Tests of Steel Moment Connections

LYNN S. BEEDLE AND RICHARD CHRISTOPHER

THIS REPORT has been written to consolidate and discuss the results of some of the more important studies of rigid moment connections in building frames, and to suggest some possible areas of future work.

Three types of connecting media are considered: welding, riveting and bolting. In turn, three types of connections receive primary attention: the corner connection, the beam-to-column connection, and the beam splice. Primary attention is focused upon the moment capacity and deformation capacity of these connections. The criterion used in this report is the ability of the connection to undergo inelastic strain resulting in joint rotation many times that associated with initial yielding, while providing a predictable resisting moment.

The tests reported herein were conducted at the following schools: Cambridge University, Cornell University and Lehigh University. Investigations of significance conducted at other schools are also briefly considered.

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 large plastic rotation capacities were observed. There were no premature failures except those which could have been predicted and prevented.

INTRODUCTION

With the increasing use of plastic analysis and design, additional emphasis is being placed on the design of all types of moment resisting connections. Simple plastic theory assumes that a connection will be capable of developing the full plastic moment of one or more adjoining members. It also assumes that the connection will be strong enough so that these adjoining members will have a rotation capacity sufficient to allow the formation of the prescribed mechanism without premature failure.

This report is primarily a review of test results of steel moment connections and will include a discussion of welded, riveted and bolted joints for buildings. Bridge connections will not be considered here because of the nature of the applied loading which includes impact, repeated loading and moving loads.

Principal attention also will be confined to "rigid" moment connections—those that must develop the full plastic moment of the connecting member as shown by the upper curve in Fig. 1. In "simple" and "semi-rigid" connections, also shown in this figure, primary attention focuses on the stiffness of the connection (the elastic slope of the moment-rotation curves in Fig. 1). Essentially what is desired in a rigid connection is the ability to develop the plastic moment. The stiffness of a rigid connection is not of primary interest, since the rigid type always introduces enough material to limit the "elastic" unit rotation to the same order of magnitude as that of the member being connected. Assuming that the columns provide complete fixity, the resulting rigidity gives an elastic distribution of moment (shown with the dashed line at the right in sketch (**a**)) of $wL^2/12$ at the ends and $wL^2/24$ at the center. The inelastic rotation that occurs after M_p is reached at the ends of the beam permits redistribution of moment so that subsequently the plastic moment is reached at the center. Under the action of gravity plus wind loading, shown in diagram (**b**) of Fig. 1, the "hinge action" permits full participation of the beam and the connections in resisting lateral load.

The welded connection is a familiar moment resisting joint, and has been used extensively. Figure 2 shows a welded beam-to-column connection of the "top-plate" type. There have been many investigations of welded connections (primarily under the auspices of the Welding Research Council), and a number of the more important studies will be discussed.

The riveted connection is by far the oldest of the three types. However, only brief attention will be given to this type for two reasons:

Lynn S. Beedle is Research Professor of Civil Engineering and Director of Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa. and is a Professional Member of AISC.

Richard Christopher is Research Assistant, Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University, Bethlehem, Pa.

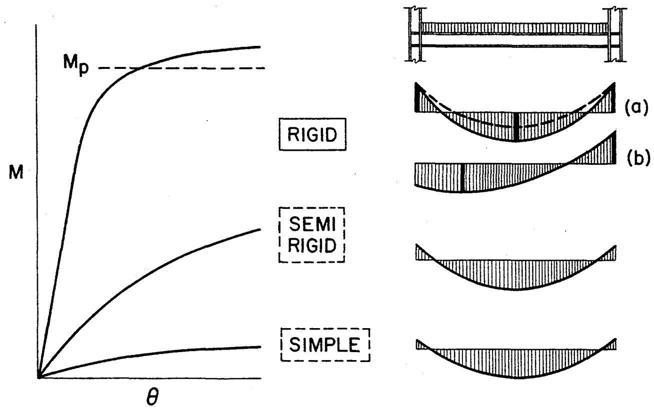

Fig. 1. Types of connections

Fig. 2. Welded moment connection

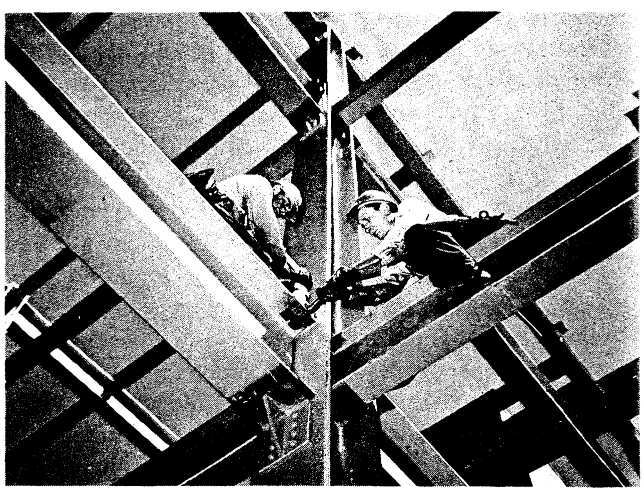

Fig. 3. Bolted beam-to-column connection

1. Most tests of riveted moment connections were performed many years ago and were not carried sufficiently past working loads to indicate their behavior near the plastic moment.
2. Bolts are replacing rivets in the design of many moment connections.

The bolted connection has been steadily growing in prominence since the first specification for high-strength bolts was introduced in 1949. The use of high-strength bolts represents a significant contribution, and was made possible through extensive structural research under the guidance of the Research Council on Riveted and Bolted Structural Joints. Figure 3 shows a bolted interior beam-to-column connection.

