C 1007

~~C 1007~~
194.2

824
List

Tests of Miscellaneous Welded Building Connections

By Bruce Johnston and Gordon R. Deits
Lehigh University, Bethlehem, Pa.

Presented before the 22nd Annual Meeting
of the
American Welding Society
Philadelphia—October, 1941
and is a
Contribution to the Welded Research Committee

Preprinted November 1941 from a forthcoming issue of THE WELDING JOURNAL

Tests of Miscellaneous Welded Building Connections*

By Bruce Johnston† and Gordon R. Deits‡

Introduction

THE primary purpose of this investigation was to check the adequacy of certain types of welded building connections now being used in actual practice. The laboratory specimens were designed similar to details on actual design sheets. Twenty-one beam-to-column connections were tested in all and there were five different types of connections. The investigation was not expected to furnish complete information about any one type, but some of the tests may furnish a limited amount of information and point the way to desirable future research programs.

The assumptions usually made in current design practice were followed. It was intended to test designs which might prove either good or poor, therefore the inclusion of a connection type in this program does not necessarily indicate it to be good design.

This investigation is one of a series which has been sponsored at Lehigh University by the AMERICAN WELDING SOCIETY. The program forms a part of the activity of Committee G on Structural Steel, of the Industrial Division of the Welding Research Committee of the Engineering Foundation. A special subcommittee organized to work on the Lehigh test program consisted of Mr. F. H. Dill, representing Mr. C. F. Goodrich, of the American Bridge Company, Mr. Heath Lawson, representing Mr. Jonathan Jones, of the Bethlehem Steel Company, Mr. Carl Kreidler, of the Lehigh Structural Steel Company, and the authors. General details were outlined at a meeting of the subcommittee on July 8, 1940. These details were criticized by the main committee, after which shop details were prepared. The shop details were revised in accord with the subcommittee's suggestions and the revised details were checked by Mr. Lawson. Fourteen of the specimens were fabricated by the Bethlehem Steel Company under simulated field positioning for welds usually field welded and seven of the specimens were fabricated in the Fritz Laboratory by a qualified welder of the same company under similar conditions. Acknowledgment is made to Mr. Howard Godfrey, formerly Engineer of Tests at the Fritz Laboratory, and to Professor Hale Sutherland, Head of the Department of Civil Engineering and Director of the Fritz Engineering Laboratory for their interest and assistance in carrying out the program. Acknowledgment is also due to Mr. Leon S. Moisseiff and Mr. William Spraragen, Chairman and Secretary, respectively, of Committee G.

Connection Design Requirements and Test Procedure

The design requirements for a beam-to-column or beam-to-girder connection in a steel building frame will include the following:

1. The connection must have vertical shear strength sufficient to safely carry the vertical beam end reaction produced by the vertical loads.
2. If lateral forces such as wind loads are to be considered, the connection must have moment strength as well as shear strength.
3. If the connection is designed only for vertical loads it must either be (*a*) flexible, or non-moment resisting, or (*b*) moment resisting with moment strength inversely proportionate to its flexibility.
4. The connection should be economical in design, easily accessible for welding and convenient for erection procedures.

The connections tested in this investigation are designed for vertical loads alone, but, in view of condition (3) above, it is necessary to investigate their moment-rotation characteristics.

Building connections may be classified under three different headings with respect to their moment-rotation characteristics.

1. *Rigid Connections* are those in which the rotation between the end of the beam and the column is reduced to a minimum by the use of heavy connection details. The behavior and bending moment diagrams for a beam with such connections is illustrated in Fig. 1 (*a*). In such a connection the bending moments may be calculated by analysis of the structure as a continuous frame. Rigid welded building connections were investigated by Wilbur M. Wilson and reported on in THE WELDING JOURNAL.*[,1] Rigid connections are usually uneconomical in ordinary building construction except in cases where lateral forces must be resisted and continuity is required. Another application of the rigid connection is in the modern industrial or auditorium building.
2. *Flexible Connections* are those which are capable of carrying the required end reaction, but which allow relatively free rotation between the end of the beam and the column, as illustrated in Fig. 1 (*c*). A flexible connection conforms to the common assumption of simple supports in which case the beams are designed for full simple beam moment. A program of tests of flexible seat and top angle connections was recently reported in in THE WELDING JOURNAL.[2] Although no connection is completely flexible, satisfactory connections may be designed to develop less than ten per cent of the end moment in a fully rigid connection.
3. *Semi-Rigid Connections* are those which transmit bending moment with some degree of rotation between the ends of the beam and the columns, thereby providing a degree of restraint somewhere between complete rigidity and complete flexibility. The behavior and bending moment diagram for a beam with such connections is illustrated in Fig. 1 (*b*). Such connections afford possibilities of balanced economy in beam and connection design.[3] Investigations of welded semi-rigid seat and top angle connections have been carried out at

---

* Paper presented at the Annual Meeting, A. W. S., Philadelphia, Pa., Oct. 20 to 24, 1941. Contribution to Industrial Research Division, Welding Research Committee.

† Associate Director, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

‡ Formerly American Welding Society Research Fellow, Lehigh University, Bethlehem, Pa.

* These numbers refer to the references at the end of the report.

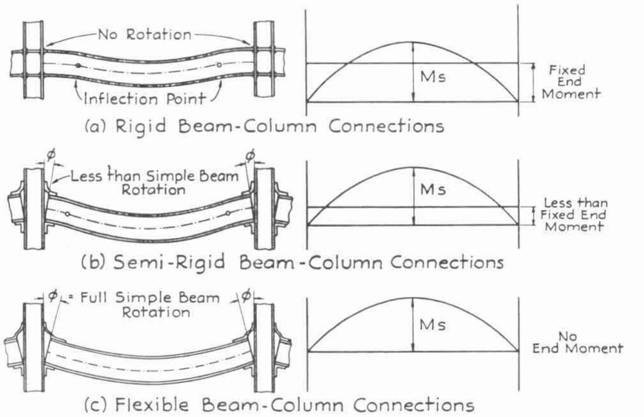


Fig. 1—Three Classifications of Welded Beam to Column Connections

Lehigh University.[4, 5] Such connections are difficult to design because they must provide a balance between strength and flexibility with regard to moment and rotation, respectively.

Many connections designed only for vertical reaction are inherently semi-rigid, thereby developing end moments which may not have been considered in the design. Such connections may not be strong enough to carry the end moments imposed by the normal tendency for the ends of the beam to rotate when the beam is loaded, in which case the design is faulty. In the design of welded connections it cannot be too strongly emphasized that end restraint and flexibility, as well as shear or reaction, must be given due consideration. The mere assumption of simple-beam end conditions is not adequate.

A typical set-up to test the moment and rotation requirements of a connection is shown in Fig. 2, which is similar to that used in previous tests.[2, 4] The connection assemblage, consisting of two beam stubs attached to a column stub, is supported in an inverted position. The supports supply shear and moment at the connection approximately equivalent to the shear and moment at the end of a building beam framed at each end of a column. After a preliminary determination of the moment-rotation characteristics of the connection at low loads the moment arm length, *a* (Fig. 2), may be adjusted so that the ratio between moment and shear will simulate any desired beam length and load condition. This ratio of moment to shear will remain the same as in the simulated structure as long as there is a linear relation between moment and rotation.

Figure 2 shows the rotation bars used to measure relative rotation between beam and column by use of a 20-in. level bar of the same type used and described in previous investigations.[2, 5] The level bar was sensitive to changes in angle of $^1/_{20,000}$th of a radian or to ±10 seconds. Figure 2 also shows $^1/_{1000}$-in. dial gages in position to measure horizontal movement, thereby locating the center of rotation as well as checking the rotation bar results. Dial gages are also shown in position to measure vertical movement. The test set-up was modified in various tests to suit the particular conditions, such a modification being shown in Fig. 3, which is a photograph of W19 during test with the level bar, rotation bars, and dials all in position. The lateral outriggers shown in this photograph were used only on the spandrel connection tests to help prevent twist of the free ends of the beam stubs.

During the progress of a typical test the relation between moment and rotation was approximately linear at low loads. As the connections began to yield, the rotations increased disproportionately to the moments. In an actual beam framed in a building the non-linear increase in rotation would result in an increasing ratio of vertical reaction to moment. The laboratory specimens were tested with a constant load arm and, therefore, a constant ratio between reaction and moment.

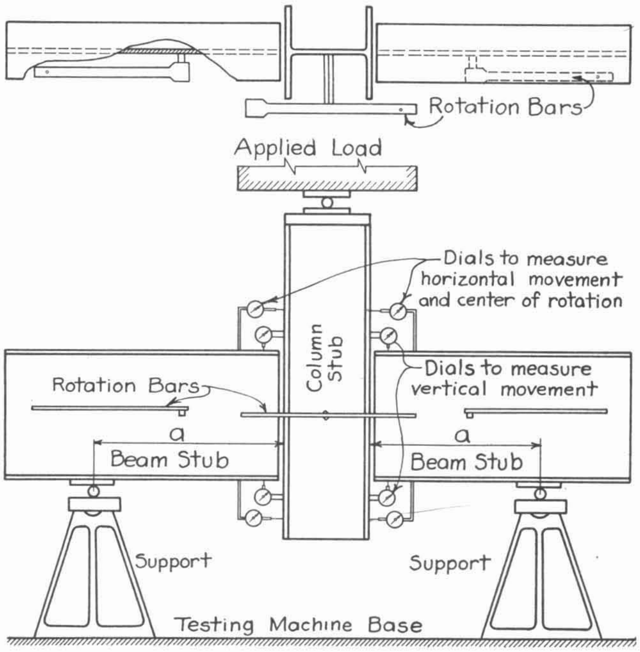


Fig. 2—Typical Arrangement for Moment-Rotation Test

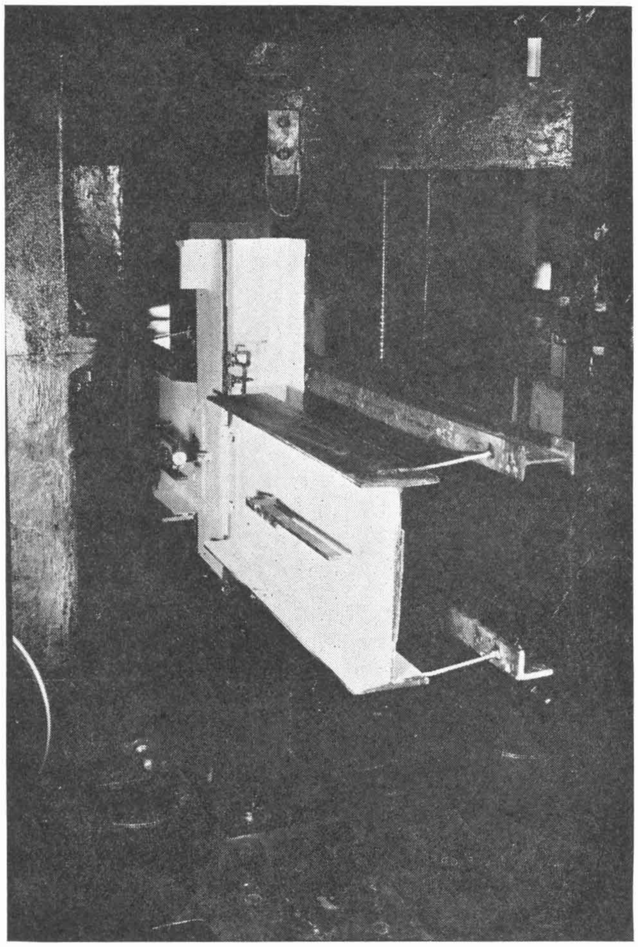


Fig. 3—Test Setup W19

Hence the laboratory tests in the non-linear range provided insufficient vertical reaction and too much moment.

Additional information regarding reaction capacity was obtained after the rotation test had been carried to failure or to a large distortion of one side of a connection assemblage. The remaining duplicate connection, already distorted in rotation, was tested for direct reaction capacity by the test set-up shown in Fig. 4. In calculating the connection reaction in this set-up, the assumption of simple supports at each end was made. The actual reaction at the connection would be slightly higher and the assumption is therefore on the safe side. The results of these tests provide information listed under the heading "Supplementary Shear Tests."

The results of the laboratory moment-rotation tests may be correlated with the corresponding design requirements for any particular beam length-depth (l/d) ratio and for any specified load distribution. Figure 5, with moments as ordinates and rotations as abscissas, shows curves which might have been plotted from test results in which successive increments of moment and rotation were measured. Curve A shows hypothetical test results for a nearly "rigid" connection which develops more than twice the fixed end moment without yield and therefore has a factor of safety of more than 2.

The sloping straight lines intersecting curves A, B, C and D indicate the design requirements and two times the design requirement in moment and rotation combined. These design requirement lines were called "beam lines" by C. Batho,[6] who showed that by connecting the ordinate axis equal to fixed end moment with the abscissa axis equal to simple beam rotation a graphical method of relating the M-φ (moment-rotation) curve to actual beam performance was obtained. The ordinate intersection of the test curve with the "beam line" shows the end moment that would be developed at the ends of a particular beam attached to non-rotating columns by the connection in question. The ratio of actual end moment to fixed-end moment multiplied by 100 is termed the "percentage rigidity." The abscissa at the beam line intersection gives the actual rotation of the semi-rigid connection between beam ends and non-rotating columns.

The equation of the "beam line" may easily be shown to be[6]

$$M + \frac{2EI\varphi}{l} = M_R$$

This is a straight line in terms of the two variables M and φ.

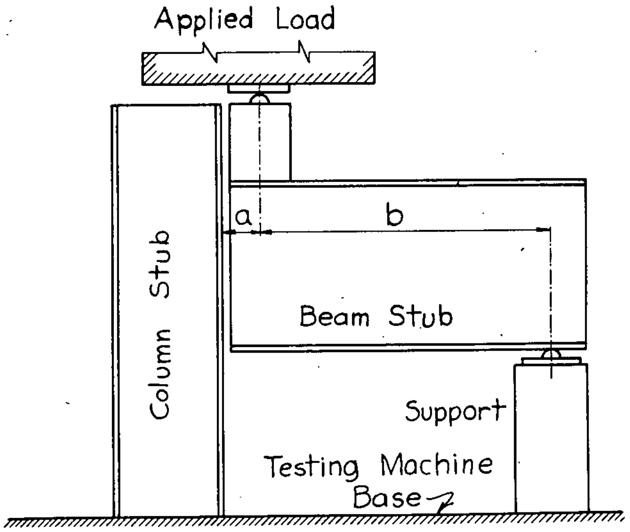


Fig. 4—Arrangement for Supplementary Shear Test after Moment-Rotation Test

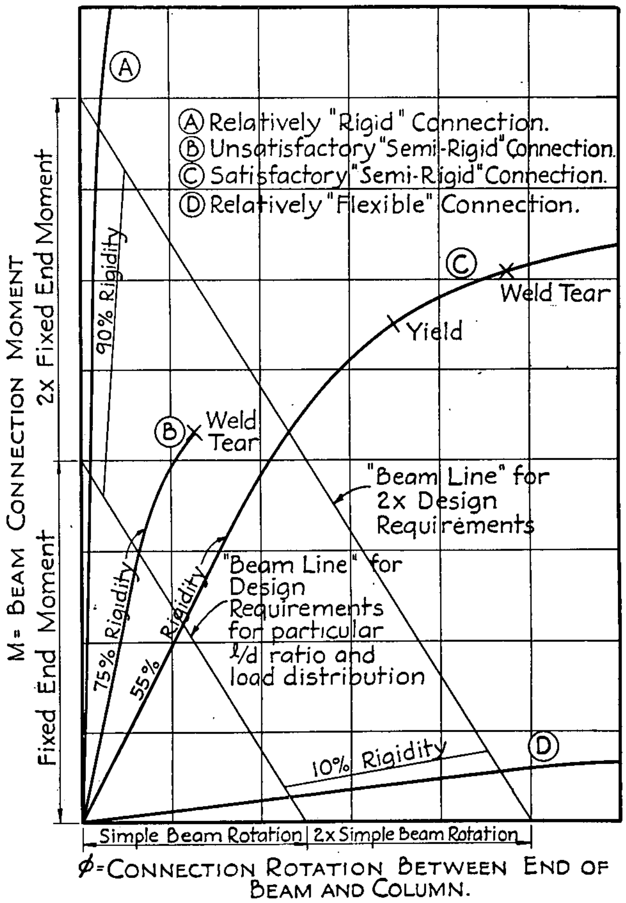


Fig. 5—Moment and Rotation Design Requirements for a Beam to Column Connection

E = modulus of elasticity
I = moment of inertia of beam
l = beam span
M_R = fixed end moment for a beam with "rigid" connections

When $\varphi = 0$, $M = M_R$ and when $M = 0$, $\varphi = M_R l/2EI$ = simple-beam end rotation. If the columns bend by frame action the actual design requirements may be slightly more or less but the beam line for the fixed column provides a simple and graphical comparison for various connections.

