectangular extended connection plates appeared to be equal in performance to tapered extended connection plates.

- Connections with both extended connection plates and backup stiffeners were able to perform satisfactorily with connection plates the same thickness as the beam flange.

- Increases in the thickness of the connection plate can compensate for the elimination of the backup stiffeners, and even for the elimination of fillet welds between the connection plate and the column web. Design recommendations for the increases in thickness were based on the effective area to resist shear lag or on the size of the shear forces to be
transmitted to the column flanges.

- Improved knowledge of the distribution of forces between the parts of the tension flange connection means that the size of some welds can be reduced without impairing the functioning of the connection.

In view of these findings, it is recommended that a limited series of full-scale beam-to-column connections be designed by the recommendations presented and tested. They should be tested both statically and with reversed repeated loading in order to test the suitability of the connections for static and earthquake design.

Only a limited number of weak axis beam-to-column moment connections have been tested [Popov, 1969, Rentschler, 1980]. Of these, even a smaller number have been tested with a cyclic loading pattern. A representative sampling of the specimens tested exhibited some tendency toward fracture.

No specimens as large as those proposed here have been tested under cyclic loading, therefore, none have had as much opportunity to exhibit the tendency toward fracture which is ever present in thicker members. There are some effects of size which can only be determined experimentally.

No prior tests have been made to determine the effect of the extended connection plates proposed to reduce the tendency toward fracture.

No prior tests have been made to examine the effects of different thicknesses of connection plate used to provide for different attachment cases of the connection plate and stiffener.

No prior tests have verified the effect of providing different weld sizes to match the force distributions in the connection plate.
2 EXPERIMENT DESIGN

Several factors are involved in the design of an experiment to test connections. Among them are selection of member cross sections, design of the actual connection, selection of lengths of beam and column in the subassemblage containing the connection, selection of actuators or hydraulic jacks, design of a reaction frame, instrumentation, and lateral support to prevent instability of the system.

Requirements for each factor are discussed as well as estimates of ranges in which quantities may fall. Proposed values for the planned test series are listed along with available options.

Some of the parameters in the test series will be selected to maintain continuity with the earlier series of full-scale connection tests by Rentschler [1980] and simulated detail tests by Pourbohloul, et al [1983].

2.1 Beam and Column Cross-Sections

In a building, beam and column cross sections are selected as needed by the structural design. To meet the objectives of this research project, a column is needed with flanges stiff enough to help induce triaxial stresses in the connection plate near the tips of the column flanges. A beam is needed with flange thickness near one inch or more so that any tendency to fracture due to triaxial stresses will be exposed. The weak axis plastic moment capacity of the column should be half or more of the strong axis plastic moment of the beam to enable equilibrium.

These requirements could be met by a variety of 8, 10, 12, and 14-inch column shapes ranging up to the W 14 x 426 and even extending into the jumbo shapes. Beams could begin as low as some of the heavier 18 and 21-inch beam shapes up to the largest 36 W shape.

In Rentschler's tests, the shapes were a W 14 x 246 column and a W 27 x 94 beam. W 14 x 257 column shapes were used in Pourbohloul's tests.
For the proposed tests, a W 14 x 257 and a W 27 x 94 will be selected. (The W 14 x 246 shape is no longer rolled.) Larger shapes are feasible, but they might unnecessarily increase the cost or place larger demands on the loading equipment.

2.2 Connection Design

Three different connection types are proposed as shown in Fig. 1:

1. the beam flange connection plate is welded to the column flanges and column web and a fully-welded backup stiffener is provided (Fig. 1(a)).

2. the beam flange connection plate is welded to the column flanges and column web, but no stiffener is provided (Fig. 1(b)).

3. the beam flange connection plate is welded only to the column flanges (Fig. 1(c)).

The components that must be designed for each connection are: the beam flange connection plate, the beam web connection plate to the column, welds connecting the web connection plate to the column, fastening of the beam flange to its connection plate, fastening of the beam web to its connection plate, and design of backup flange stiffeners and welds, if any. Key dimensions of the specimens are listed in Table 1.