TYPES OF CONNECTIONS

There are many types of steel moment connections, only a few of which can be discussed in this paper. Figure 4 shows different classifications of rigid connections in building frames. They include:

1. *Splices*, which could be splices in beams, at the peak of a roof, or in columns
2. *Corner Connections*, which may be of the straight or of the haunched type
3. *Miscellaneous Connections*, which cover attachments of purlins and girts to the main frame
4. *Beam-to-Girder Connections*, which include the attachment of beams to girders at right angles to the plane of the frame
5. *Beam-to-Column Connections*, which could be of the "side", "top", or "interior" type as indicated in the figure
6. *Column Bases*

Principal attention in this paper will be given to beam-to-column connections, to splices (especially beam splices), and to corner connections.

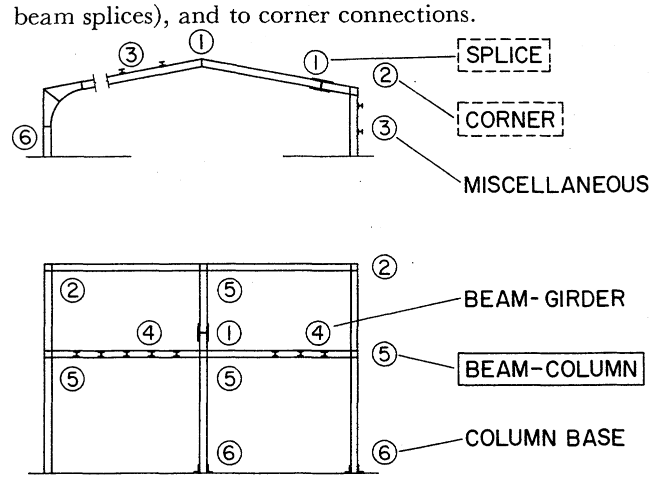

Fig. 4. Types of rigid connections in building frames

WELDED CONNECTIONS

In examining the behavior of welded joints, straight corner connections (those without a haunch) are of initial interest. In Fig. 5, load is plotted against deflection for a rigid straight corner connection fabricated from 30WF108 shapes. It was tested as part of a Lehigh University study. The dotted line shows the theoretical behavior,[1] and the solid line through the points shows the results of the test and demonstrates the ability of such connections to develop the plastic moment, M_p, and to rotate through a considerable angle. This curve is characteristic of welded corner connections designed to develop M_p:

1. The initial portion of the curve closely follows the predicted elastic slope.
2. Strain-hardening produces some increase in strength above the load corresponding to M_p.
3. After considerable deformation the curve "unloads" due to local and lateral buckling, and eventually falls below M_p.
4. In this case the deformation capacity was about eight times the "elastic limit" deformation (the deformation at which the member theoretically first reaches M_p).

Connections such as the one shown in Fig. 5 are most easily proportioned using elementary plastic analysis.[2] For example, as shown in Fig. 6, if the top flange is isolated it is seen that the reaction to the force T is made up of the resistance of the web, T_w and that of the stiffener, T_s. Now, the maximum possible values of these resisting forces exist when both the web and the stiffener are completely yielded. Therefore, the area of stiffener required to resist the plastic moment can be obtained simply by solving the equilibrium between the force T and the sum, T_w plus T_s, or

$$\frac{\sigma_y Z}{d} = \frac{\sigma_y}{\sqrt{3}} wd + \frac{\sigma_y A_s}{2}$$

where Z is the plastic modulus, w is the web thickness, and d is the beam depth. The only unknown in this equation is the area of the stiffener, A_s.

Figure 7 shows the results of a series of welded haunched connections tested at Lehigh.[3] The minimum deformation capacity was equal to about six times the elastic limit deformation. The only deficient connection was Test No. 44, and its performance could have been predicted before the test since bracing was omitted at the reentrant (compression) corner.