Curve B indicates a hypothetical M-φ curve for a semi-rigid connection with 75 per cent rigidity in the low load range. In curve B the weld is presumed to tear before the beam line representing two times design requirement is reached. Such a connection may be termed unsatisfactory if a factor of safety of 2 is desired. Curve C indicates a satisfactory semi-rigid connection, which passes two times the design requirements with small loss in proportionality, therefore, having a factor of safety of more than 2, both in respect to moment and rotation.

It is again emphasized that connections designed for shear but not designed for moment must nevertheless pass the moment-rotation design requirements indicated in Fig. 5 in order to be satisfactory.

The behavior of a relatively flexible connection with less than ten per cent rigidity is shown in Fig. 5 by curve D. Such a connection satisfactorily meets the moment-rotation design requirements and if it also meets the shear requirements it is a satisfactory connection.

The plot of M-φ values in an actual test, W20, is related in Fig. 6 to the moment-rotation design requirements of a 14WF34 beam. Beam lines are shown for both uniform load and for a concentrated load in the

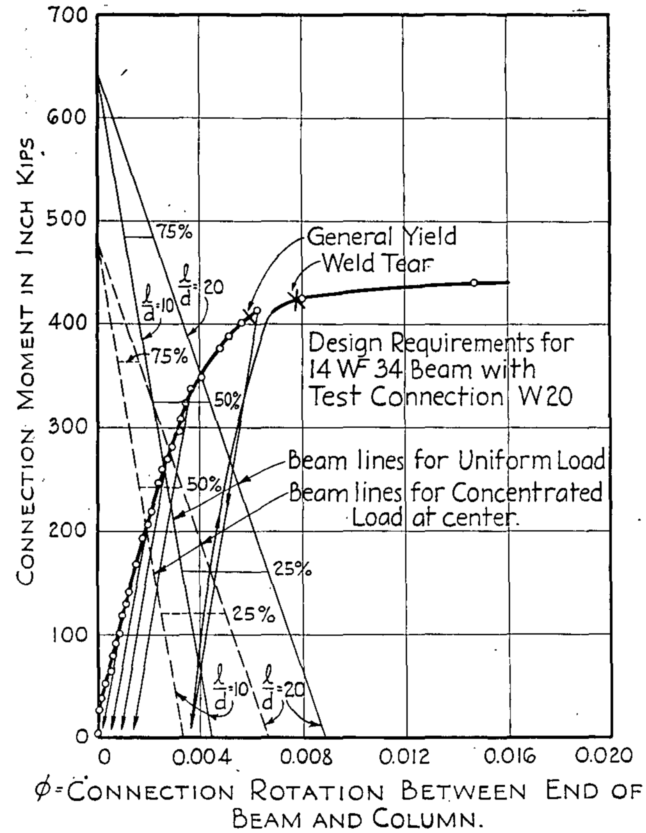


Fig. 6—M-φ Curve for Test W20 Related to Moment-Rotation Beam Design Requirements

center, as well as for l/d ratios of 10 and 20, between which occur most length-depth ratios of building beams. The construction of the beam lines is aided by the following relations which are easily derived. It is assumed that the beams are designed for full simple beam moment, using a maximum allowable fiber stress of 20 kips per sq. in. E is assumed to equal 30,000 kips per sq. in., S = section modulus, l = beam length, d = beam depth.

	Uniform Load	Concentrated Load at Center
Fixed end moment	$M_R = \frac{40}{3} S$	$10S$
Simple beam end connection rotation	$\varphi s = \frac{4}{9}\left(\frac{l}{d}\right) \times 10^{-3}$	$\frac{1}{3}\left(\frac{l}{d}\right) \times 10^{-3}$

In view of the fact that the M-φ beam design requirements are more severe for uniform load the concentrated load beam lines are omitted in the remainder of the report, for the various connection test results.

General Remarks on Test Program and Design of Specimens

The various test specimens have been classified as to type or function under five general headings, as follows:

Group I.—The beam web is welded directly to the column or girder by fillet welds or, as an alternate, side plates are used to bridge the clearance beyond the end of the beam web.

Group II.—(*a*) A seat angle carries the vertical reaction and the beam web is supported laterally by a side plate or clip angle.

(*b*) A seat angle carries the vertical reaction and a top plate is fillet welded to the top beam flange and butt welded to the column flange or web.

Group III.—A tee is welded to the column or girder, with provision for slot and fillet welds between the outstanding leg of the tee and the beam web.

Group IV.—In these connections a clip angle is shop welded to the beam web with fillet welds. Erection bolt holes are provided in the outstanding legs of the angle. After bolting up, the beam web is butt welded to the column or girder with weld laid over the shop fillet weld.

Group V.—These consist of several different methods used for spandrel beam connections applied eccentrically to one column flange.

The following general remarks pertain to all of the test specimens, details of which are presented in this report. All material was required to meet A. S. T. M. Specification A-7-39 for Building and Bridge Steel. All sections of the same size were cut from a single rolling and an extra sample was furnished for tensile coupons. All welding was done by qualified welders, with field positions simulated for field welds, and with weld electrodes corresponding to A. S. T. M. Specification A 233-40 T, Class E6010 for vertical welds and fillet, Class E6020 for horizontal and flat welds. The specimens were assembled in position by tack welding and then welded up according to the shop details. Beam and column sizes of all specimens are tabulated along with test results in Table II.

The beams were connected to the column web rather than column flange when no special requirements governed the choice. Connections to the column flange

Fig. 7—Test W1

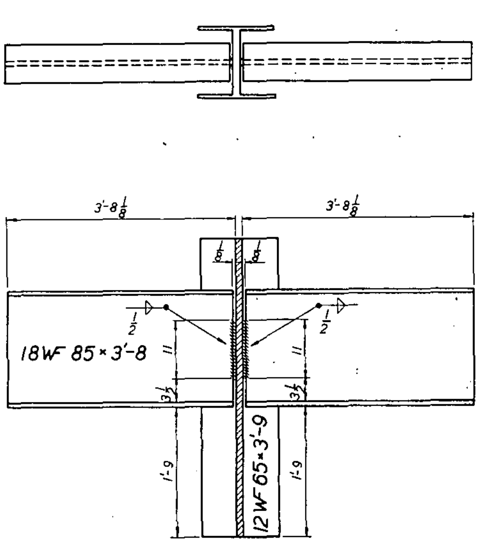


Fig. 8—Test W3

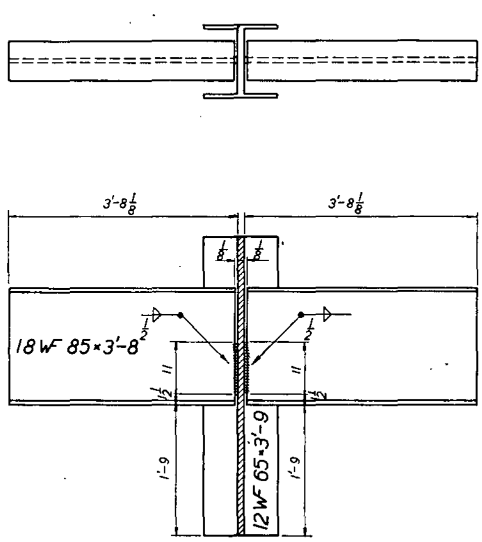
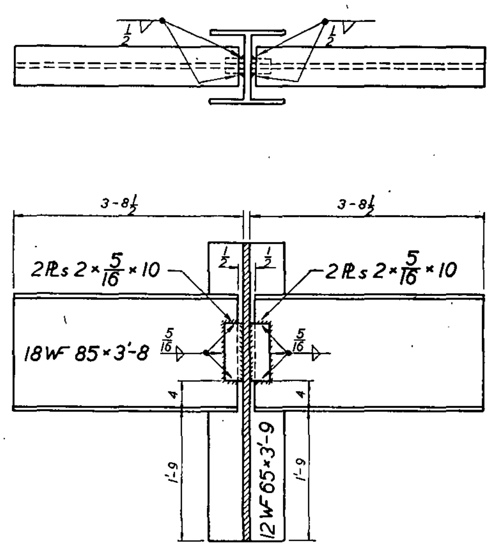


Fig. 9—Test W5



are given additional flexibility by the bending of the flange. Hence the most critical test condition is to the column web, and in addition such connections also simulate conditions which would exist in a beam-to-girder web connection.

The procedure used in designing the test specimens was based on current practice, with required weld areas calculated wherever possible on the basis of the following allowable stresses.

	Allowable Stress, Psi
Butt welds in tension	15,600
Fillet welds	13,600
Allowable shear in beam web	13,000

The 1938 edition of the AMERICAN WELDING SOCIETY Handbook, together with design data sheets prepared by the American Bridge Company and Bethlehem Steel Company, was consulted in designing the specimens. The specimens, however, do not necessarily represent design practice approved by any one of these organizations. Further details on design will be discussed under the various group sub-headings.

Group I—Design Details and Test Results

Design Details

The six specimens in this group, W1 to W6, inclusive, were all welded at the Fritz Engineering Laboratory. Specimens W1, W3 and W5 are classified as Sub-Group I (*a*), and the details of these specimens are shown in Figs. 7 to 9, inclusive. In this Sub-Group all of the beams were 18WF85 welded to 12WF65 column sections, and were designed for a 72-kip vertical reaction. All of the specimens were welded to the column web and the test results apply to beam-to-girder connections equally as well as to beam-to-column connections. In tests W1 and W3 only $^1/_8$-in. clearance or "setback" was allowed as the beam webs were welded directly to the column web. In test specimen W5 the use of side plates

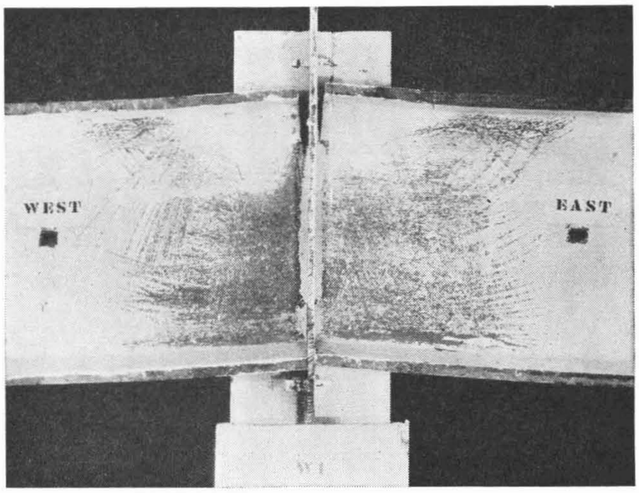


Fig. 10—Test W1

allowed a $^1/_2$-in. setback which would facilitate erection.

Tests W2, W4 and W6, constituting Sub-Group I (*b*), are similar to W1, W3 and W5, respectively, except that a lighter weight beam size (18WF47) and smaller welds were used. These connections were designed for a 24-kip vertical reaction. This Sub-Group is detailed in Figs. 14 and 15.

From the point of view of erection, connections of this type offer difficulties because of the small $^1/_8$-in. clearance and because of the difficulty in holding beams in place while welding. The use of side plates eliminates the small clearance. In designing the four connections, which had the beam webs welded directly to the column web, shear in the beam web adjacent to the welds was the determining factor for weld length. The weld size was governed by the beam web thickness and the $^1/_8$-in. setback, this being considered sufficient to force yielding in the beam web.

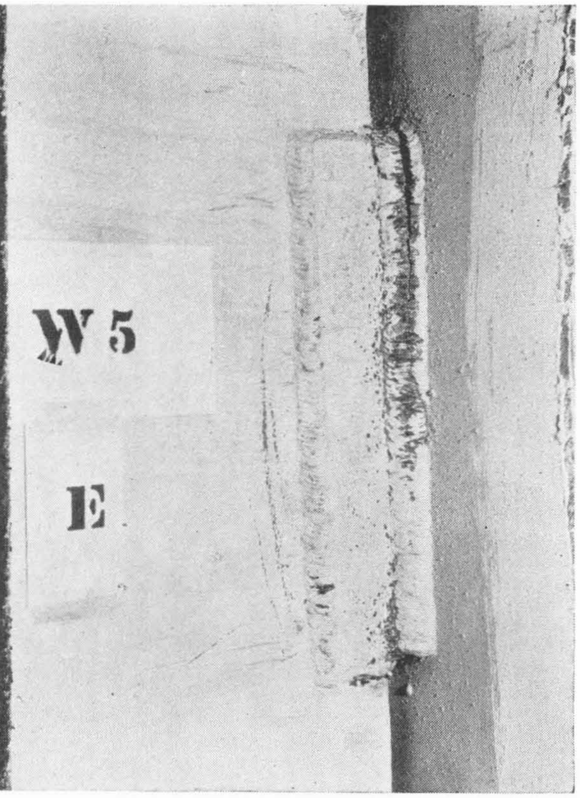


Fig. 11—Test W3

Fig. 12—Test W5

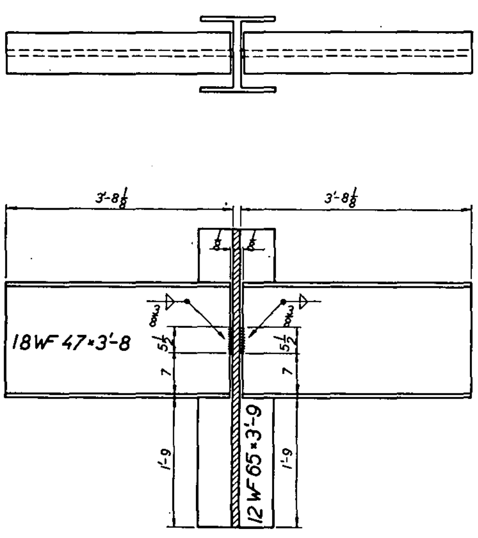


Fig. 14—Test W2

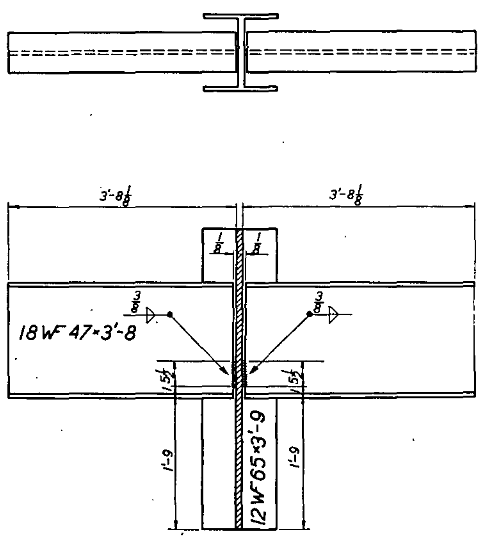


Fig. 15—Test W4

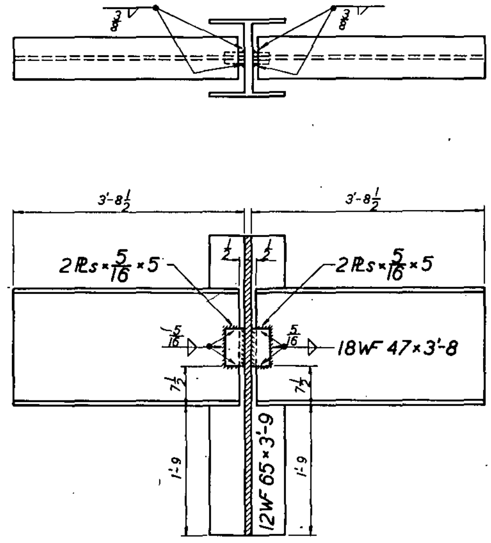


Fig. 16—Test W6

Individual Test Results, Sub-Group I (a), Tests W1, W3 and W5

Test W1.—Figure 10 shows by photograph the condition of the specimen after the moment-rotation test. Figure 13 shows the graph of the moment-rotation readings during the test together with the beam line for design requirements for uniform load. Figure 13 shows this connection to have yielded rather generally before reaching design requirements for $l/d = 10$, but the connection shows considerable rotation and strength reserve beyond general yielding.

A 12-in. moment arm was used in the moment-rotation test. At $V = 30$ kips and $M = 360$ kip-in. the beam web showed initial signs of yielding adjacent to the lower ends of the welds, the strain lines showing no definite orientation.