Of the connection components, the connection plate design and the welds connecting the connection plate to the column will be emphasized. These two aspects represent new design recommendations to be verified. The remaining parts of the connection design are treated adequately by previously available methods. In planning these connections, it is assumed that part of the audience for final results is interested in design by allowable stress methods and part is interested in plastic design. No matter which configuration or method of design is used, an earthquake may "elect" to load the connection into the inelastic range. Alternate designs and plans will consider some combinations.
Beam Web Connection

The beam web connection was designed with the thickness of the web plate and the welds (f) of the web plate to the column proportioned to carry the shear of the beam only. ¹ Welds (g) at the top and bottom of the web plate to the flange plates were proportioned to carry the small bending moment calculated for the beam shear acting at the plane of the web bolts with the consequent eccentricity from the column web. The design is considered to be routine, using normal design concepts.

Before choosing the 'beam shear only' concept for designing the beam web connection, an alternate was considered in which the bolts would also be calculated to carry the portion of beam bending moment existing in the portion of the beam web engaged by the web bolts. It was found that single and double rows of web bolts would have required capacities much larger than could be supplied by typical structural bolts.

By welding the beam web to the web connection plate instead of bolting it, the beam web moment could be transmitted. When bolts sized for web shear only are used, a much larger axial force due to bending is transmitted to the flange connection plates. This may explain in part why the connection with a welded beam web connection in Rentschler's test series survived without fracture, whereas those with bolted web connections sustained the fractures.

Beam Flange Connection Plate

The design of the beam flange connection plate will test new design recommendations developed from the previous work. The extension may be either rectangular or tapered in width. The detail tests showed both types of extensions to be essentially equal in performance. If the detail were used in a bridge or other fatigue situation, AWS requirements would dictate that a tapered change in width be used in the connection

¹Welds in Fig. 1 have an identifying letter in the forked tail of the welding symbol. Weld sizes are given in Table 1.
plate. The same transition is sometimes selected in building structures for esthetic reasons. The advice of the task group will be sought in selecting the extension detail for the test specimens.

The thickness of the beam flange connection plate and the sizes of the welds joining the plate to the column will be different for each of the three different connection types proposed.

In the first case (Fig. 1 (a)) with a backup stiffener and fillet welds to both the column web and flanges, a 1-1/8 in connection plate can be used because of the favorable stress distribution. Four 1/2-in fillet welds (a) will be used between the connection plate and column flanges to carry approximately 60 percent of the beam flange force. Two 7/16-in fillet welds (b) will be used between the connection plate and column web to resist the remaining 40 percent of the beam flange force. The same 7/16-in welds (c) will be used on the opposite side of the column web to connect with the backup stiffener. Finally, four 3/8-in fillet welds (d) will transmit the stiffener force to the column flanges.

The second design case has the connection plate welded to both the column flanges and web, but no backup stiffener is used (Fig. 1 (b)). A thickness of 1-3/8 in will be used for the connection plate and four 11/16-in fillet welds (a) will be used to carry approximately 80 percent of the beam flange force. Calculated weld sizes of 1/4 in would be satisfactory to carry the remaining 20 percent of the force to the column web. However, minimum welds (b) of 3/8 in will be used because of the thickness of the members involved.

The third design case (Fig. 1 (c)) has the connection plate welded only to the column flanges, the most severe case. It will require a connection plate thickness of 1-5/8 in. Four 7/8-in fillet welds (a) will be required to fasten the connection plate to the column flanges.

In the two connections of Fig. 1 (b) and (c) having thicker connection plates, transition in thickness may be accomplished by sloping the surfaces of the groove welds or by chamfering the thicker plate.
these specimens, it will be planned to chamfer the extended portions of
the connection plates before they are welded to the column.

For comparison of the new weld designs with prior practice, it
should be noted that 3/4-in fillet welds were used for all specimens in
the prior full-scale and detail tests. The weld sizes proposed here
average the same amount, but vary somewhat based on known variations in
force distribution.

Length of Column

The length of column in a test specimen may simulate the height of a
story in a prototype structure; it also must allow for clearance and
stroke for loading jacks, for height and flexibility of a reaction frame,
and for deflection of the specimen beam.

The height of column proposed for specimens will be either 10 feet
or 15 feet depending on whether the loading equipment consists of 6-inch
bore Parker-Hannefin jacks currently in use in Fritz Lab or 8-inch bore
actuators now on order. This column length will allow for the depth of
the beam, the lengths of two jacks and fittings, and the deflection of
the beam or the stroke of the jacks.

The top and bottom of the column will be welded to their base plates
with web welds (h) only, causing the ends to perform as nearly pinned
ends. (See Fig. 1 (d).) The end condition will cause the specimen to
simulate the distance between inflection points in a prototype structure,
which is approximately the floor-to-floor height.