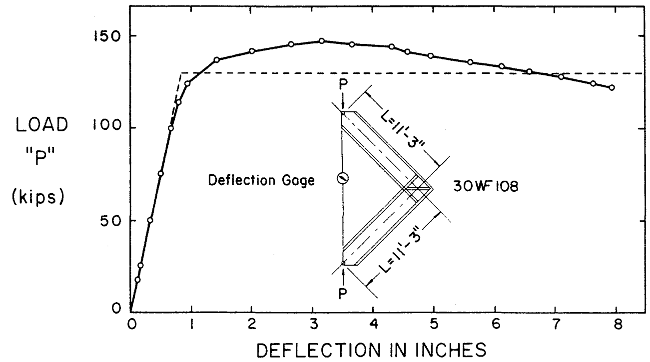

Fig. 5. Welded corner connection

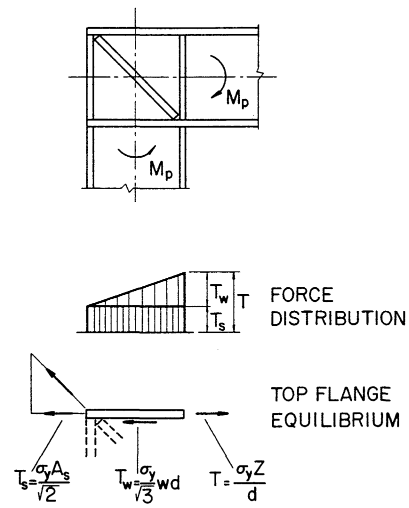

Fig. 6. Plastic analysis of a corner connection

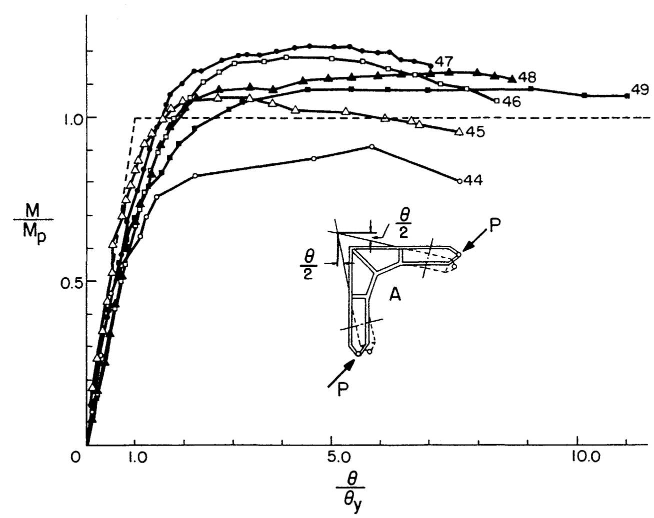

Fig. 7. Behavior of haunched corner connections

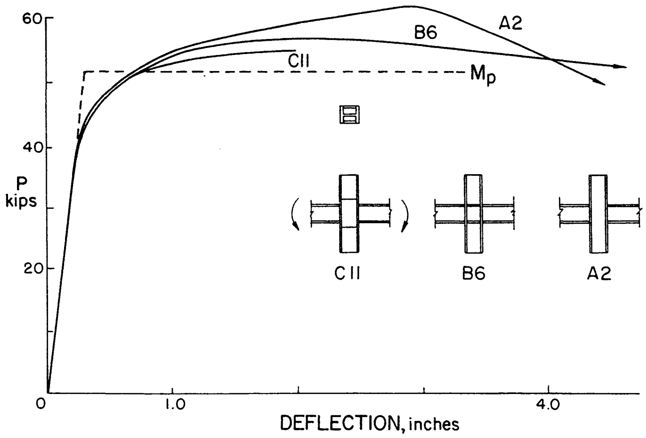

Fig. 8. Behavior of welded beam-to-column connections

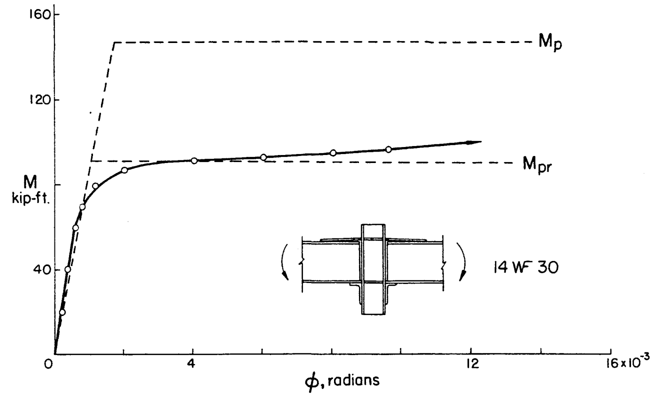

Fig. 11. Welded connection designed for a reduced moment

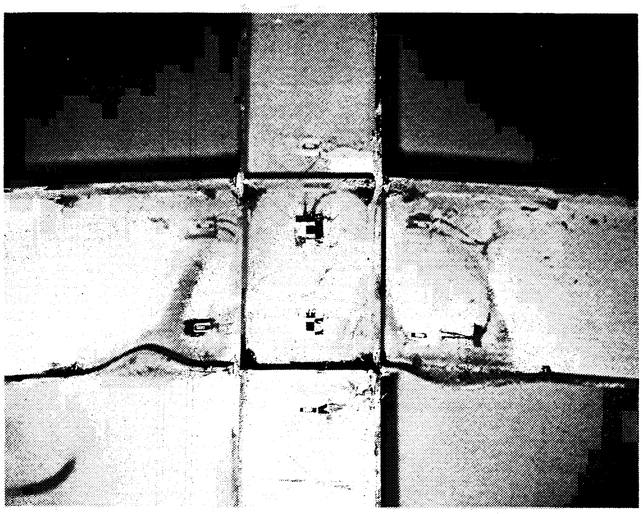

Fig. 9. Beam-to-column connection after testing

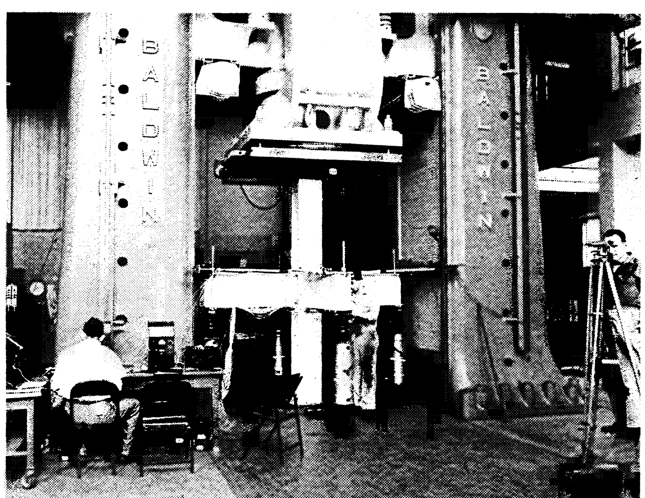

Fig. 10. Testing a four-way connection

In tall buildings the typical connection is the interior beam-to-column connection. Three different designs are shown in Fig. 8. Of the three designs, the one on the right is fabricated by simply welding the beams to the columns. Alternately, as shown in the center, horizontal stiffeners could be added to transmit the flange force. As shown at the left, a vertical type stiffener is another possibility. The Lehigh test results (the three solid curves) show that plastic moments can be developed and hinges formed. In fact, the deformation capacity of the beams is very great indeed.[4]

Figure 9 is a photograph of one of the two-way tests in the previous series. The connection design obviously forced failure in the beam, local buckling of the beam web and flange following plastification.