At $V = 55$ kips and $M = 660$ kip-in., a strain line adjacent to and roughly parallel with the entire length of the weld showed that the beam web had yielded along the full length of weld. This was followed by further progressive yielding in the beam web.

At $V = 72.5$ kips and $M = 870$ kip-in., both welds on the east beam cracked very slightly at the top. Progressive tearing through the weld took place at higher loads. Owing to the cracks in the welds and yielding in the beam web the lower flanges of the beams began to bear against the columns, after which the connections became stiffer.

At $V = 159.9$ kips and $M = 1918$ kip-in., the ultimate load was reached and the load thereafter fell off as further tearing in the east beam weld developed. The test was stopped when the welds had torn about three inches. The weld fracture started in the root of the weld but shifted as the test progressed to the area along the end of the beam web.

In the supplementary shear test this connection developed 200 kips shear on the previously cracked weld of the west beam with no further weld distress.

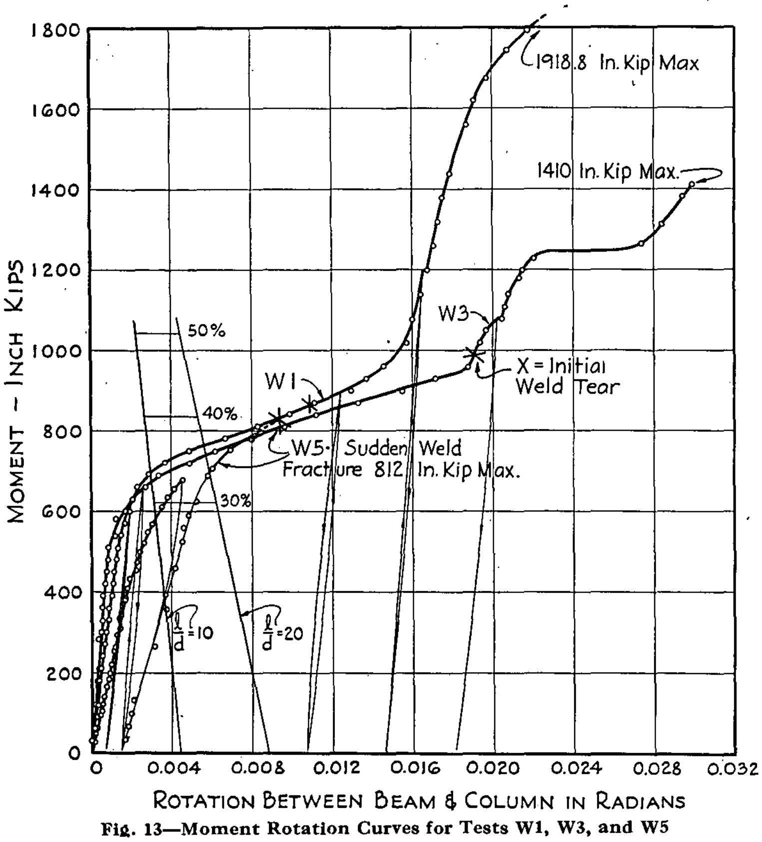


Fig. 13—Moment Rotation Curves for Tests W1, W3, and W5

Test W3.—Figure 11 shows a photograph of the condition of the specimen after the moment-rotation test. The general behavior of this test was similar to that of W1, as shown by the moment-rotation graph in Fig. 13. As in the case of W1, general yielding developed in the connection before the moment-rotation requirements were passed, but the connection showed considerable reserve beyond general yielding.

A 12-in. moment arm was used in the moment-rotation test. At $V = 30$ kips and $M = 360$ kip-in., the beam web showed initial signs of yielding. Continued loading to $V = 50$ kips and $M = 600$ kip-in. caused progressive yielding in the beam web adjacent to the weld.

At $V = 80$ kips and $M = 960$ kip-in., both beam flanges were beginning to bear against the column and the connection stiffness increased.

At $V = 82.5$ kips and $M = 990$ kip-in., the top of the northeast weld cracked through the root, followed at higher loads by progressive tearing of the northeast and southeast welds.

At $V = 117.5$ kips and $M = 1410$ kip-in., the maximum load was reached after the crack had progressed about two inches. The cracks originally had appeared in the root of the weld, but after a short distance the crack moved to the column side of the weld and the base metal of the column pulled out.

In the supplementary shear test the connection developed 188 kips without weld failure, but the test was

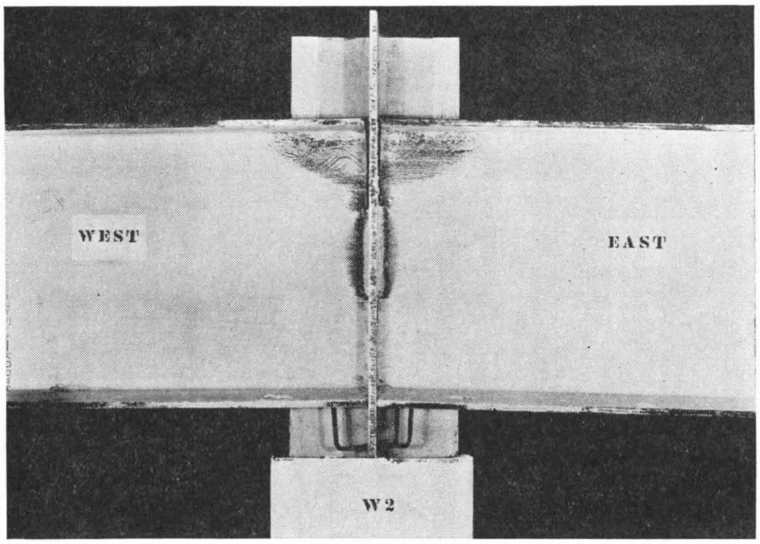


Fig. 17—Test W2

stopped because of crippling of the beam web under the bearing block.

Test W5.—Figure 12 shows a photograph of the condition of the specimen after the moment-rotation test. As shown in Fig. 13 this connection had much less rotation reserve after yielding than did W1 and W3. The comparatively poor showing of this connection is attributed to the fact that the side plates transfer the stress to a very large area of the beam web. The relatively light welds cannot develop web yield strength prior to weld failure. Since a large part of the ultimate rotation of this general type of connection must come from beam web yielding, the result is low flexibility with insufficient strength to develop the accompanying end-restraint moments.

A 9-in. moment arm was used in the moment-rotation test up to V = 75 kips, after which the specimen was unloaded and reloaded with a 13-in. moment arm.

At V = 72.5 kips and M = 652 kip-in., initial yielding was noted in the middle of the beam web near the side plate.

At V = 62.5 kips and M = 812 kip-in., after the change in moment arm, both welds on the east connection failed suddenly through the weld roots, and the connection would not carry any increase in load. The welds were cracked for a distance of about six inches in the east connection.

In the supplementary shear test the west connection developed a reaction of 170 kips, at which time the web buckled under the loading block.

Individual Test Results, Sub-Group I (b), Tests W2, W4 and W6

Test W2.—The specimen is detailed in Fig. 14, photographed after moment-rotation test in Fig. 17, and the moment-rotation curve is plotted in Fig. 20. General yielding of this connection is noted in Fig. 20 between the design requirements for l/d = 10 and l/d = 20. As in the case of the heavier connections of the same type, there is a considerable reserve of connection rotation beyond general yielding.

The moment-rotation test was started with a moment arm of eight inches. At V = 13 kips and M = 104 kip-in., initial yielding in the beam web adjacent to the lower end of the weld was noted. The yielding in the beam web gradually progressed until at V = 25 kips and M = 200 kip-in., the yielding adjacent to the weld was general.

At V = 28 kips and M = 224 kip-in., the ultimate load was reached as the southwest weld fractured for a length of one inch down from the top. At this point the load was removed and the supports were moved in to a distance of two inches from the column face. Load was applied up to V = 117.5 kips and M = 235 kip-in., when the west beam web failed with a crack at both top and bottom. No supplementary shear test was made since the test with the two-inch moment arm had adequately checked the reaction value of the connection.

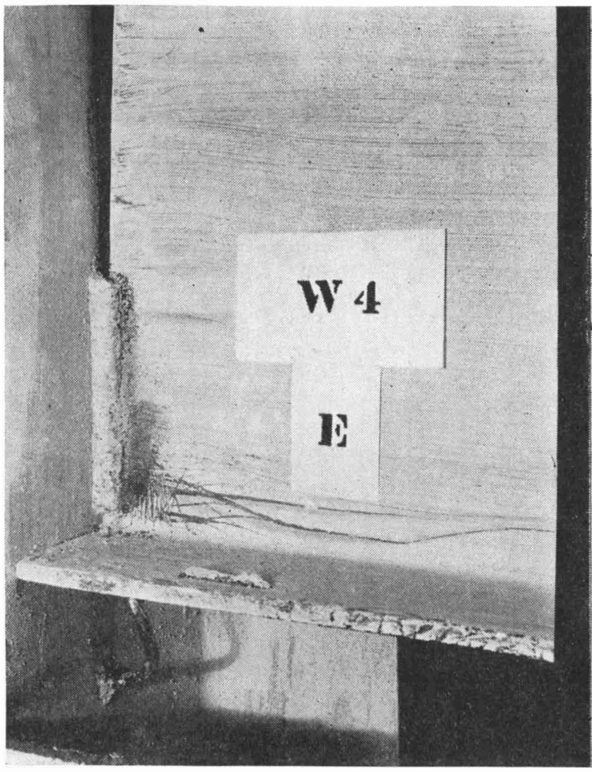


Fig. 18—Test W4

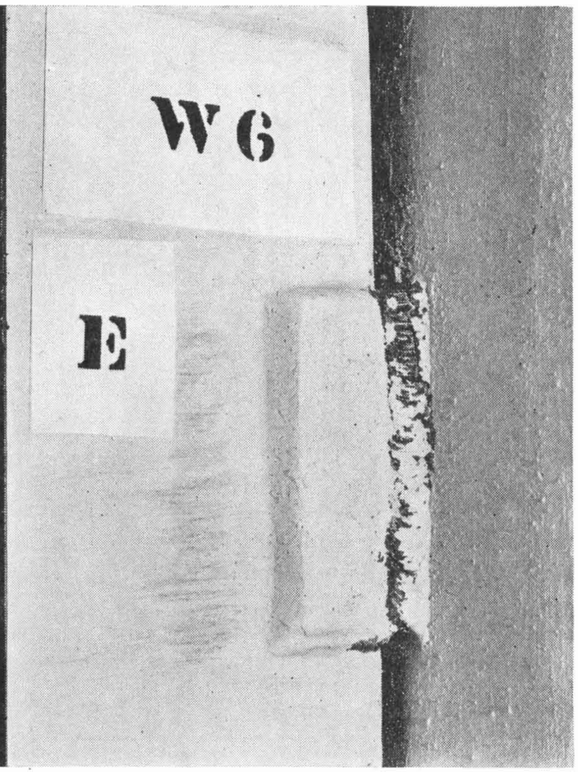


Fig. 19—Test W6

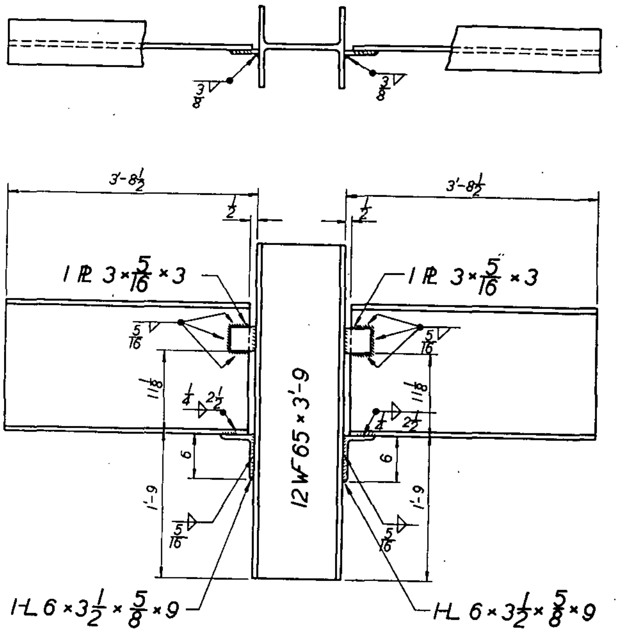


Fig. 21—Test W7

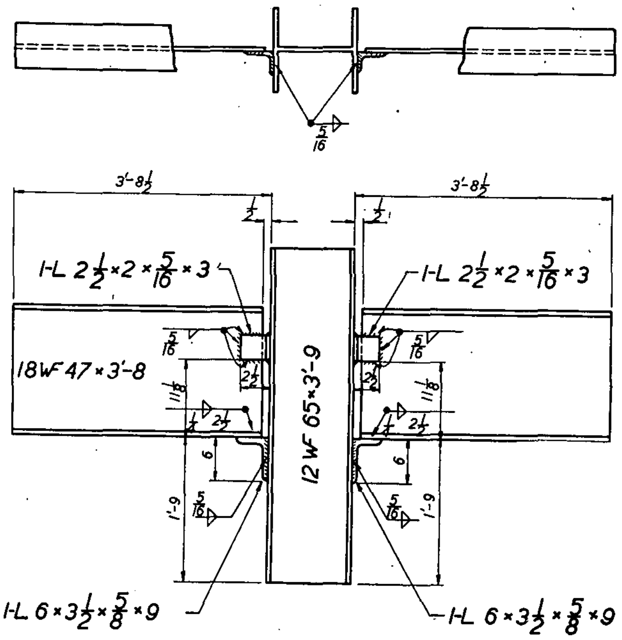


Fig. 22—Test W8

Test W4.—The specimen is detailed in Fig. 15, photographed after moment-rotation test in Fig. 18, and the moment-rotation curve is plotted in Fig. 20 along with W2 and W6. General yield of this connection took place below the design requirements for $l/d = 10$, but there was a fair reserve beyond yield. An eight-inch moment arm was used throughout the test.

At $V = 17$ kips and $M = 136$ kip-in., yielding was first noted at the bottom of the weld in the beam web, spreading progressively with increasing load.

At $V = 27$ kips and $M = 216$ kip-in., the beam web began to yield near the top of the weld.

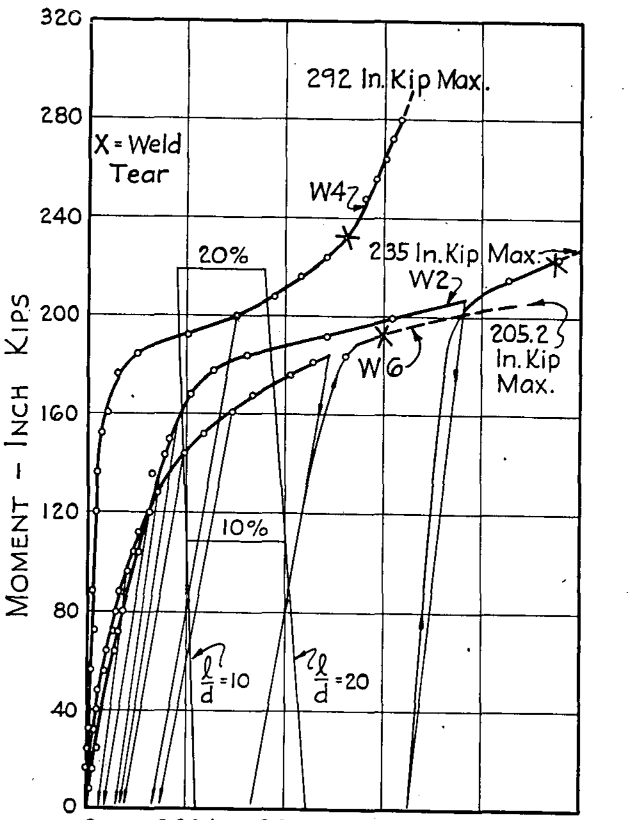


Fig. 20—Moment Rotation Curves for Tests W2, W4, and W6

At $V = 29$ kips and $M = 232$ kip-in., a crack developed in the top of the east weld, followed by progressive tearing in the beam web up to the maximum load of $V = 36.5$ kips and $M = 292$ kip-in., when the tear in the beam web was about three inches long and the test was stopped.

In the supplementary shear test a reaction of 110 kips was applied to the west beam without weld failure and the test was stopped because of crippling in the beam web.

Test W6.—The specimen is detailed in Fig. 16, photographed after the moment-rotation test in Fig. 19, and the moment-rotation curve is plotted in Fig. 20 along with tests W2 and W4. This test showed a deficiency similar to its companion test W5 in the heavier beam size, although to a less degree. Too much web area is used in resisting the moment, reducing the flexibility of the connection and causing failure to be primarily in the weld metal.