The length of the column also participates in determining the amount
of horizontal shear that will be applied to both the top and bottom ends
of the column. The horizontal shear in turn determines design criteria
for base plates and fastenings and for the stresses and deformations of
the loading frames.
Length of Beam

The length of an end-loaded cantilever beam in a test specimen may simulate the distance from a point of maximum moment to an inflection point in a prototype structure. The length will influence the amount of shear required in the beam to develop the plastic hinge moment at the point of maximum moment. The shear, in turn, determines the capacity required for load actuators, fittings, and loading frames. The length of the beam also influences deflections caused by bending of the beam, as well as reflecting the influence of additional deflections caused by rotation of its supporting joint and translation of the supporting column.

Selection of a beam length for the specimen is further guided by the test bed anchorage pattern where convenient locations for bases of specimens, test frames, and loading frames are spaced 5 feet apart. With this in mind, beam lengths should be considered with the load point 5 ft and 10 ft from the centerline of the column.

The consequences of selecting the two beam lengths will now be examined.

With the load point located at the 5 ft station, a jack capacity exceeding 230 kips would be required. Because of the length of the jacks, this would require a column length of 15 ft. The calculated elastic component of beam deflection would be about 0.4 in. If excessive slip of the column top fixture can be prevented, the stroke capacity of the jacks should be adequate for reasonable excursions into the ductile range in the order of 4 to 5 times the elastic range.

With the load point located at the 10 ft station, the requirement for jack capacity would reduce to 115 kips. Testing could be accomplished using either the jacks with 6-in bore or those with 8-in bore. A 10-ft column height could be used with the 6-in bore jacks. The elastic component of beam deflection would be about 1.0 in. In this test setup, there would be a risk of running over the capacity of the jacks in both load and stroke. A slight excess of yield strength of the specimen
could elevate the maximum test load beyond the capacity of the jacks, leading to excessive leaking of the hydraulic system. Any slip of the column top could lead to deflections beyond the stroke of the jacks.

Apparently, a more comfortable experimental operation could be achieved by using 8-in bore jacks and a column height of 15 ft. The elastic component of deflection would be about 1.1 in. Adequate stroke of the jacks would probably be certain and the operating pressure of the hydraulic system would be at about half its rated capacity. Margin for overrun of both load and stroke would be available.

2.3 Test Setup

The test setup will consist of loading equipment, reaction frame, lateral support, and instrumentation. The loading equipment and test specimen are sketched in Fig. 2. A three-dimensional schematic diagram of the test setup is given in Fig. 3.

Actuators or Hydraulic Jacks

Load will be applied by a pair of double-acting hydraulic rams located above and below the end of the cantilever beam portion of the specimen as shown in Fig. 2. The hydraulic system will be connected so that one ram will be acting in tension at the same time the opposing ram acts in compression. The net load on the specimen will be the sum of the forces applied by the two rams. When both rams are subjected to equal cylinder pressures, the ram acting in tension will exert a lower load since the tension rod passes through the tension side of the cylinder, reducing the exposed cross-sectional area of the piston. A lower compression limit will be imposed on some hydraulic jacks having slender rods and fittings because of column buckling capacity.

Two different sets of actuators will be considered here. One will be a pair of 6-in bore Parker-Hannifin jacks presently available in Fritz Lab. They are rated at 55 kips compression or 70 kips tension with 12-in stroke. The second set is a pair of 8-in bore hydraulic cylinders presently on order from a manufacturer. These are rated at 150 kips
compression and 103 kips tension with a stroke of 24 in at 3000 psi nominal pressure.

New base plates and pin blocks of higher capacity will be needed for the new jacks. Previously fabricated connecting devices which have been tested in service are available to be used with the currently available jacks with 6-in bore.

Reaction Frame

Two main assemblages will be needed for the reaction frame, a support for the top of the specimen column and a frame to resist the vertical thrust and pull of the upper loading jack. A three-dimensional schematic diagram of the reaction frame is given in Fig. 3.

The column support frame will be required to hold the top of the specimen column in position both in its plane of loading and perpendicular to its plane of loading. The main requirement will be to resist a horizontal shear approximately two-thirds of the vertical load on the beam or about 80 kips. It is proposed to erect the reaction frame on the dynamic test bed using the available test frame hardware as much as possible. Two 23-ft long columns will be mounted 12 ft apart to support a heavy cross beam of either W 30 or W 36 size. A pair of diagonal struts will be provided to brace the bent against out-of-plane loading due to the horizontal shear exerted by the specimen column. A top base plate with special bolted cleats to prevent horizontal slip will be provided for the specimen column. (See Fig. 4(a).) In past tests with similar rigging, slip of the column top caused excessive sway of column tops.