In Fig. 10, a connection similar to that in the previous figure is shown being tested in the five-million-pound hydraulic testing machine at Lehigh. It is a "four-way" test, beams also being loaded at right angles to the "main plane" of the frame. (The tests shown in the previous figure were "two-way" tests.)

As a final illustration of the performance of rigid welded connections, Fig. 11 shows the behavior of a "top plate" type of connection similar to that shown in Fig. 2. Such joints can be designed either to develop the full moment capacity or they may be designed to restrain the beam by some lesser amount. This particular connection was designed to resist a moment that was about 60 percent of the full plastic moment of the beams joined.[5] As seen, the connection behaved as predicted:

1. It provided a stiffness approximately equal to that of a "uniform beam."
2. It attained a predictable (although reduced) plastic moment.
3. At the computed value, large inelastic deformations occurred—the "hinge action" essential to proper response to loading.

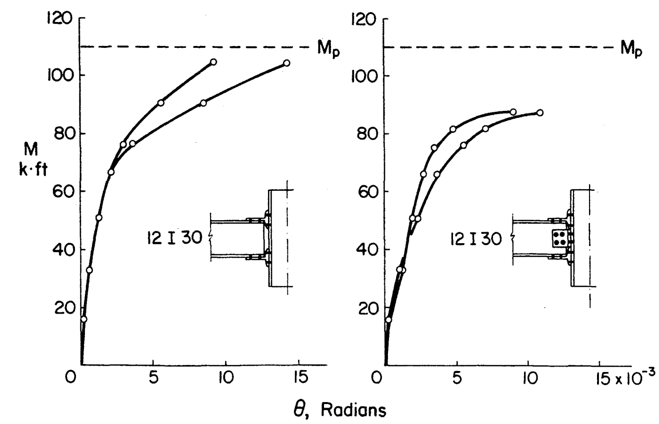

Fig. 12. Early tests of riveted connections

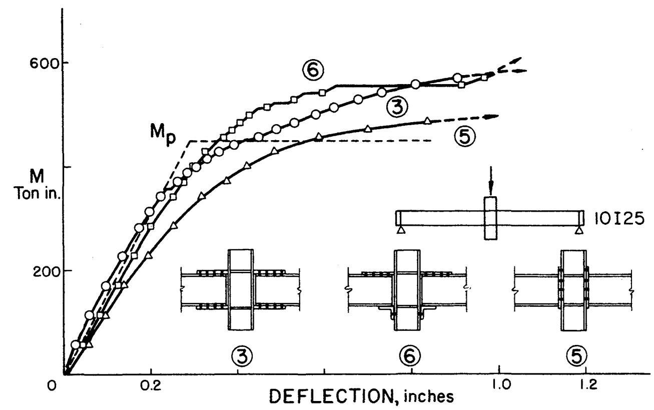

Fig. 13. Behavior of bolted beam-to-column connections

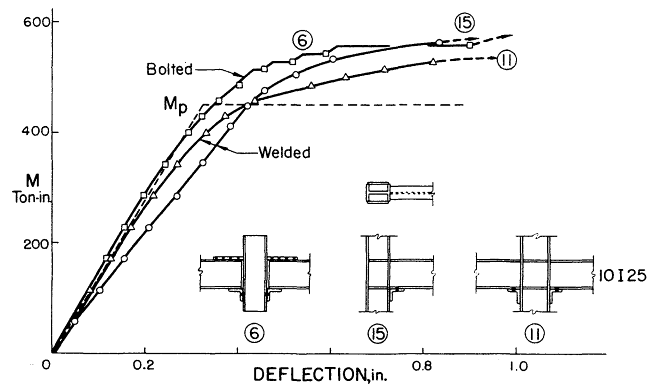

Fig. 14. Comparison of bolted and welded beam-to-column connections

RIVETED CONNECTIONS

As mentioned earlier, little attention is being given in this report to riveted connections. Rigid connections were tested in the 1930's at the College of the City of New York,[6] but behavior in the elastic region was being studied and therefore deformations were not recorded in the plastic region.

Figure 12 shows results of a series of British tests performed at about the same time.[7] An estimated value of the plastic moment of the beam is indicated. These particular tests were selected because of the relatively large deformations recorded, but the tests were simply stopped when the elastic and initial inelastic regions has been passed.

BOLTED CONNECTIONS

Beam-to-Column Connections (*Cambridge University, Series 1*)—The most recent developments in steel moment connections are probably in the area of joints using high-strength bolts. Some of the first of these were performed at Cambridge University in 1957–1958.[8] Figure 13 shows, in the lower portion, three different schemes for bolting interior beam-to-column connections. (Actually they are combinations of bolting and welding.) In the first (Test 3), the beam is bolted to two plates which are welded to the column flanges. In Test 6 the beam rests on an angle clip and is bolted to a welded plate at the top flange. In Test 5, end plates were welded to the beams and these were bolted to the column flanges, the bolts working in tension. ¾-inch bolts were used and the beams were 10 I 25 shapes, loaded as indicated in the figure.