The fillet welds which are detailed as $^3/_8$ in. in Fig. 16 were $^1/_8$ in. oversize on the west connection and slightly less than $^1/_{16}$ in. undersize on the east connection.

At $V = 18$ kips and $M = 144$ kip-in., a strain line appeared at the bottom of the northeast side plate adjacent to the weld, and at $V = 22$ kips and $M = 176$ kip-in., a similar strain line appeared on the southeast side plate. No further strain lines were noted until the connection failed in the undersized southeast weld at the top, and at load of $V = 24$ kips and $M = 192$ kip-in.

At $V = 25.7$ kips and $M = 205.2$ kip-in., a maximum load was reached as the northeast weld cracked. Continued loading increased the length of the crack to about two inches, when the test was stopped. The weld fractured along the root on both sides.

In the supplementary shear test a reaction of 127 kips was applied to the west connection. At this load the side plates were yielding and the beam web crippled under the bearing block.

Group II—Design Details and Test Results

Design Details

The five specimens in this group are subdivided into Sub-Group II (*a*), specimens W7 and W8 detailed in

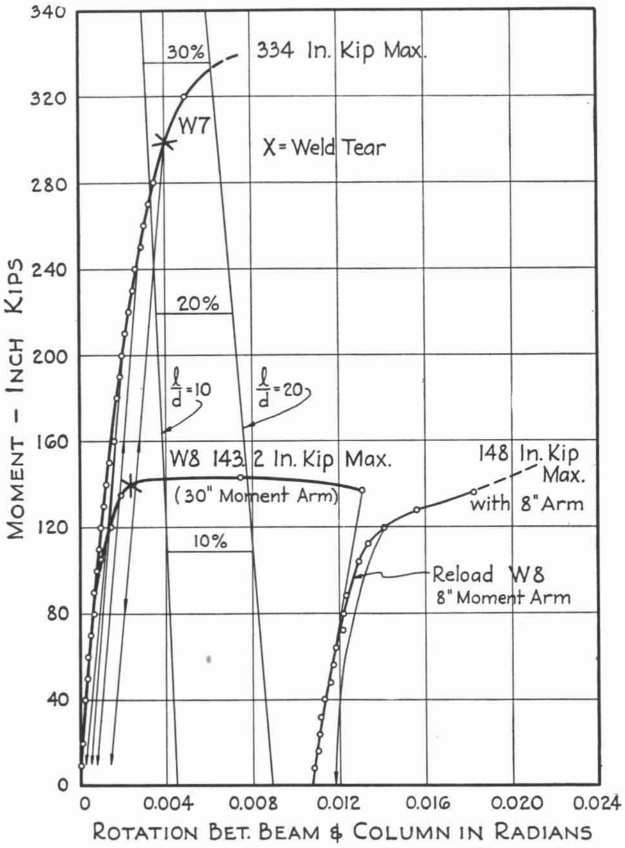


Fig. 25—Moment Rotation Curves for Tests W7 and W8

Figs. 21 and 22, and Sub-Group II (*b*), which includes specimens W9, W10 and W21, detailed in Figs. 26, 27 and 28. All of the specimens except W21 were designed for a vertical reaction of 24 kips and W21 was designed for 72 kips.

Test specimen W21 was fabricated in the Fritz Laboratory and the other four specimens were fabricated by the Bethlehem Steel Company. All of the specimens were supported by seat angles, designed to carry all of the vertical reaction. In tests W7 and W8 (Group II (*a*)), side plates and side angles, respectively, were used to provide lateral support for the top of the beam. In tests W9 and W10 top plates welded between the top flange of the beam and the column flange were used to provide lateral support for the top of the beam. After these tests were completed it was apparent that the flexibility of the column flanges had contributed to the elastic behavior of tests W9 and W10. Test W21 was accordingly designed with an 18WF85 beam supported by a stiffened seat and held laterally by top plates welded to the column web.

The allowable reaction values for the welded seat angles were determined by design tables and charts used in the Bethlehem Steel Company design office. A previous A. W. S. investigation at Lehigh corroborates the use of these values.[7]

Individual Test Results, Sub-Group II (a), Tests W7 and W8

Test W7.—The specimen is detailed in Fig. 21, photographed after the moment-rotation test in Fig. 23, and the moment-rotation curve is plotted in Fig. 25 along with W8. The first weld tear developed before reaching the design requirements for $l/d = 20$ and the weld failed shortly after passing this beam line.

The connection was much too weak with respect to moment to resist the moments which would be developed in ordinary beam action by the inherent stiffness of the connection. The eccentric pull on the column weld tended to tear the weld at the root and the connection does not take advantage of the flexibility of the column flanges, but rather pulls directly in line with the comparatively rigid column web.

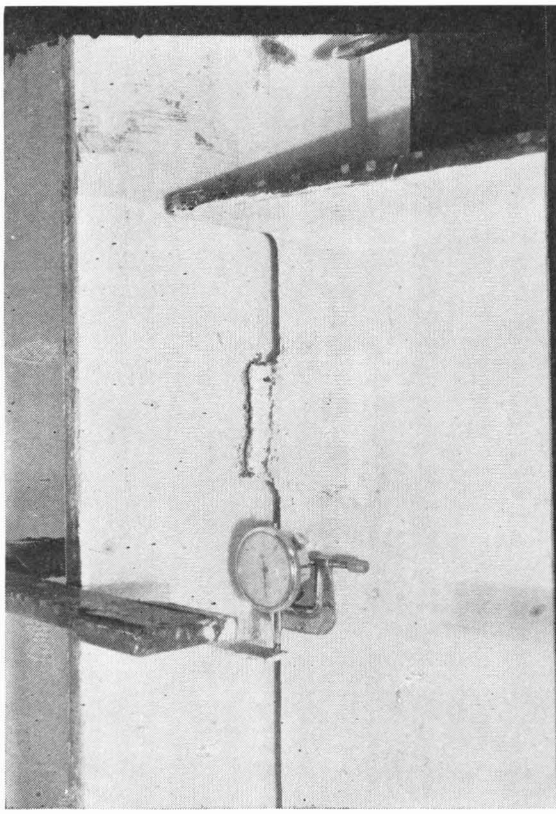


Fig. 23—Test W7

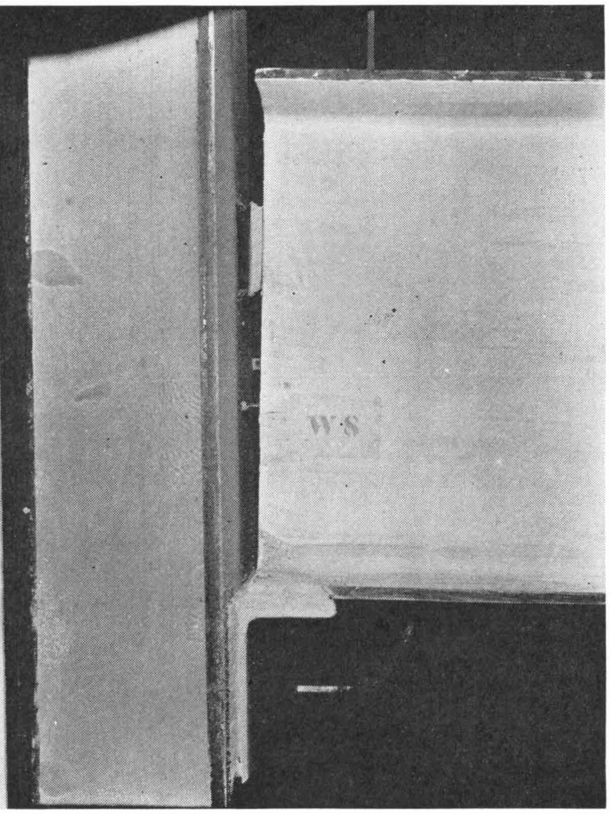


Fig. 24—Test W8

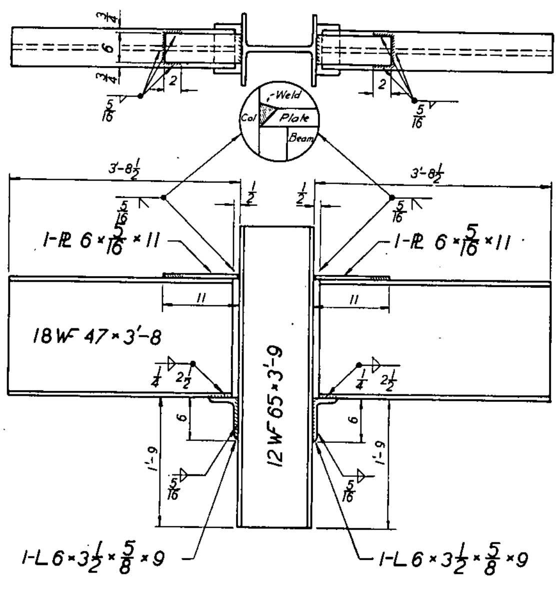


Fig. 26—Test W9

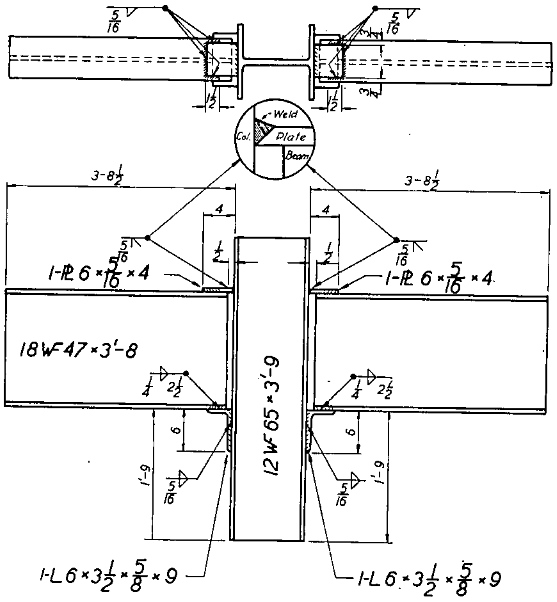


Fig. 27—Test W10

At V = 10 kips and M = 200 kip-in., the clip plate began pulling away from the column along the unwelded edge; and at V = 11.5 kips and M = 230 kip-in., a weld ping was heard, usually a sign of an initial weld crack.

At V = 15 kips and M = 300 kip-in., a visible crack was noted in the east plate weld; and at V = 16.7 kips and M = 334 kip-in., the entire east plate connection pulled away suddenly and the test was stopped. Failure occurred through the root of the plate-to-column weld.

A reaction load of 89 kips was applied to the west connection in the supplementary shear test, with failure due to cracking of plate-to-column weld, combined with crippling of beam web under bearing block and above the seat angle.

Test W8—The specimen is detailed in Fig. 22, photographed after the moment-rotation test in Fig. 24, and the moment-rotation curve is plotted in Fig. 25 along with W7. Initial weld tear and general yield both occurred before the moment rotation design requirements for l/d = 10 were reached.

This connection, like W7, tends to develop high moments because of its inherent rigidity, but is too weak to resist them. The disposition of weld metal, although similar to some actual details, puts the connection at a disadvantage. The omission of welds along the outstanding legs of the clip angle adjacent to the column, as advocated in a previous paper,[2] would have provided greater flexibility with respect to rotation.

The heel of the clip angle began to pull away from the column at V = 4.5 kips and M = 135 kip-in., and the column welds at the same point broke suddenly at V = 4.77 kips and M = 143.2 kip-in. With continued loading the welds progressively tore away from the column. The load points were moved from 30 in. moment arm in to 8 in. and a maximum of V = 18.5 kips and M = 148 kip-in. was reached.

In the supplementary shear test a maximum reaction of 81 kips was reached and the test stopped because of beam web crippling above the seat angle.

Individual Test Results, Sub-Group II (b), Tests W9, W10 and W21

Test W9.—The specimen is detailed in Fig. 26, photographed after the moment-rotation test in Fig. 29, and the moment-rotation curve is plotted in Fig. 32 along with W10. This connection exhibits essentially elastic behavior beyond design requirements for l/d = 20 and there is an adequate safety factor both with respect to moment and rotation. The welds detailed as $^5/_{16}$-in. butt welds were actually placed in an incomplete 45° bevel and, hence, do not have a full $^5/_{16}$-in. effective thickness. The welds have the appearance of, and probably approach, fillet welds rather than true butt welds.

The downward deflection of the top flange of the beam, produced by seat-angle deflection and compression in the beam web, relieves the stress in the root of the butt weld and results in favorable behavior of the top plate.

First yielding was noted in the column web adjacent to the east seat angle at V = 31 kips and M = 620 kip-in.

At V = 35 kips and M = 700 kip-in., the beam web began to yield locally above the seat angle. At V = 40 kips and M = 800 kip-in., additional yielding was noted in the column web adjacent to the top plate. The seat angle showed signs of initial yielding due to bending at V = 53 kips and M = 1060 kip-in., although the con-

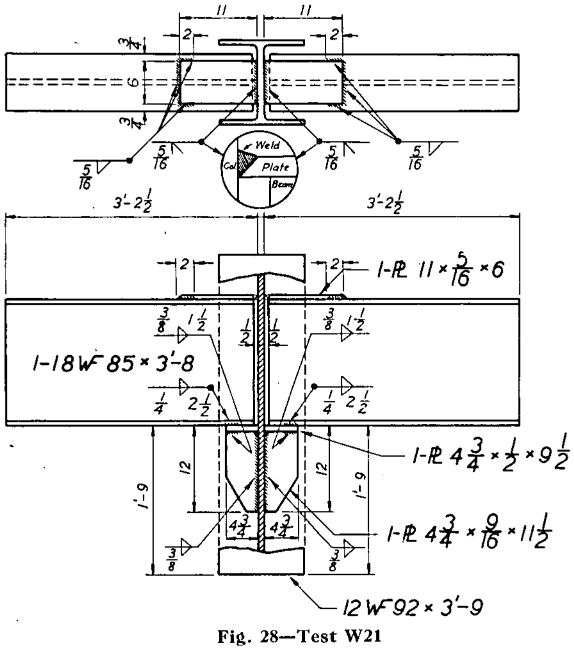


Fig. 28—Test W21

nection in general was essentially undamaged at this load.

At V = 67.5 kips and M = 1350 kip-in., the column flanges were showing signs of "dishing." The downward deflection of the top flanges of the beams was causing the top plates to pull down, leaving a gap between the plates and the top beam flanges. The top plates were stretched beyond the yield point at this load. Yielding progressed further at points already noted, and at V = 76.85 kips and M = 1537 kip-in., the beam web buckled laterally above the seat angle after general crippling had occurred in the beam web. Both top plate butt welds were intact when the test was stopped although the top plates had yielded generally. No supplementary shear test was necessary since the beam web had failed due to web crippling and buckling above the sea angle.

Test W10.—The specimen is detailed in Fig. 27, photographed after the moment-rotation test in Fig. 30, and

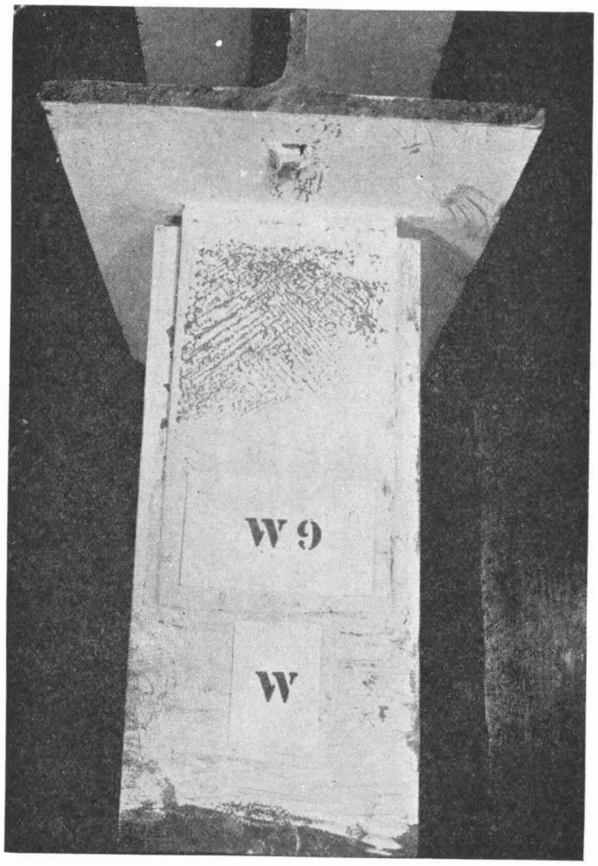


Fig. 29—Test W9

the moment-rotation curve is plotted in Fig. 32 along with W9. Although the top plate was considerably shorter in this test than in W9, the same behavior, essentially elastic in general, was noted considerably beyond the maximum design requirements.