Ten feet from the column support frame will be a jacking frame also constructed from two 23-ft long columns and a heavy cross beam. The cross beam will be mounted with its lower surface 15.5 ft above the floor to accommodate the height of two jacks, the specimen beam, and fittings. A yoke consisting of thick plates and threaded rods will be mounted over the cross beam to carry the upper jack and resist its maximum tensile load (Fig. 4(b)).
Lateral Support for Beams and Columns

Lateral support to the top and bottom ends of the specimen column will be provided by its connection to the test bed floor and the reaction frame. Lateral support to the connection end of the cantilever specimen beam will be provided by the stiffness of the specimen column. Both members selected are calculated to be stable for plastic behavior without additional bracing. The cantilever beam is predicted to be stable even if the loaded end were entirely free of support. The attachment of jacks at the loaded end will tend to have a stabilizing effect on the cantilever beam.

2.4 Instrumentation

Deflection-Measuring Instrumentation

The principal deflection of interest in each connection specimen is the vertical deflection of the loaded end of the cantilever beam relative to the point of intersection of the beam and column. Because of movement of the reaction frame and slip between the specimen column top and reaction frame, it will be necessary to measure several other deformations.

Deflections measured will be vertical at the load, horizontal at the connection, rotation of the joint, horizontal at the column top, and twist of the column (because it is fastened off-center to the reaction frame at its top).

Isolated, unloaded reference members will be mounted on the test bed a short distance from the specimen and reaction frames. Deflections between the specimen and reference members will be measured using LVDT's. Combinations of readings will be calculated to determine the actual deformations of the structure. Electrical rotation gages designed by Yarimci, et al, will be used to measure rotations at certain points on the specimen. [Lu, 1968]
Strain-Measuring Instrumentation

Electrical resistance wire strain rosettes will be applied in suitable patterns to detect the stress concentration trends and force distributions within the beam-to-column connection. Data for all electrical strain gages and displacement gages will be collected and processed using a HIMC-11/23 Modular Instruments Laboratory Computer. [Wang, 1983]

3 LOADING SEQUENCE

The objectives in planning a loading sequence for this set of experiments are both to verify suitability of the revised designs in resisting fracture under static loads and to obtain information on the behavior of this type of connections when they are subjected to seismic loads. Each test will begin with 3 cycles of load applied with a nominal maximum stress of 0.66 Fy. During these cycles, the instrumentation will be checked out and the elastic behavior of the connections will be observed.

Following the initial 3 cycles, the cyclic load program to be used in the experiments will have a monotonically-increasing amplitude which consists of a series of intervals with constant amplitude load cycles as shown in Fig. 5. The number of constant amplitude load cycles is arbitrary. Similar cyclic loadings were employed by [Popov, 1969] in his series of tests on steel connections. Of course, it is very unlikely that a connection in a building will be subjected to such orderly arranged load excursions within its lifetime; however, it is not likely either that any two seismic loadings will be identical. This fact highly complicates the loading simulation of a real connection in the laboratory and arbitrary loadings are adapted which assume a cumulative type of damage implicitly. Although the period of the load application on the connection test is much longer than that in an earthquake, investigations by [Hanson, 1966] have shown that the hysteresis curves from static and dynamic loadings are reasonably close. Also, the effect of frequency of loading on the amount of plastic strain energy and plastic strain is negligible at temperatures well below the creep range, which justifies
the quasi-static type of loading instead of the more complicated dynamic type. [Morrow, 1964]

4 SUMMARY

Tests are proposed to examine the effects of revisions in design procedure for beam-to-column web moment connections. The purpose of the tests is to both confirm new design recommendations and provide data on seismic performance of connections in a size and detail range outside the range of presently available test results.

Prior theoretical and experimental studies disclosed fracture problems in beam-to-column web moment connections. In full-scale connection tests with certain details involving thick tension flanges restrained at their connection to heavy column flanges, some fractures occurred. Studies of various tension flange connection details led to recommendations for modifications to design procedure predicted to alleviate the tendency toward fracture.