Test 3 was designed with a slip coefficient of 0.45 to slip at the plastic moment. Actually from the results of the test the slip coefficient was 0.315, this lower-than-expected value being attributed to the fact that the surfaces were not in complete contact.

On Test 6, more care was taken to see that the plates were in contact, and thus the moment at first slip (as is evident in Fig. 13) was greater than M_p. In Test 5, after considerable deformation, the test was stopped due to a failure that occurred in a defective plate. (There were no bolt failures.) In Tests 3 and 6 the deformation continued far beyond that shown in the figure; the gages were removed at the end of the solid portion of the curves.

In Fig. 14 the data from Test 6 of Fig. 13 is repeated and is compared with the test results for two welded connections.[9] In general it can be seen that the behavior of the bolted connection is basically the same as the welded ones insofar as the ability to develop plastic moments is concerned. One difference is the slight additional stiffness in the "elastic" range for the bolted joint because of the added plate material in the lap area. The high clamping force of the bolts develops enough friction so that the whole assembly tends to act together. In both the bolted and welded joints of this series the final performance is not limited by the connection material but instead depends upon the ability of the beam to rotate plastically.

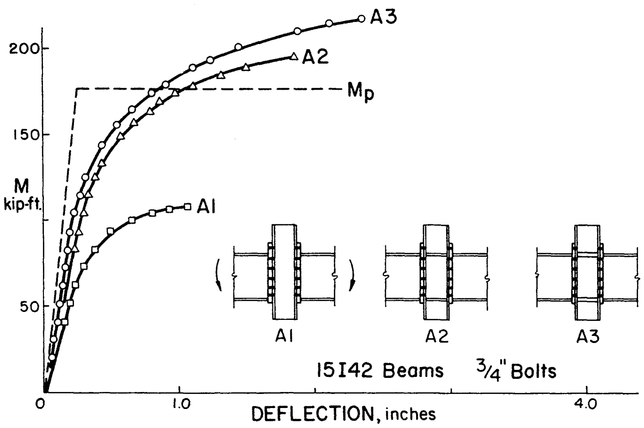

Fig. 15. Behavior of end plate connections

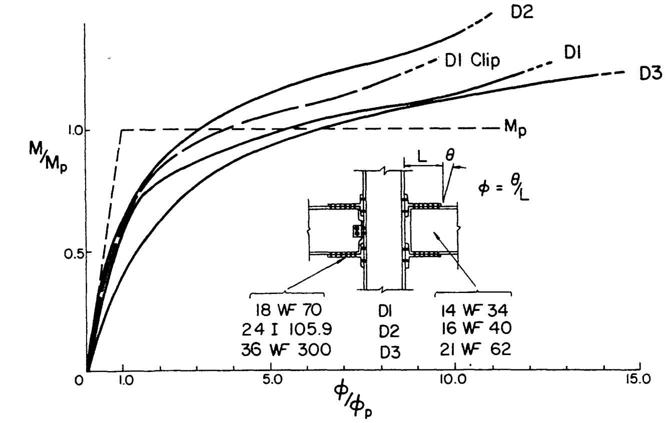

Fig. 17. Bolted T-stub connections

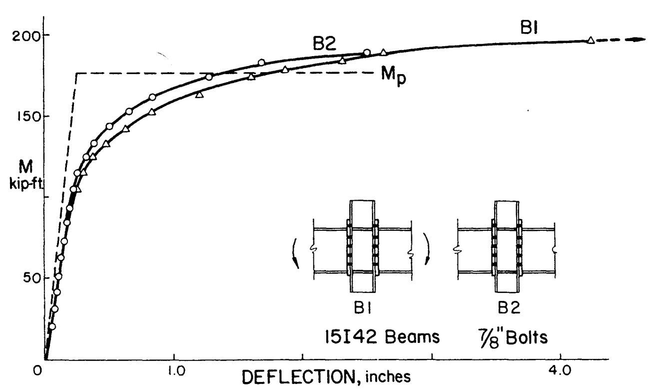

Fig. 16. Behavior of end plate connections

Beam-to-Column Connections (*Cambridge University, Series 2*)—The next series of tests conducted at Cambridge University, England, was confined to the type that offers the greatest simplicity and economy from the standpoint that it requires the fewest number of bolts.[10] The three specimens in "Group A" are shown in the lower part of Fig. 15. In such connections the bolts transmit some shear, but are primarily loaded in tension. The specimens were loaded as in the previous series (Fig. 13), with a concentrated load applied to the column stub; the reactions were carried at the beam ends.

The first group used ¾-in. bolts, recognized at the outset as being undersized. The basis for the design was that all bolts would be at "yield" (selected as the proof load) when the beam reached M_p. Thus, a redistribution of forces in the bolts was assumed.

The results of the tests confirm what would have been expected:

1. Test A1 had no stiffener in the column web; since its proportions were such that it was inadequate to resist the applied forces, the web buckled.
2. In Test A2 a stiffener was used with a heavy end plate. As expected the outer bolt failed; the joint was underdesigned.
3. Test A3 had a thinner end plate which permitted some deformation which relieved the loading on the end fastener. There was somewhat better overall performance.

The connections in the next group of tests (Fig. 16) were similar to Group A but were designed with ⅞-in. high-strength bolts. Specimen B2 was "underdesigned" since calculations showed that the required thickness for the end plate was 1.1 in. and the specimen was designed with a ¾-in. plate. The result was a weld fracture at a deflection about ten times the value at the outset of idealized plastic behavior. Test B1 was completely adequate, the final failure being in the beam. The end plate thickness was 1 in.