The higher ultimate reaction carried by this connection for l/d = 20 indicates the participation of the short top plate in carrying the vertical reaction and in reducing web buckling.

Visual inspection of the top plate butt weld after failure showed that the top plate had not been properly beveled according to details and that the weld approached a fillet rather than the butt weld as detailed. Slag inclusions and poor penetration were noted in the root of the weld. The tendency of the top plate to pull down, as in the case of W9, tended to relieve stress concentrations and the tendency to tear in the root of the weld.

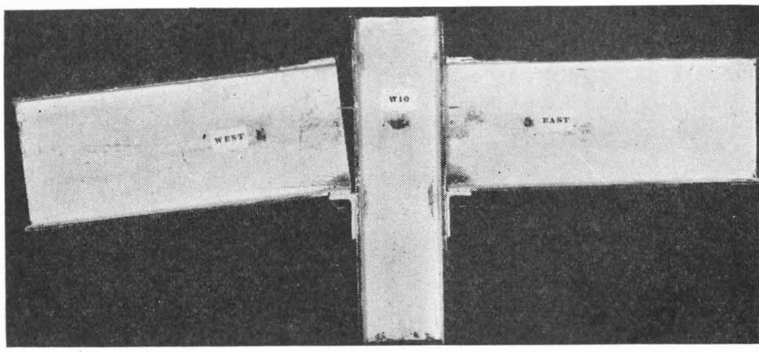


Fig. 30—Test W10

The beam web showed signs of yielding locally above the seat angle at V = 38 kips and M = 760 kip-in., but the connection behaved elastically in general up to a load of V = 52 kips and M = 1040 kip-in., when initial yielding was noted in the top plate. At V = 60 kips and M = 1200 kip-in., the column web yielded adjacent to the seat angle. The column flanges showed signs of dishing near the top plate at V = 72 kips and M = 1440 kip-in. Progressive yielding continued in various parts of the connection until at V = 93.25 kips and M = 1865 kip-in. the west top plate butt weld broke suddenly with a ragged tear. The east top plate weld was beginning to tear at the same time.

No supplementary shear test was made because the connection had shown adequate strength in both shear and moment.

Test W21.—The specimen is detailed in Fig. 28, photographed in Fig. 31, and the moment-rotation curve is plotted in Fig. 33. In view of the high quality of performance shown by tests W9 and W10, test W21 was designed to serve as a more critical check on the possibilities of this type of connection. Attachment was made to the column web rather than column flange to eliminate the flexibility of the column flanges. The

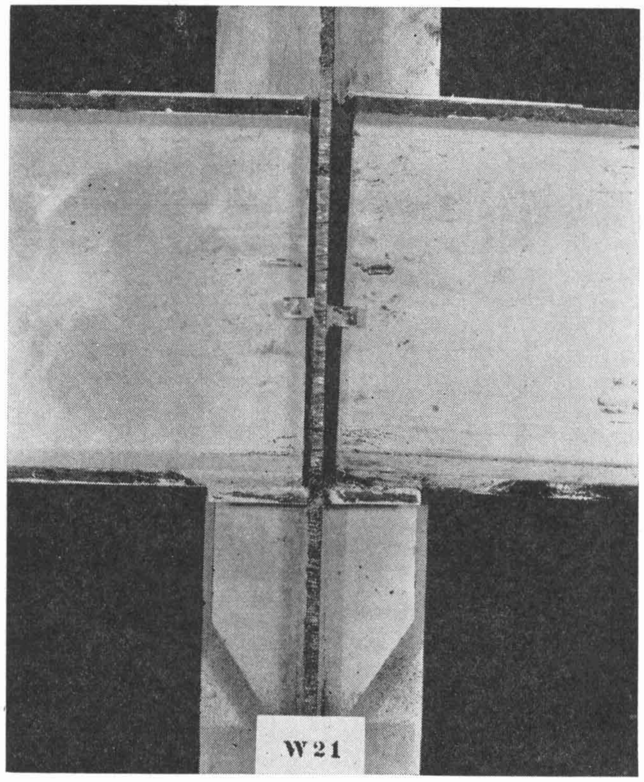


Fig. 31—Test W21

stiffened seat construction required by the larger design reaction of 72 kips reduced the flexibility in the seat construction. The heavier beam size reduced the possibility of web buckling which, in the case of W9 and W10, had tended to pull the top plate down and render it more effective.

This specimen was made in the laboratory by a qualified welder, but the quality of the butt welds was very poor in spite of the fact that the top plates were carefully beveled as specified. The welds appeared satisfactory on the outside, prior to fracture, but inspection after failure revealed slag inclusions and poor penetration. Since the butt weld was reinforced by a fillet weld with a long horizontal leg, the equivalent size of the weld through the fractured section is about $^1/_4$ in. On this one specimen the welder used the small laboratory A.-C. machine rather than the D.-C. machine, which was used on all other specimens fabricated in the laboratory. Unfamiliarity with the laboratory A.-C. machine, particularly the low power output, were among the main causes of the poor welds. Specimens made at the Bethlehem Steel Company were all A.-C. welded. The quality of the butt welds on W9 and W10 was also poor for other reasons, but these connections showed up very favorably.

Although the moment-rotation curve for this connection passes through the design-requirement for l/d = 20 without definite yielding in general, maximum load was reached shortly afterward and the connection is deficient both with respect to rotation and moment strength.

At V = 60 kips and M = 1200 kip-in., initial yielding was noted in the beam web directly above the seats. Shortly afterward, local yielding was noted in the top plate, but general yielding of the top plate was not developed even at maximum load.

At V = 82.5 kips and M = 1650 kip-in., maximum load was reached, accompanied by sudden failure of the top plate butt weld on the east beam. The thrust welds on the lower flange failed simultaneously and the entire beam stub dropped free of the connection.

In the supplementary shear test of the west beam a reaction of 153 kips was sustained by the seat, with failure caused by crushing and bending of the beam web above the support.

Group III—Design Details and Test Results

Design Details

There are only two specimens in this group, W11 and W12, detailed in Figs. 34 and 35.

The beams were connected to the column webs by means of structural "tees" which were made by splitting a 7 I 20. The tees were shop welded to the column and the outstanding legs of the tees were provided with erection bolt holes to hold the beams in place while the field fillet welds were made.

The nominal design reaction was 50 kips for W11 and 14 kips for W12. The tests provide a comparison of the behavior of this type of connection for widely varying tee lengths.

Individual Test Results

Test W11.—The specimen is detailed in Fig. 34, photographed after the moment-rotation test in Fig. 36, and

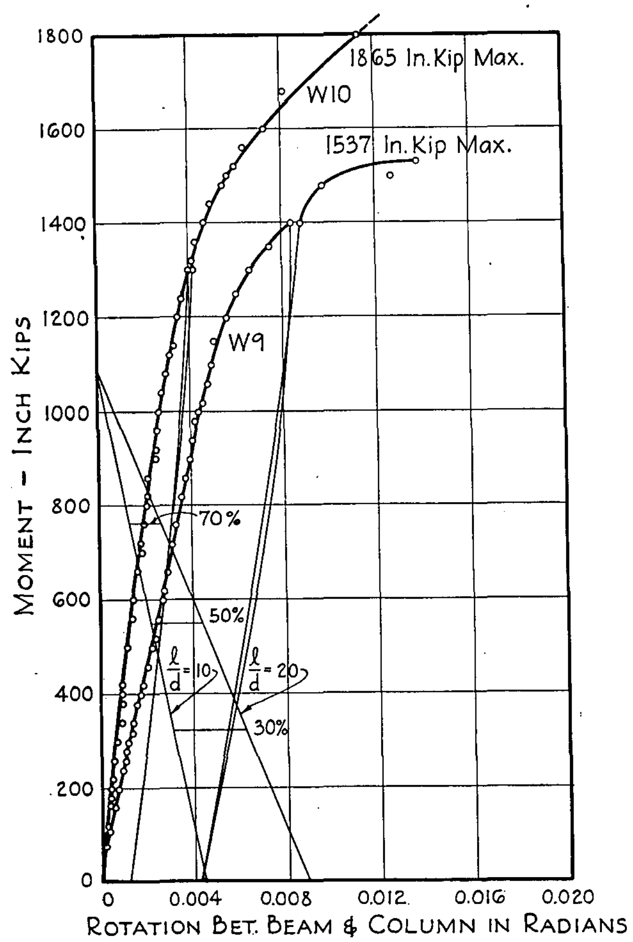


Fig. 32—Moment Rotation Curves for Tests W9 and W10

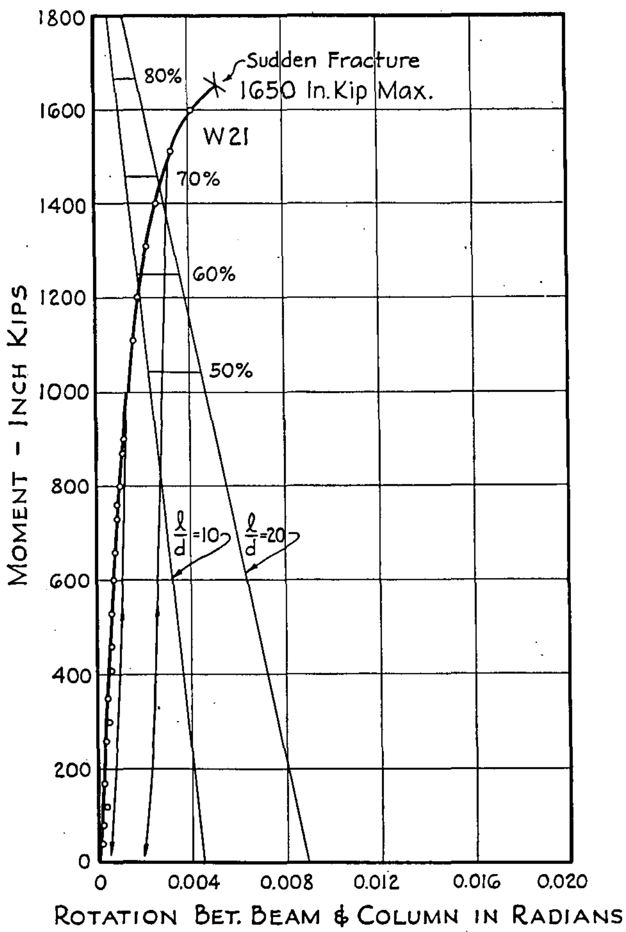


Fig. 33—Moment Rotation Curve, Test W21

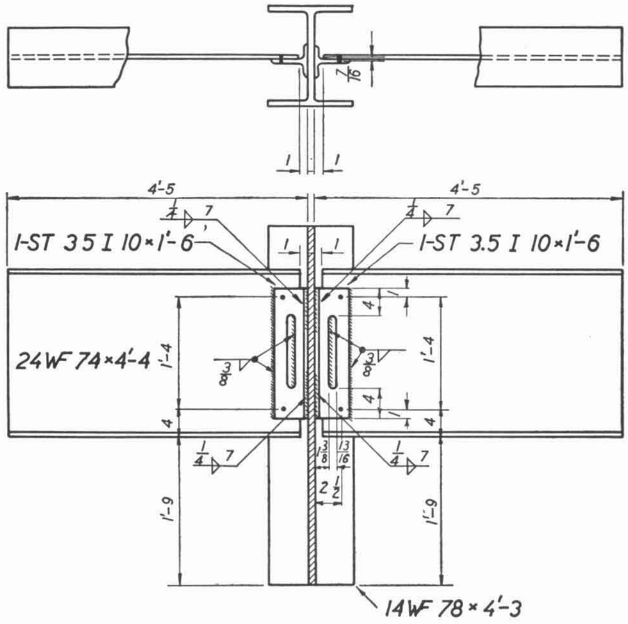


Fig. 34—Test W11

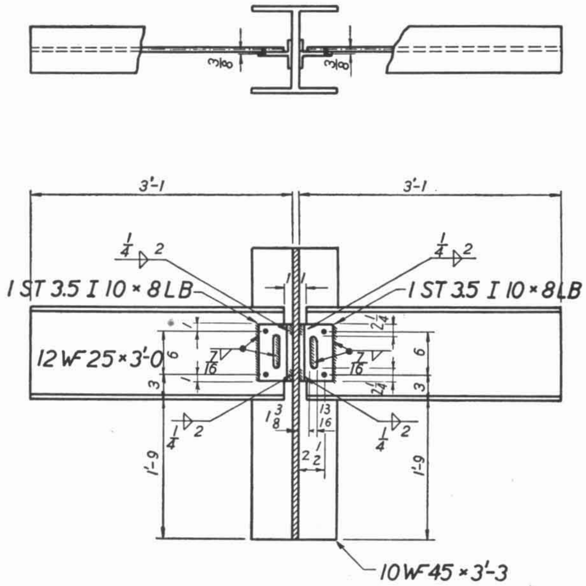


Fig. 35—Test W12

the moment-rotation curve is plotted in Fig. 38. The erection bolts were loosened prior to test.

The connection exhibits a fair amount of reserve moment strength and rotation flexibility beyond design requirements before the tear of any load-carrying weld. Failure of erection tack welds near the juncture of the tee at relatively low loads interrupted the smoothness of the moment-rotation curve.

The slot for fillet weld and corresponding distribution of metal in the outstanding leg of the tee is of poor design as there is no medium other than a narrow strip of tee through which stress supposedly developed by the fillet weld can be transmitted. The fillet welds between the tee and the column should have been larger to more effectively develop the bending strength of the tee flange.

At $V = 26$ kips and $M = 520$ kip-in., first yielding due to bending of the tee flanges was noted and tack welds failed at $M = 600$ kip-in. and 740 kip-in. Aside from the tack weld failures, continued yielding proceeded in a very gradual and uniform manner.

At $V = 40.5$ kips and $M = 810$ kip-in., the entire, slightly undersized, southeast top weld failed suddenly. All other welds on the connection were intact except the northeast which was cracked for a distance of one inch.

In the supplementary shear test a reaction of 168 kips was applied to the west connection with no visible sign of failure in the connection. The load was stopped because of beam web crippling under the load-bearing block.

Test W12.—The specimen is detailed in Fig. 35, photographed after the moment-rotation test in Fig. 37, and the moment-rotation curve is plotted in Fig. 38.

The general behavior of the connection during test was similar to W11, with somewhat more reserve beyond design requirements both with respect to moment strength and flexibility. The tee-to-column welds were too small to develop the inherent restraint of the connection although the welds were larger than required for shear.

At $V = 11$ kips and $M = 77$ kip-in., a strain line was first noted near the top corners of the tee; and at $V = 15$ kips and $M = 105$ kip-in., strain lines developed in the bottom fillet of the tee. Progressive yielding of the tee flange continued until at $V = 23$ kips and $M = 161$ kip-in. the southeast top tee-to-column weld fractured suddenly. The erection bolts in the connection remained loose throughout the test.

In the supplementary shear test a reaction of 56 kips was sustained after which the load fell off due to twisting of the beam and tee leg.

Group IV—Design Details and Test Results

Design Details

The four connections in this group, W13 to W16, inclusive, are all of the same type in that a single clip angle is shop welded to the beam web as shown in detailed drawings, Figs. 40 to 43, inclusive. The outstanding leg of the clip angle is provided with erection bolt holes for temporary connection to girder or column. After erection bolts are in place the beam web adjacent to the clip angle is butt welded to the girder or column, the butt weld being laid over the shop fillet weld. The connection angle serves as a backing strip for the butt weld. In the test specimens the connection was made to the

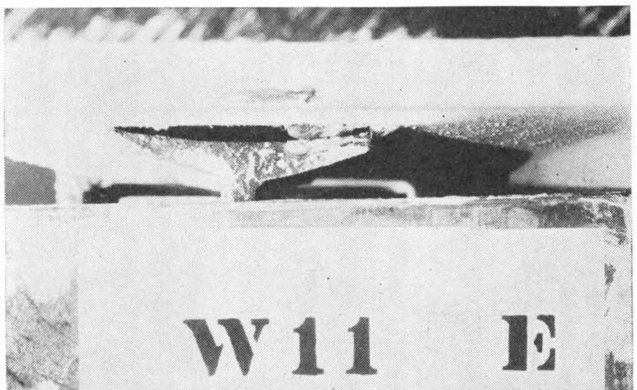


Fig. 36—Test W11

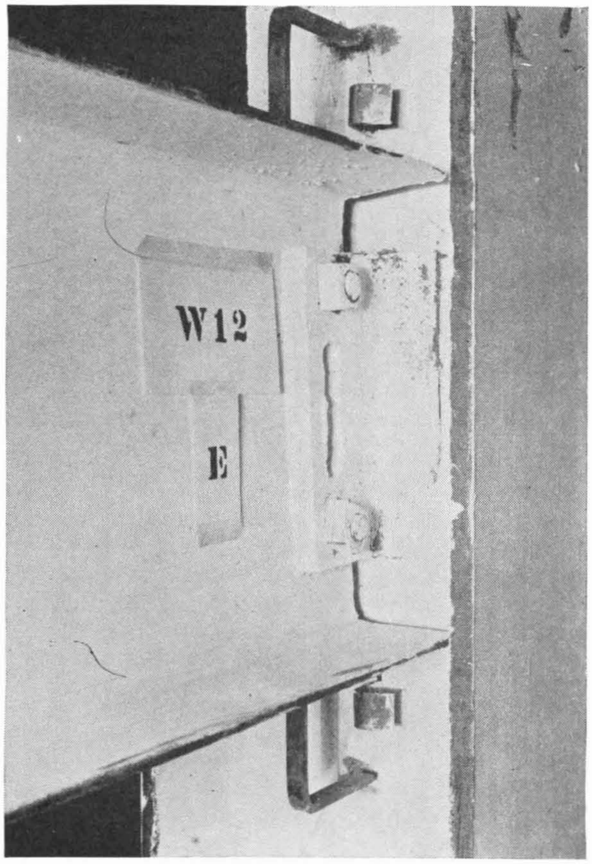


Fig. 37—Test W12

center of the column flange and the flexibility of the adjacent column web is probably sufficient so that the connection behavior would be somewhat stiffer if welded directly to a column or girder web.