Recommendations for modifications to design procedure included using extended flange connection plates and thicker connection plates when conditions warrant them. Fillet welds joining connection plates to columns can be sized to match improved knowledge of the distribution of forces, thereby permitting some weld sizes to be reduced without impairing the performance of the connection. The recommended changes in design procedure depend on the presence or absence of backup stiffeners to the beam tension flange and of welds between connection plates and the column web.

It is proposed to fabricate 3 specimens varied to test as wide a range as possible of the findings and obtain information on performance of connections under seismic type loadings. Options in both specimen design and test setup design must be considered to complete the test plans. Some of the options relate to the strength of the connection, and some relate to spatial relationships for the test setup.

A W 27 x 94 beam and a W 14 x 257 column are selected to simulate
members of a typical prototype connection. Beam web connections for all specimens are proportioned by routine structural design concepts to transmit the beam shear only. The beam flange connection and connection plate draw upon the proposed new design concepts. The thinnest plate and smallest fillet welds are required when a backup stiffener is provided and welds to both column web and column flange are provided for connection plate and backup stiffener. When the backup stiffener is omitted, larger fillet welds and a thicker connection plate are required. When there is no backup stiffener and the welds are omitted between the column web and beam flange connection plate, the thickest connection plate and largest fillet welds are required.

It is found that the length of the specimen beam and column are related to the dimensions and load capacity of the loading equipment and also must provide adequate space for stroke of the loading equipment to allow full deflection of the specimen during cyclic loading.

The test setup consists of loading equipment, reaction frame, lateral support, and instrumentation. Options are discussed for a range of sizes of specimen and loading and reaction system. The key to the design of the experiment lies in the length of hydraulic actuators of sufficient load capacity and stroke to yield the specimen and displace it through several cycles without resetting the equipment. The recommended option uses a larger specimen column length of 15 feet and the larger of the available loading jacks, but keeps the load and displacement demands of the test well within the capability of the equipment.

A crucial part of the test setup is the reaction frame which supports the top end of the specimen column and resists the horizontal shear. Diagonal bracing is required to prevent sway of the reaction frame and a special column top fitting is proposed to prevent slip of the column with respect to the frame. A second parallel reaction frame is required at the load end of the specimen beam to accept the vertical force of the dual-acting hydraulic jacks. A loading yoke is designed to enclose the heavy cross beam of the reaction frame and carry the tension force of the vertical jacks at a higher value than any previous tests in
Fritz Laboratory using tension jacks.

Deformation of the reaction frames is calculated to be of sufficient magnitude to be a factor in planning of the tests. Losses due to slip must be avoided to prevent running out of stroke with the loading equipment. Instrumentation must be mounted on frames supported independently of the reaction frames in order to separate rigid body motions from the deformations of the specimen.

A loading sequence of monotonically-increasing series of constant amplitude load cycles will be adopted. Although this sequence does not truly simulate earthquake loadings, other investigators have shown that the hysteresis curves generated are reasonably close to those from dynamic loadings.

The experiments proposed are needed to test the suitability of new design recommendations. They will also provide needed data on cyclic behavior of connections. They will add to the currently available data in the following manner:

- Only a limited number of weak axis beam-to-column moment connections have been tested, and only part of these were tested cyclically. Some of the specimens tested did exhibit some tendency toward fracture.

- No specimens as large as those proposed here have been tested under cyclic loading. The tests give an opportunity to observe effects of size which can only be determined experimentally.

- No prior tests have been made to determine the effect of the extended connection plates and different thicknesses of connection plates in reducing the tendency toward fracture and providing for the different attachment cases of the connection plate and stiffener.

- No prior tests have verified the effect of providing different weld sizes to match the force distributions in the connection plate.
Table 1: KEY DIMENSIONS OF SPECIMENS

<table>
<thead>
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<th>Key Dimensions</th>
<th>Specimen (a)</th>
<th>Specimen (b)</th>
<th>Specimen (c)</th>
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<td>Beam Member</td>
<td>W 27 x 94</td>
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<td>Beam Web Connection</td>
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Figure 1: Possible Connections for Full-Scale Cyclic Tests
Figure 1, continued: Possible Connections for Full-Scale Cyclic Tests
Figure 2: Setup of Specimen and Jacks
Figure 3: Schematic Diagram of Reaction Frame
Figure 4: Details of Column Top Fitting and Top Jack Yoke
Figure 5: Cyclic Loading Scheme [Popov, 1969]
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