Tests similar to those just reviewed were conducted at Georgia Institute of Technology. In view of their ready availability[11] they are not presented again here.

Beam-to-Column Connections (*Cornell University*)—Quite a complete program is just being finished at Cornell University. A portion of it is described in References 12 and 13. The T-stub beam-to-column connections that were tested in that program are shown in Fig. 17. (In addition the program included an end plate assembly and a number of beam splices which will be described later.) As shown in this figure the program included a considerable variation in size of T-stub (18-, 24-, and 36-in. members) and also a variation in the size of beam joined (14WF34, 16WF40 and 21WF62). The program included the use of both ⅞ and 1⅛-in. high-strength bolts. Some of the connections had shear clips and some did not.

The results are shown in the nondimensional plot of Fig. 17, the idealized behavior being shown by the dashed line. The curvature is computed as the total rotation angle measured, divided by the gage length.

The theoretical elastic slope is the idealized flexibility of the gross section beam.

Each assembly involved two connections. Three of the curves are for the connections without clips, and one shows the results with a clip. There is somewhat more flexibility without a clip than with one, but the basic behavior is the same. The essential feature of these results is that the connections were able to carry the plastic moment of the beam and to allow the beam to rotate inelastically through a very large angle.

Figure 18 shows a photograph of Test D3, the connection which employed the largest T-stub (36WF300). The beam was a 21WF62 and the column was a 14WF150 shape. The fasteners were $1\frac{1}{8}$-in. A325 high-strength bolts. Most of the plastic deformation is in the beams but there is also a small amount to be seen in the column web. Final failure was by local buckling of the beam compression flange. The moment at the first line of bolts in the beam was 110 percent of M_p, an increase that is primarily because of strain hardening. Correspondingly, at the maximum load the moment at the column line was 50 percent greater than M_p.

Figure 19 shows the effective performance of the "end-plate" type of connection tested in the Cornell series. It was used to join 16WF36 beams to the column with $\frac{7}{8}$-in. high-strength bolts, tightened to the proof load. The photograph of the connection (Fig. 20) indicates clearly the ability to develop the plastic moment in the beam and the final compression-flange buckling. Yielding visible in the column web resulting from the compressive and tensile thrusts introduced by the beam flanges reflects the "balance" that was achieved in the design of this joint. The nature of the "yield lines" in the beams also shows that considerable shear was present. However, this did not prevent the connection from developing the full plastic strength of the beams joined.

Beam Splices (*Cambridge Series*)—The final group of connections which will be examined briefly are bolted splices in beams. Figure 21 shows the test setup used at Cambridge in 1958 and the two splices which were tested.[8] The first was designed as a riveted full-moment splice, bolts being substituted for rivets on a one-to-one basis. Test 2 (quite arbitrarily) was designed using just half as many bolts. The results are shown on a moment versus deflection basis, with the theoretical deflection of the uniform beam as indicated. Also the plastic moment of the beam and the reduced plastic moment allowing for bolt holes, M_{pr}, are shown.

Test No. 1 exceeded both the reduced plastic moment and M_p, no slip being observed in the test. After the gages were removed deformation was continued until the beams buckled. The stiffening effect of the splice plates,