The connections in this series were chosen so as to use a heavy and light 18-in. WF beam, and a heavy and light 12-in. WF beam, with nominal design reactions of 80, 24, 30 and 14 kips, respectively.

The erection bolts were loosened for all tests of this group.

Individual Test Results

Test W13.—The specimen is detailed in Fig. 40, photographed after the moment-rotation test in Fig. 44 and the moment-rotation curve is plotted in Fig. 48. Definite yield and initial weld crack occurred before the design requirement for $l/d = 20$ was reached and the maximum moment was reached shortly after with little reserve in moment strength or rotation. Initial premature failure of this connection was caused by a slag inclusion in the butt weld.

The butt weld on the west connection was improperly made, with a patched appearance, and the east weld was placed with a setback of $^5/_{16}$ in. instead of $^1/_2$ in. as detailed. The crowding of the butt weld may have been partly responsible for the slag inclusion, which was about $1^1/_2$ in. long and was not apparent in a careful visual inspection prior to testing.

At $V = 28$ kips and $M = 336$ kip-in., initial yielding was noted in the beam web near the bottom of the butt weld. The yielding in the beam web gradually spread parallel with the weld until, at $V = 62$ kips and $M = 744$ kip-in., the strains lines were about two-thirds the length of the weld. The east butt weld cracked slightly

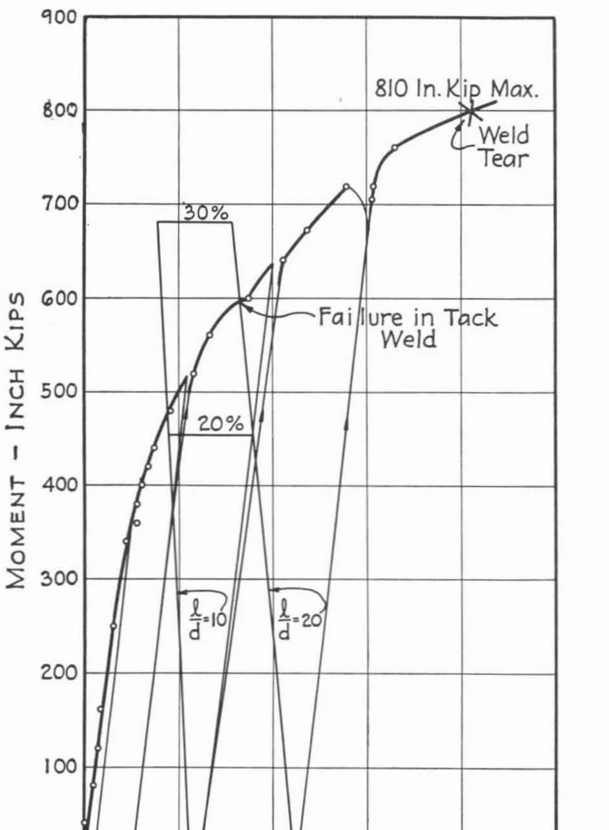


Fig. 38—Moment Rotation Curve, Test W11

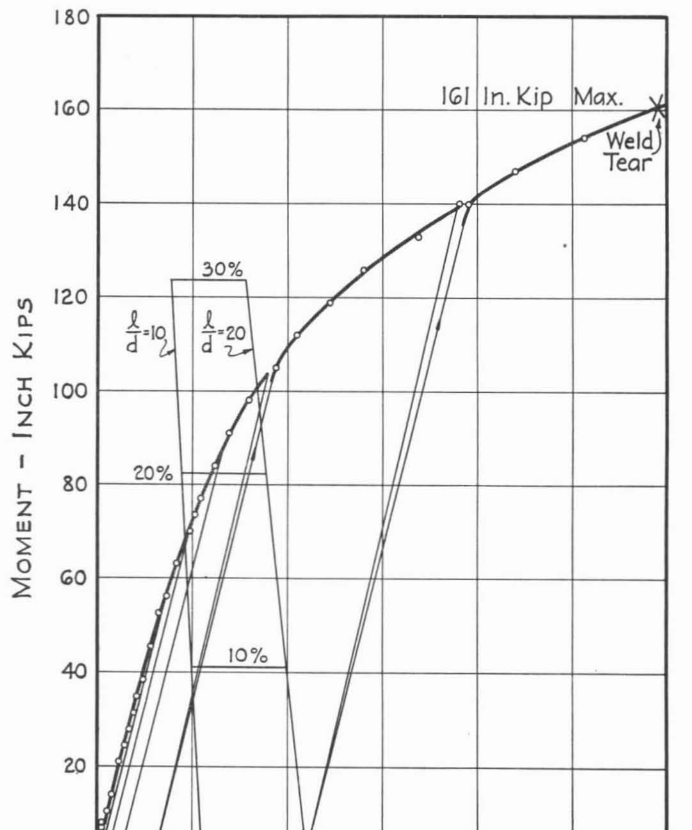


Fig. 39—Moment Rotation Curve, Test W12

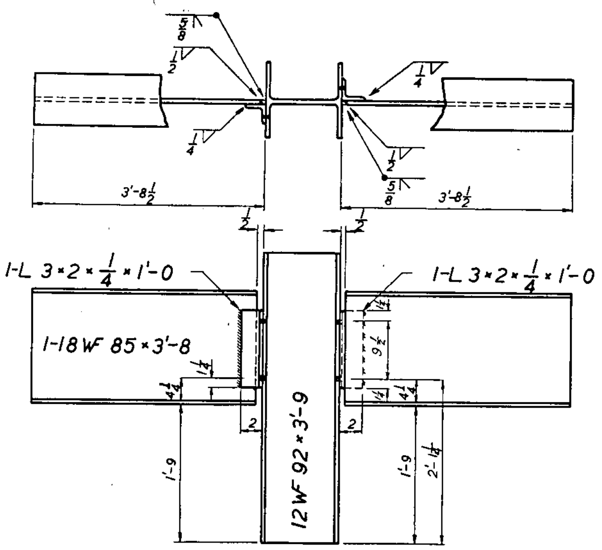


Fig. 40—Test W13

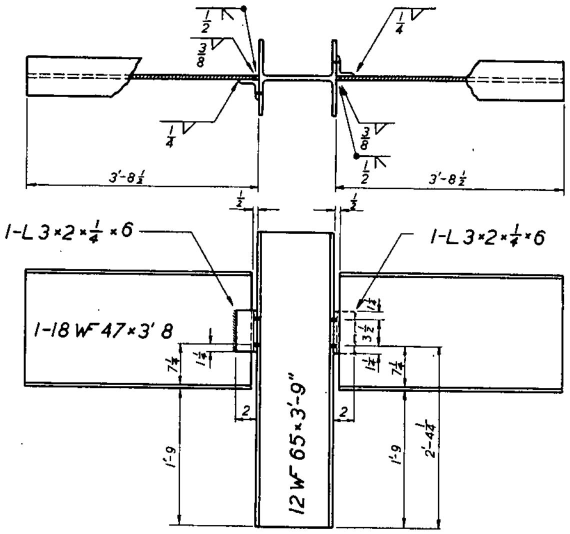


Fig. 41—Test W14

at V = 64 kips and M = 768 kip-in. The ultimate load of V = 65.5 kips and M = 786 kip-in. was reached as the weld cracked through the $1^1/_2$-in. slag inclusion. No supplementary shear test was run because the beams had been used in connection W21.

Test W14.—The specimen is detailed in Fig. 41, photographed after the moment-rotation test in Fig. 45, and the moment-rotation curve is plotted in Fig. 49. Gradual yielding was noted before the design requirement for l/d = 20 was reached and the maximum moment was reached shortly after with little reserve moment strength or rotation.

Part of the flexibility of this connection seems to come from the column web as is indicated by the considerable yielding in the column web. The beam web underwent only light yielding, hence little flexibility is added from this source.

Initial yielding was first noted at V = 20 kips and M = 160 kip-in. in the column web adjacent to the bottom of the weld. At V = 30 kips and M = 240 kip-in., the column web had yielded for the full length of the weld. The beam leg of the erection angle showed signs of yielding shortly before the ultimate load. The ultimate load was reached at V = 32.5 kips and M = 260 kip-in., when the east weld cracked at the top and tore gradually. When the weld was torn about three inches the test was stopped.

In the supplementary shear test a maximum reaction of 117 kips was applied with no visible failure in the welds. The test was stopped because of beam web crippling under the bearing blocks.

Test W15.—The specimen is detailed in Fig. 42, photographed after the moment-rotation test in Fig. 46, and the moment-rotation curve is plotted in Fig. 50.

Gradual yielding was noted before the design requirement of l/d = 20 was reached, but the connection showed some reserve moment strength and satisfactory reserve rotation.

Part of the flexibility of this connection comes from the column web as is indicated by the heavy yielding of the column web. Unlike W14 this connection also showed heavy yielding in the beam webs.

Yielding was first noted in the beam web and column web adjacent to the bottom of the butt weld at V = 24 kips and M = 168 kip-in. Continued loading to V = 30 kips and M = 210 kip-in. caused yielding of the beam web and column web adjacent to the top of the weld. General yielding of both column and beam webs continued to V = 37 kips and M = 259 kip-in. when the east weld cracked at the top. At V = 44 kips and M =

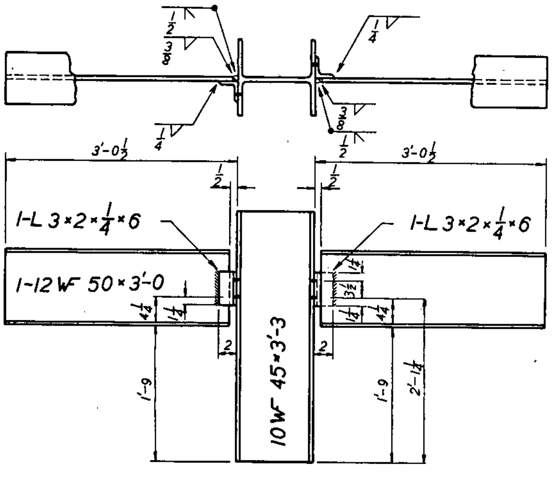


Fig. 42—Test W15

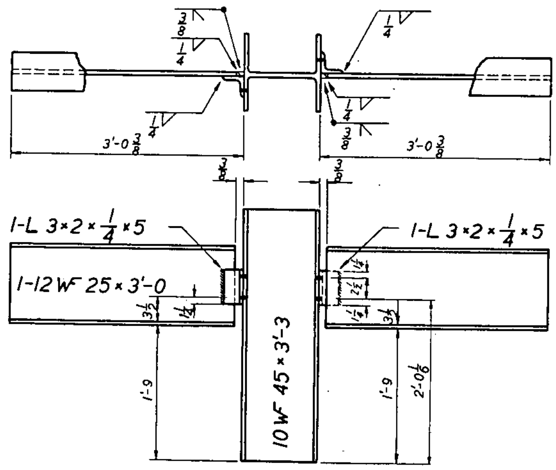


Fig. 43—Test W16

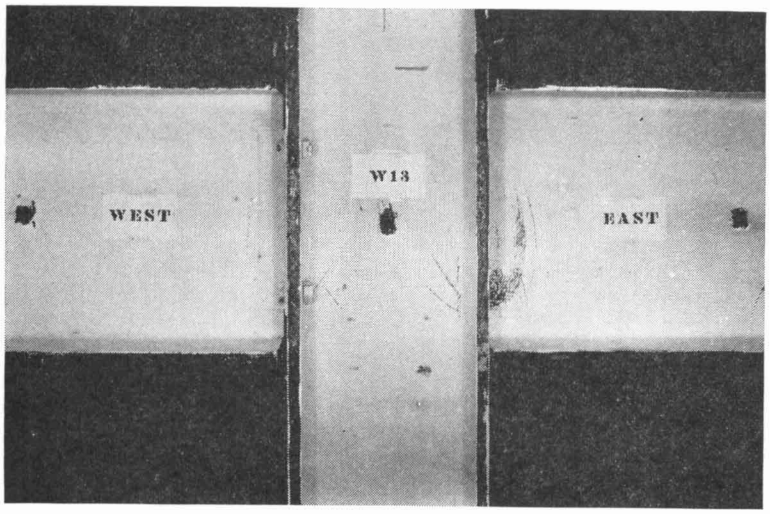


Fig. 47—Test W13

308 kip-in., the east connection broke suddenly through the top of the weld.

In the supplementary shear test a maximum reaction of 120 kips was applied to the west connection with no visible failure in the welds. The test was stopped because of beam web crippling under the bearing blocks.

Test W16.—The specimen is detailed in Fig. 43, photographed after the moment-rotation test in Fig. 47, and the moment-rotation curve is plotted in Fig. 51.

Slight yield was noted before the design requirement for $l/d = 20$ was reached and the maximum moment was reached shortly after with little reserve moment strength but with satisfactory reserve rotation.

The centers of the butt welds were placed $1^1/_4$ in. below the centerline of the beam webs. This fabrication error shifted the center of rotation toward the bottom of the beam but had no apparent effect on the connection's strength.

Part of the flexibility of this connection is caused by the slight localized yielding of the column web.

First yielding was noticed at $V = 18$ kips and $M = 126$ kip-in., in the column web adjacent to the weld. The beam web showed slight signs of yielding near the welds at $V = 20$ kips and $M = 140$ kip-in. The maximum load was reached at $V = 21.7$ kips and $M = 151.9$ kip-in., when the last weld fractured at the top.

In the supplementary shear test a reaction of 67 kips was applied to the west connection with no visible failure in the welds. The test was stopped because of beam web crippling under the bearing blocks.