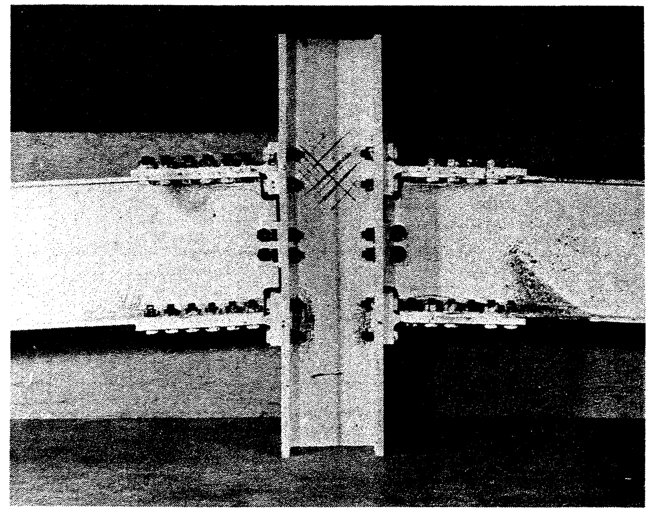

Fig. 18. T-stub connection after testing

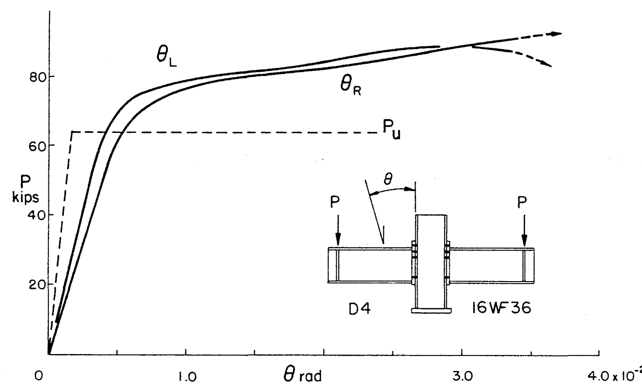

Fig. 19. End plate type of beam-to-column connection

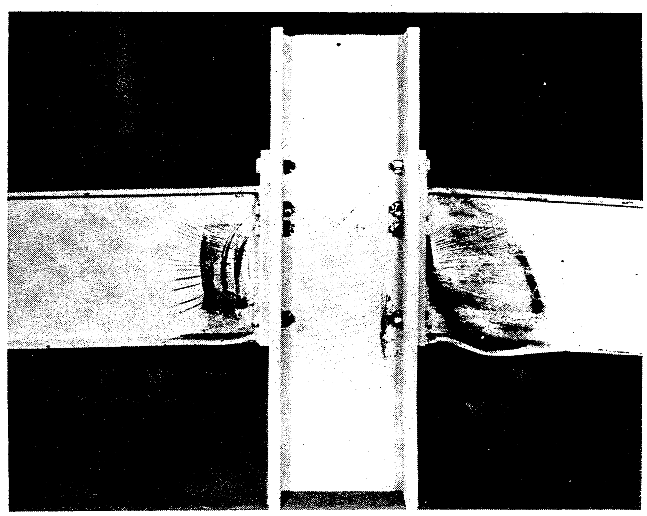

Fig. 20. End plate connection after testing

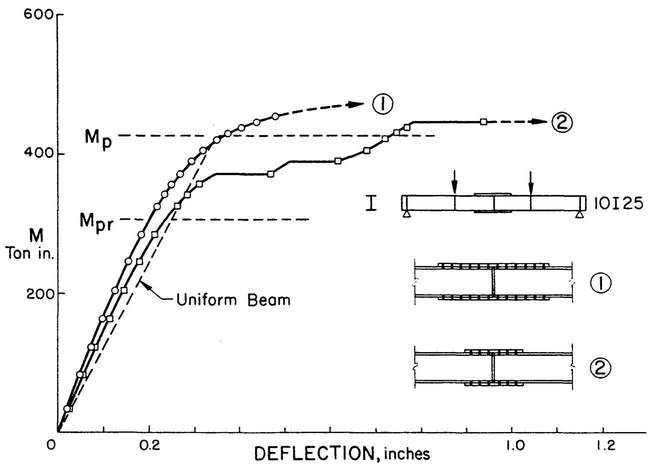

Fig. 21. Behavior of bolted lap splices in beams

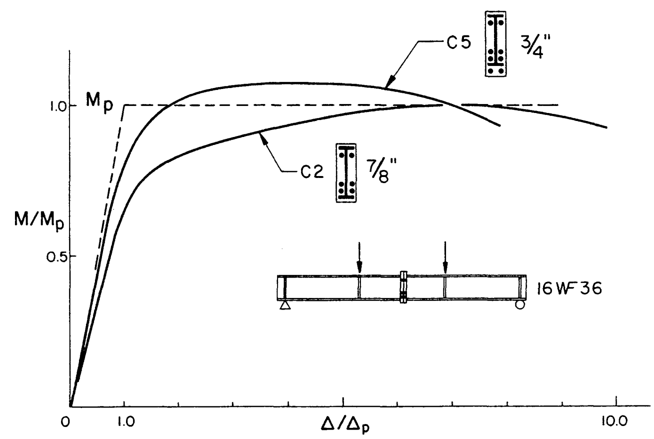

Fig. 24. Beam splices–end plate type

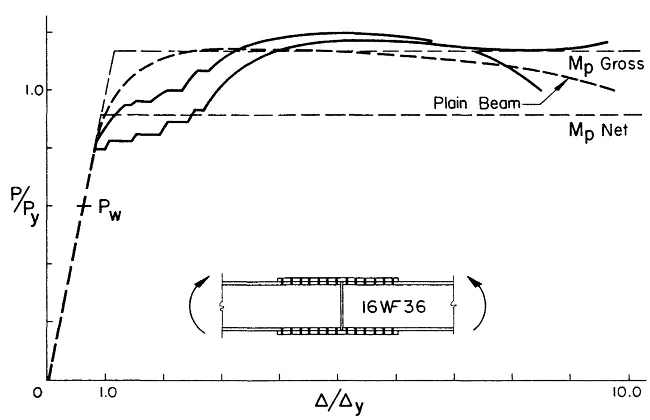

Fig. 22. Behavior of bolted lap splices in beams

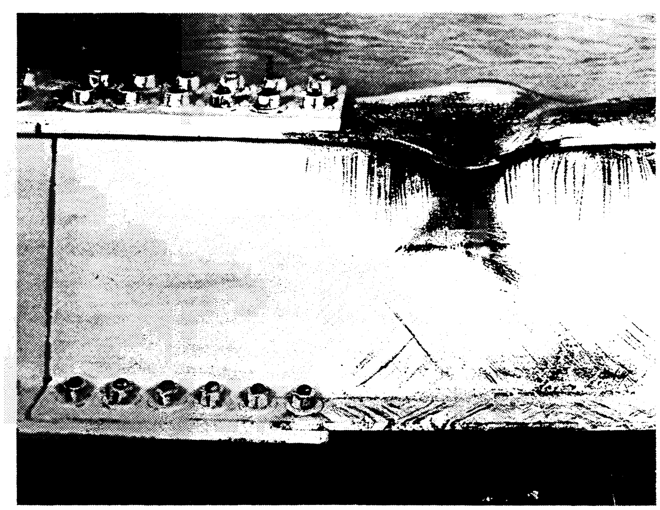

Fig. 23. Failure of bolted lap splice

noted in the connection with earlier tests, is evident. In Test No. 2 slip occurred at a moment value between M_{pr} and M_p, the slip coefficient for these particular tests being 0.49. Of special interest is the fact that strength in excess of M_p is developed by these joints even though, theoretically, 25 percent of the plastic strength is removed by the bolt holes. This is the result of the recognized effects of strain-hardening and stress-concentrations.

Beam Splices (*Cornell Series*)—Three identical beam splices with lap connections were tested in the Cornell University program previously described. 16WF36 beams were used, the 7/8-in. bolts being loaded in shear. Figure 22 shows the results on a nondimensional basis, and in comparison with the plastic moment of the gross section and of the net section of the beam. The heavy dashed curve gives the results from the test of a plain unspliced beam. The bolts were designed on the basis of 22 ksi as permitted in bearing type connections.

None of the connections slipped below the working load P_w (shown as 0.6 P_y), and all of the connections developed the full plastic moment of the gross cross section and showed satisfactory deformation characteristics. The photograph in Fig. 23 shows that the splices obviously were able to develop the full plastic moment, forcing local buckling to take place in the beam compression flange.

The final tests in the Cornell series to be mentioned in this report are shown in Fig. 24, consisting of a beam splice using plates welded to the beams and loaded in pure moment. Test C5, with eight 3/4-in. A325 bolts performed better than C2 with six 7/8-in. bolts, indicating again the advantage of extending the plate beyond the tension flange of the beam. Figure 25 is a photograph of beams C1 and C5. Test C5 is shown at the bottom of the photograph. Test C1 is similar in appearance to C2.

DISCUSSION

Figure 26 summarizes the present situation with respect to rigid moment connections. There is practically no information available on the ability of riveted connections to develop the plastic moment. This is not because of any fundamental difficulties, but is simply because no tests have been conducted. With respect to welded connections, the major problems appear to be solved, although special items will require attention.

Many of the basic problems involved in bolted connections are solved, but some further research is needed. It is of interest that no tests have yet come to light on column base plate assemblies bolted to the foundation. Among other areas of needed research are:

1. Beam-to-column connections with unequal beam moments
2. Limiting conditions under which bolt holes may be neglected when computing M_p
3. Response of joints to reversed loading of the type which would be encountered in strong earthquakes

Nevertheless, information available to date makes it abundantly clear that if a rotation capacity (or ductility factor) of 8 to 10 is not realized at a steel moment connection, it is because some detail has been underdesigned.

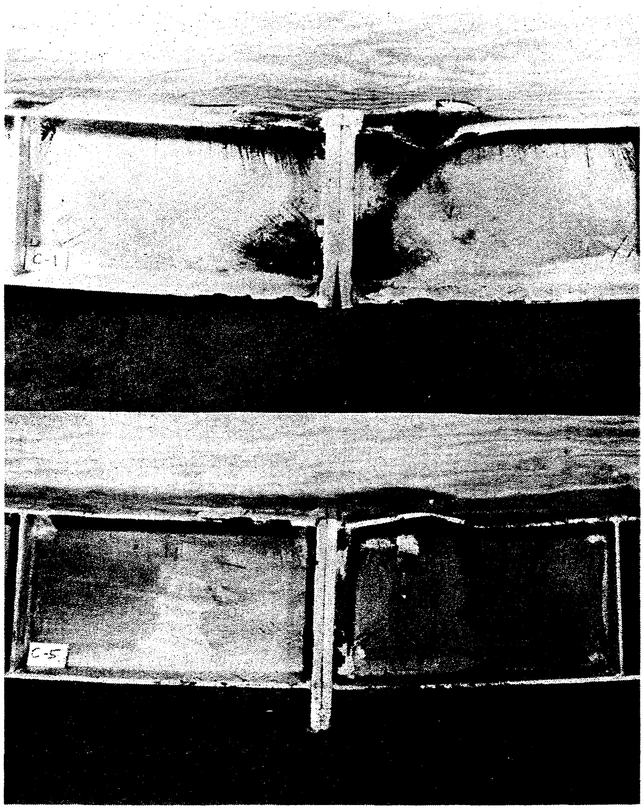

Fig. 25. Comparison of end plate designs

	TYPE		TESTS	ANALYSIS	DESIGN GUIDES
Riveted	Splice, Corner, Bm.-Col.		O	O	O
Welded	Corner	Straight	X	X	X
		Haunched	X	X	X
		Bracket	X	X	X
	Beam-Column		X	X	X
	Splice		X	X	X
Bolted	Corner		O	O	O
	Beam-Column		X	X	X
	Splice		X	X	X
	Column Base		O	X	X

Applicable Results: X Yes; O No

Fig. 26. Present status of rigid connection research

SUMMARY

This review of tests on rigid steel moment connections of the type used in building construction may be summarized as follows:

1. For all properly designed and detailed welded and bolted moment connections, the plastic moment of the adjoining member was reached and large deformation capacities were observed. There were no premature failures except those which could have been predicted and prevented.
2. Welded connections can be designed for a reduced moment without sacrificing elastic rigidity or plastic deformation capacity.
3. Very little information is available concerning riveted moment connections. Most early work did not give attention to the plastic region.
4. The behavior of bolted connections is essentially the same as that of welded connections with respect to moment and deformation capacities. The lap bolted connection has a somewhat greater initial stiffness due to the added effective plate area.
5. In lap type bolted connections, the actual buckling failure is forced to occur outside the connection in the member itself, even though a higher moment may be present within the connection.
6. The reduced area caused by the bolt holes in lap type connections had no adverse effect for any of the tests examined.
7. There are a number of areas where research is needed, some of which have been noted in the discussion. One of the most important of these is the response of steel moment connections to reversed loading of a limited number of cycles.

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