Group V—Design Details and Test Results

Design Details

The four connections in this group, W17 to W20, inclusive, are all spandrel types as shown in the detailed drawings in Figs. 52 to 55, inclusive. The first three

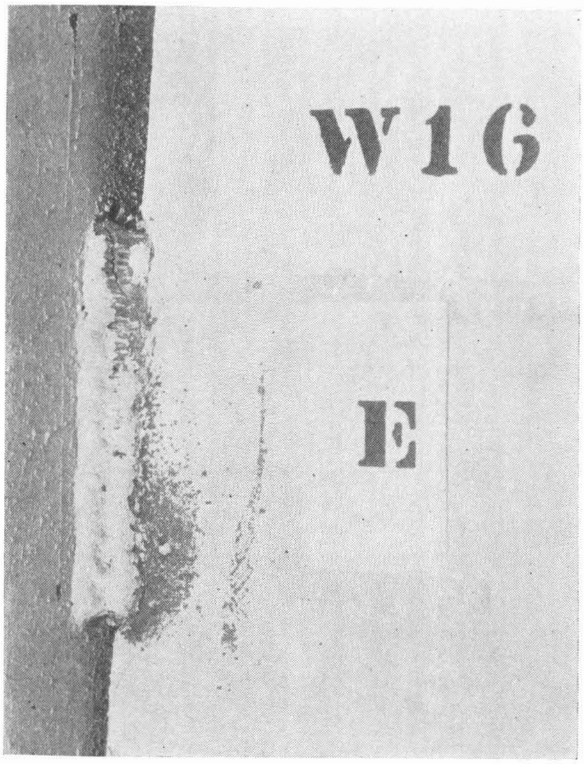


Fig. 47—Test W16

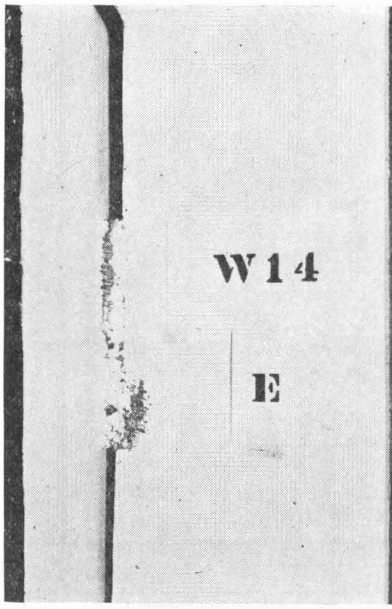


Fig. 45—Test W14

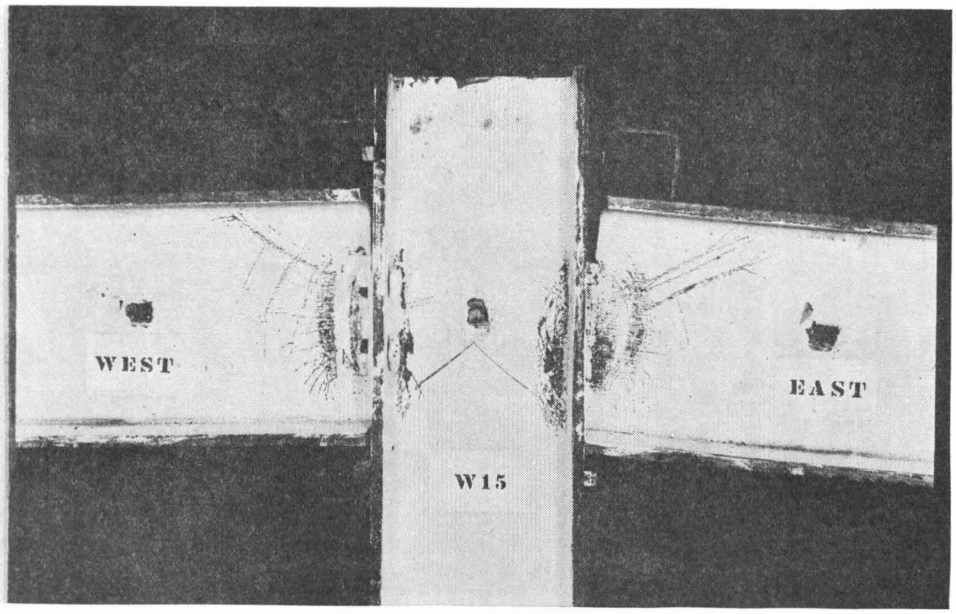


Fig. 46—Test W15

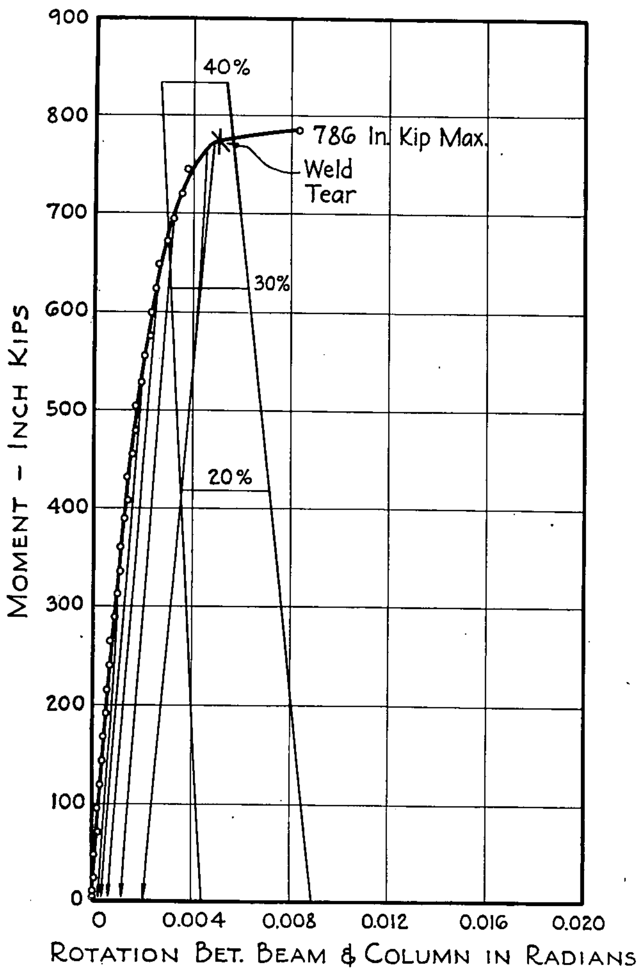


Fig. 48—Moment Rotation Curve for W13

connections are similar types with the beam web connected to the column flange by means of plates. The fourth connection, W20, has a seat angle for vertical reaction and a top angle for lateral support.

General Test Data

All connections in this group were restrained from lateral displacement and rotation by the use of two "outriggers" or flexible ties, one at the top and bottom flange of each beam as shown in Fig. 3. These lateral test supports may be considered to offer less lateral support than provided in actual building construction by fireproofing, floorslab and other framing beams.

The erection bolts provided in the connections were loosened during the test.

General Test Results

The erection bolt holes are too close to the edge of the plate, as W17 and W18 both failed by tearing through the top bolt hole. The hole is a "stress-raiser" and is placed in a portion of the plate subjected to high bending stresses.

Individual Test Results

Test W17.—The specimen is detailed in Fig. 52, photographed after the moment-rotation test in Fig. 56, and the moment-rotation curve is plotted in Fig. 60 along with W18 and W19.

Definite yield was noted before the design requirement for $l/d = 20$ was reached; however, the connection has fair reserve moment strength and a very large reserve rotation.

Initial yielding appeared in the connection plate in the area between the bolt holes at $V = 34$ kips and $M = 408$ kip-in. Continued loading to $V = 36$ kips and $M = 432$ kip-in. caused the bottom of the plate to yield generally and then to cripple in the free distance. Crippling was aided by a slight initial curvature in the plate. At the same load the top of the plate began to yield between the bolt hole and the free distance. Additional loading caused the top of the east plate to neck down in the free distance and at $V = 44$ kips and $M = 528$ kip-in. the plate began to tear at the top toward the top erection bolt hole. The maximum load was reached at $V = 44.5$ kips and $M = 534$ kip-in. when the east plate had torn to the top bolt hole. The test was stopped when the plate had torn about seven inches. The welds adjacent to the free distance had cracked slightly owing to the large deformations of the plate. In the supplementary shear test a reaction of 109 kips was applied to the west connection with no visible weld failure. The test was stopped because of beam web crippling.

Test W18.—The specimen is detailed in Fig. 53, photographed after the moment-rotation test in Fig. 57, and the moment-rotation curve is plotted in Fig. 60 along with W17 and W19.

Definite yield occurred before the design requirement for $l/d = 20$ was reached; however, the connection had fair reserve strength and a very large reserve rotation.

The first yielding was noted in the area of the connecting plate between the bolt holes at $V = 37.5$ kips and $M = 450$ kip-in. General yielding continued in the beam side of the connection plate until $V = 40$ kips and $M = 480$ kip-in., when the bottom of the plate began to crush in the free distance. Simultaneously, the top of the last plate began to yield between the bolt hole and the free distance, followed by necking, and finally started

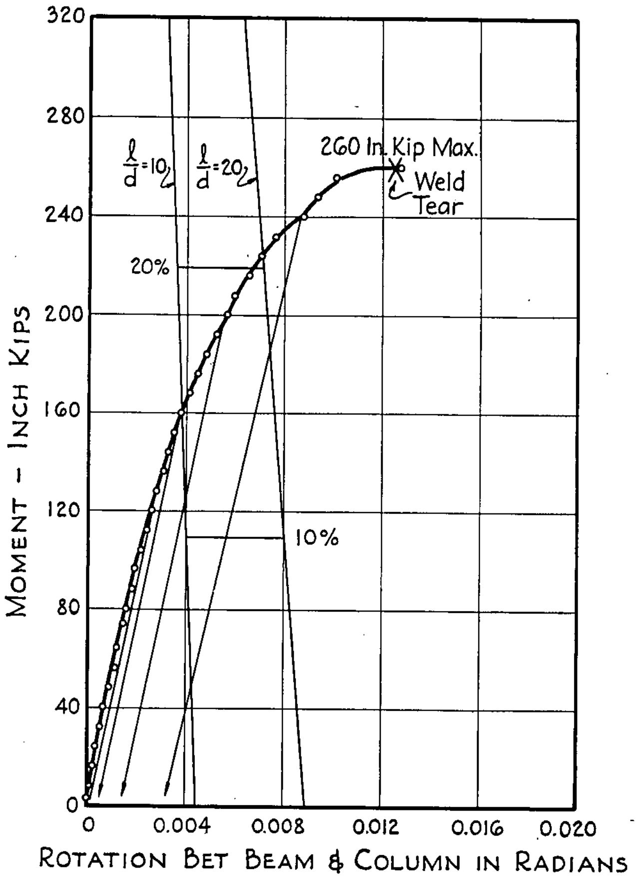


Fig. 49—Moment Rotation Curve for W14

to tear at $V = 47.5$ kips and $M = 540$ kip-in. The bottom of the plates began to cripple, aided by the initial curvature and the eccentricity of load caused by the filler plate. The ultimate load was reached at $V = 51.2$ kips and $M = 614$ kip-in., when the east plate had torn to the bolt hole. The test was stopped after the plate had torn for several inches because the lateral ties were taking load owing to the large beam rotations.

In the supplementary shear test a maximum reaction of 121 kips was applied to the west connection resulting in twisting of the connection plate and beam.

Test W19.—The specimen is detailed in Fig. 54, photographed after the moment-rotation test in Fig. 58, and the moment-rotation curve is plotted in Fig. 60 along with W17 and W18.

Definite yield was noted before the design requirement for $l/d = 20$ was reached; however, the connection has fair reserve strength and a very large reserve rotation.

Initial yielding was first noted at the bottom of the connection plate in the free distance at $V = 17$ kips and $M = 204$ kip-in. The top of the plate showed signs of yielding in the free distance at $V = 21$ kips and $M = 252$ kip-in. The top plate was yielding in the area between the top bolt hole and the free distance, and also between the bolt holes. Owing to the tension in the top of the plate there was a tendency to pull the top of beam web into a straight line with a column flange, and to cripple the bottom of the plate tending to pull the beam web in the opposite direction. The beam is therefore subjected to a torsional rotation. At $V = 24$ kips and $M = 288$ kip-in., the beam was about $^3/_4$ in. out of a vertical line. The west top plate was pulled straight, showing signs of necking, and finally began to tear vertically at $V = 25$ kips and $M = 378$ kip-in. The bottom of the plate was bent in to a right angle and the west beam bottom flange began to tear on the column. The load increased slowly until the east beam flange came into bearing on the column at $V = 31.5$ kips and $M = 378$ kip-in., increasing the stiffness of the connection. The top plates continued to tear until $V = 40$ kips and $M = 480$ kip-in., when the test was stopped because the lateral tie bars were taking load.

TABLE I

PHYSICAL PROPERTIES OF MATERIALS

	TENSILE NO.	SPEC. NOS.	SECTION	LOCATION	Y. P. K/SQ. IN	ULT. K/SQ. IN	8" ELONG %	2" ELONG %
CONNECTION MATERIAL	A 1	W5, W6	2" BAR		47.1	60.4	28.6	48.0
	A 2	W7, 8, 9, 10	L 6 x $3\frac{1}{2}$ x $\frac{5}{8}$	$3\frac{1}{2}$ LEG	43.3	63.3	26.4	49.5
	A 3	W 20	L 5 x 5 x $\frac{5}{8}$	LEG	35.8	59.7+	31.9	55.0
	A 4	W 8	L $2\frac{1}{2}$ x 2 x $\frac{5}{16}$	$2\frac{1}{2}$ LEG	45.4	67.8	26.0	43.0
	A 5	W 13, 14, 15, 16	L 3 x 2 x $\frac{1}{4}$	3 LEG				
	A 6	W 20	L 3 x $2\frac{1}{2}$ x $\frac{3}{8}$	$2\frac{1}{2}$ LEG	41.8	67.3	25.3	41.5
	A 7	W 11, 12	ST 3.5 I 10	WEB	39.7	69.8	26.9	47.0
	A 8	W-7	PL 3 x $\frac{5}{16}$		41.5	61.7	30.2	48.0
	A 9	W-9, 10	PL 6 x $\frac{5}{16}$		41.8	61.2	32.2	50.5
	A 10	W 17	PL $3\frac{1}{4}$ x 1					
	A 11	W 17, 18, 19	PL 12 x $\frac{5}{16}$		42.9	61.8	26.5	43.5
	A 12	W 21	PL $\frac{5}{16}$ x 6		50.7	67.5	24.4	42.5
	A 13	W 21	PL $\frac{5}{8}$ x 12		37.3	56.2+	28.1	49.0
	B 1	W 3	18 WF 85	WEB LONG	38.1	65.5	29.4	44.0
COLUMNS	C 1		10 WF 45	FLANGE	36.7	61.4	30.6	56.5
	C 2W		12 WF 65	WEB	39.1	61.2	30.0	47.5
	C 2F		12 WF 65	FLANGE	37.7	60.7	30.9	54.0
	C 3		12 WF 92	FLANGE	38.1	64.3	30.5	54.5
	C 4		14 WF 78	WEB	39.4	60.9	26.6	41.0
WELD METAL	D 1	W 1 - W 21	LAB COUPON	VERTICAL WELDS	57.1	74.0		15.0*
	D 2	W 7 - W 21	LAB COUPON	FLAT WELD	56.0	69.0		30.0

* SUB-SPECIFICATION ELONGATION CAUSED BY SLAG INCLUSIONS

\+ SUB-SPECIFICATION TENSILE STRENGTH

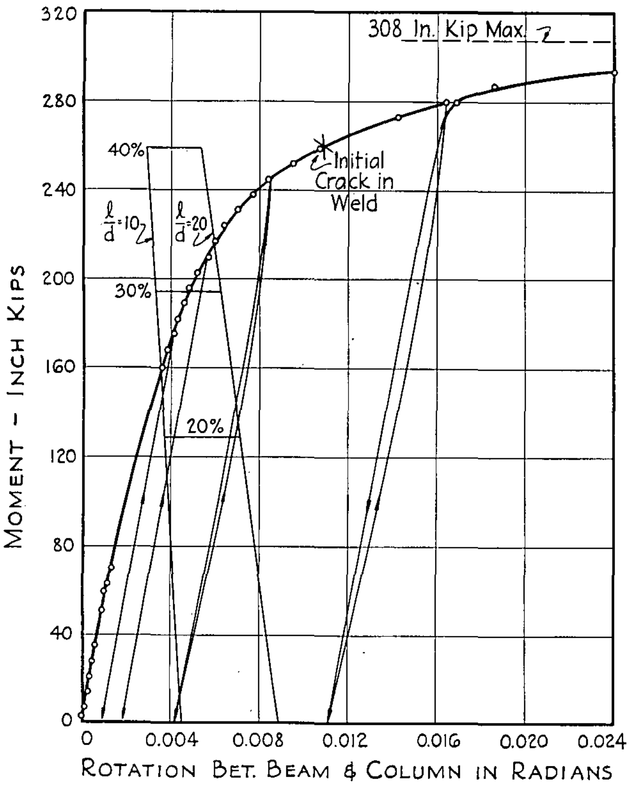


Fig. 50—Moment Rotation Curve for W15

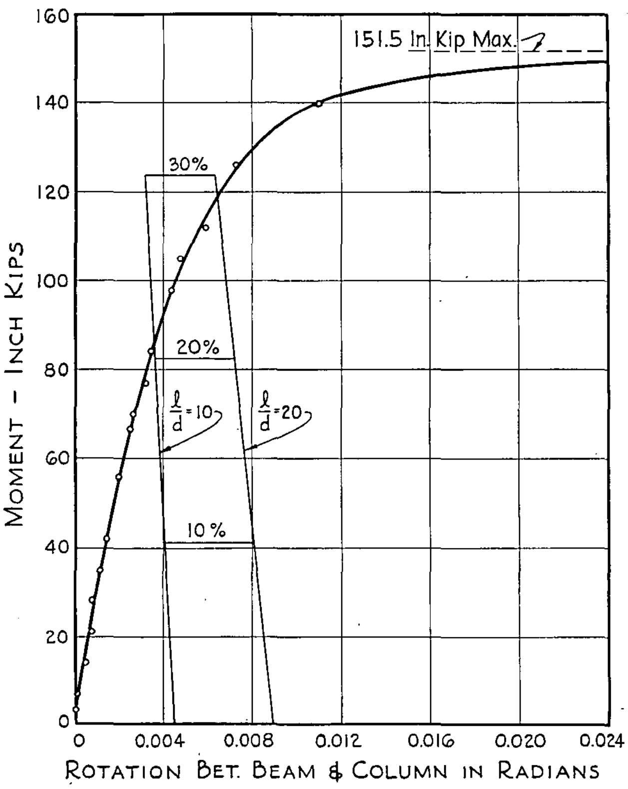


Fig. 51—Moment Rotation Curve for W16

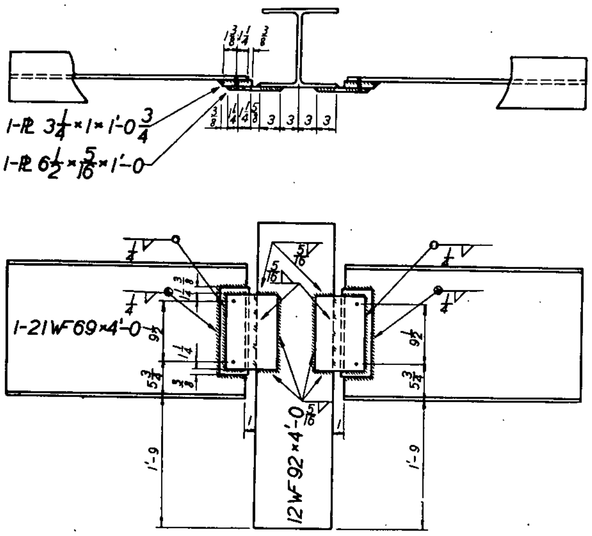


Fig. 52—Test W17

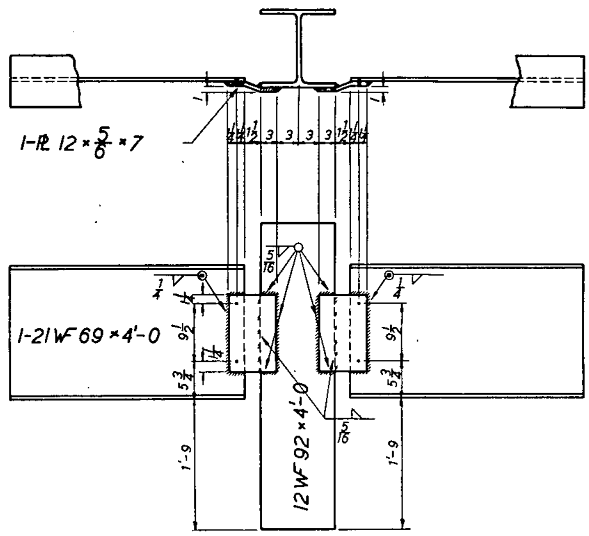


Fig. 54—Test W19

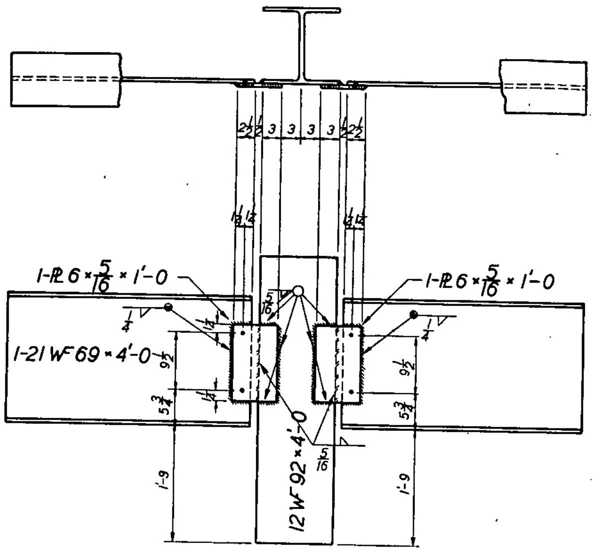


Fig. 53—Test W18

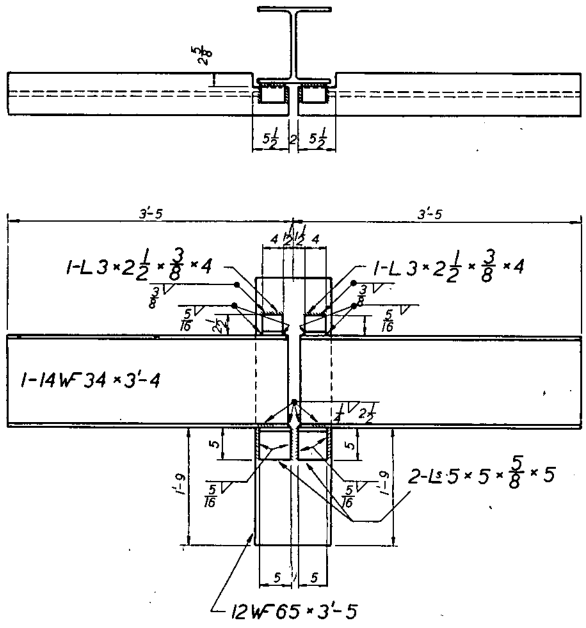


Fig. 55—Test W20

The supplementary reaction test for this connection could not be run because the beam was twisted out of line with the column.

Test W20.—The specimen is detailed in Fig. 55, photographed after the moment-rotation test in Fig. 59, and the moment-rotation curve is plotted in Fig. 61. Slight yield was noted before the design requirement for l/d = 20 was reached and the connection has only slight reserve strength; however, it has a large reserve rotation.

The east connection had poor bearing on the seat angle owing to the slope of the flanges. The flange was not at right angles to the web and a gap of $^1/_8$ in. was left between the beam flange near the column and the seat angle. This caused the eccentricity of the load to be considerably greater than assumed in the design computations; hence, the seat angle may have been overstressed.

The top east angle showed the first sign of yielding at V = 16 kips and M = 200 kip-in. The beam flanges adjacent to the east top angle showed signs of yielding, probably caused by the top angle taking vertical reaction owing to the vertical movement of the beam as it closed the gap left by the sloping flanges. At V = 26 kips and M = 325 kip-in., the beam webs began to yield over the edge of the seat angle followed at V = 34 kips and M = 425 kip-in., by yielding in the outstanding leg of the seat angle. The top angle deformed visibly until V = 34.75 kips and M = 452 kip-in., when the weld on the vertical leg of the west top angle suddenly sheared off.

In the supplementary shear test a reaction of 62 kips was applied with failure occurring by tearing of the seat-angle welds and crippling of the beam web.

Summary of Test Results

Table I provides a summary of physical properties of most of the connection material and several check tests on beam and column material. Nearly all of the material passed the A. S. T. M. Specification A-7-39 for Building and Bridge Steel, with regard to yield point, ultimate and elongation properties. The two tests of

Table II
Summary of Test Results

Group	Test No.	Beam Size (WF)	Column Size Connection to Flange or Web	Reaction: Nominal Design	Reaction: Actual Design Value	Reaction: Max. Reaction in Moment Test	Reaction: Max. React. in Suppl. Shear Test	Reaction: Load Reaction Factor of Safety	Rotation Design Requirement for Connection to Beam Loaded Uniformly with Max. Design Stress=20K.s.i.: l/d for Balanced Reaction and Mom.	ϕ_o Test ϕ for Bal. R. and M. Design	Test ϕ for Beam Des. l/d=20	ϕ_Y Rot. Test ϕ for general yield	ϕ_T Rot. Test ϕ Initial Tear-Weld or Plate	ϕ_u Test ϕ at Ultimate Rotation	Rotation Factor Safety: Yield ϕ_Y/ϕ_D	Rotation Factor Safety: Tear ϕ_T/ϕ_D	Rotation Factor Safety: Ult. ϕ_u/ϕ_D	Proportion of Beam Depth from top flange to center of rotation at design ϕ_o
Ia	W1	18-85	12WF65web	72.0	75.3	159.9	200	2.66	9.06	0.0027	0.0056	0.0031	0.0111	0.021+	1.15	4.11	7.78+	0.56
	W3	do	do	72.0	75.3	117.5	188	2.50	9.06	0.0030	0.0060	0.0026	0.0202	0.029+	None	6.73	9.65+	0.65
	W5	do	do	72.0	82.0	75.0	170	2.07	8.32	0.0032	0.0059	Indefinite	0.0080	0.008+	Indef.	2.50	2.50+	0.43
Ib	W2	18-47	do	24.0	25.0	117.5	—	4.71	14.7	0.0055	0.0074	0.0046	0.0191	0.019+	None	3.47	3.46+	0.59
	W4	do	do	24.0	25.0	36.5	110	4.58	14.7	0.0054	0.0073	0.0016	0.0092	0.012+	None	1.70	2.22+	–
	W6	do	do	24.0	26.0	25.7	127	4.88	14.1	0.0054	0.0075	Indef.	0.0120	0.012+	Indef.	2.22	2.22+	0.48
IIa	W7	18-47	12WF65flange	24.0	24.0	16.7	89	3.71	15.3	0.0047	0.0061	0.0040	0.0040	0.005+	None	None	1.07+	0.85
	W8	do	do	24.0	24.0	18.5	81	3.37	15.3	0.0059	0.0077	0.0024	0.0024	0.020+	None	None	3.40+	0.84
IIb	W9	do	do	24.0	24.0	76.9	—	3.20	15.3	0.0029	0.0032	0.0060	No failure	0.013+	2.07	—	4.48+	0.34
	W10	do	do	24.0	24.0	93.2	—	3.88	15.3	0.0020	0.0022	0.0048	0.0120	0.012+	2.40	6.00	6.00+	0.42
	W21	18-85	12WF92web	72.0	72.0	82.5	153	2.12	9.48	0.0018	0.0028	Indef.	0.0052	0.005+	Indef.	2.89	2.80+	0.67
III	W11	24-74	14WF78web	52.0	52.0	40.5	168	3.23	11.0	0.0038	0.0066	Indef.	0.0164	0.016+	Indef.	4.30	4.30+	0.50
	W12	12-25	10WF45web	14.0	19.2	23.0	56	2.92	10.8	0.0039	0.0067	Indef.	0.0250	0.025+	Indef.	6.41	6.41+	0.56
IV	W13	18-85	12WF92flange	80.0	82.0	65.5	Not Available		8.3	0.0027	0.0056	0.0045	0.0050	0.0083	1.67	1.85	3.08	0.53
	W14	18-47	12WF65 do	24.0	27.3	32.5	117	4.29	13.5	0.0050	0.0071	Indef.	0.0128	0.0156	Indef.	2.56	3.12	0.50
	W15	12-50	10WF45 do	30.0	29.0	44.0	120	4.00	14.7	0.0047	0.0060	0.0080	0.0108	0.0319	1.70	2.30	6.80	0.54
	W16	12-25	do do	14.0	15.6	21.7	67	4.30	14.6	0.0049	0.0064	0.0090	0.0327	0.0327	1.84	6.68	6.68	0.59
V	W17	21-69	12WF92 Spandrel	40.0	41.0	44.5	109	2.66	11.1	0.0040	0.0068	0.0082	0.0162	0.0325	2.05	4.05	8.12	0.49
	W18	do	do do	40.0	41.0	51.2	121	2.95	11.1	0.0038	0.0065	0.0057	0.0188	0.0296+	1.50	4.95	7.80	0.52
	W19	do	do do	40.0	41.0	40.0	Could not test		11.1	0.0045	0.0078	Indef.	0.0420	0.042+	Indef.	9.35	9.35+	0.45
	W20	14-34	12WF65 do	18.0	18.0	34.2	62	3.44	15.4	0.0034	0.0041	0.0060	0.0081	0.014+	1.76	2.38	4.12+	0.72

weld metal were made with the laboratory D.-C. welder which was used only for specimens in Group I.

A general summary of some of the significant test results is presented in Table II. The wide variety of test specimen types makes it desirable to study individual test results as well as any general summary such as given here. The first two columns in Table II give the Group and Test Numbers for the twenty-one tests and the beam stub sizes are given in column three. The fourth column gives the column stub size and states whether the connection was made to the column web, flange, or as an eccentric spandrel connection.

There are five columns in Table II under the general heading "Reaction." The first column gives the nominal reaction value used as a starting point for design and the second column is an evaluation of the design value as based on actual weld sizes and lengths that were used. The next two columns give the maximum shear which was applied during the test to moment-rotation failure together with the reaction applied to the companion connection in the supplementary shear test. The last column under "Reaction" gives the load factor of safety based on maximum test reaction and actual design reaction. No values are given for tests W13 and W19, which have no factor of safety as based on shear developed in the moment-rotation tests. However W13 was not available for the supplementary shear test and undoubtedly would have proved satisfactory in shear alone, since the companion specimens were quite adequate. Test specimen W19 has no reaction factor of safety as tested. If adequately held by the building material against twisting it presumably would develop the required reaction value for safety.

There are nine columns under the heading "Rotation Design Requirement for Connection to Beam Loaded Uniformly with Maximum Design Stress = 20 k. s. i." The first column gives the l/d which corresponds to the nominal design reaction value and simple beam moment design for a maximum bending stress of 20 k. s. i. This value of l/d easily may be computed from the beam section modulus, beam depth and nominal reaction value, as a result of the following relationships:

$$\text{Design Reaction} = V = \frac{wl}{2}$$

$$\text{Simple Beam Design Moment} = M_S = \frac{wl^2}{8} = 20S$$

hence

$$V = \frac{80S}{l}$$

and

$$\frac{l}{d} = \frac{80S}{Vd}$$

Variations in l/d for any connection test should be applied only to the beam depth used in the test.

The next five columns give relative rotation (φ) between beam and column at five different stages of the connection test, namely:

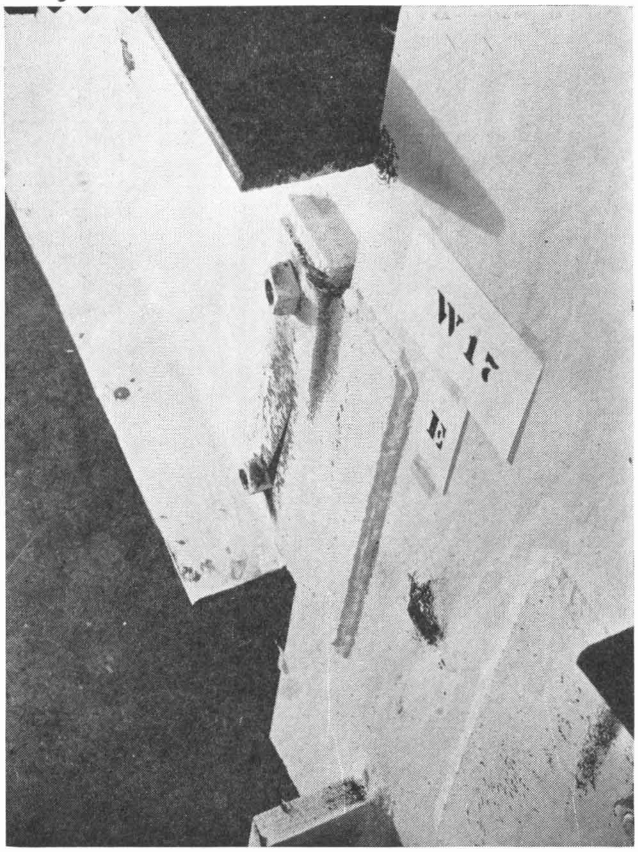


Fig. 56—Test W17

1. $\varphi_D = \varphi$ at l/d of preceding column.
2. φ at $l/d = 20$.
3. $\varphi_Y = \varphi$ at marked general yielding (omitted if indefinite).
4. $\varphi_T = \varphi$ at first definite tear in weld or plate metal (tack weld tear in W11 not considered).
5. φ_U = approximate φ, at ultimate rotation in moment-rotation test.

The next three columns give the rotation ratios or factors of safety as based on φ_D at yield, first tear and ultimate test rotation. In judging the factors of safety the following might be used as a rough guide:

Ratio	Factor of Safety
Less than 1.00	None
1.00 to 1.30	Very poor
1.30 to 1.65	Poor
1.65 to 2.00	Fair
More than 2.00	Good

The final column gives data on the location of center of rotation as determined from the dial gage measurements of horizontal movement at the top and bottom of the beams. The values given are based on a single test and would probably vary somewhat in different tests.

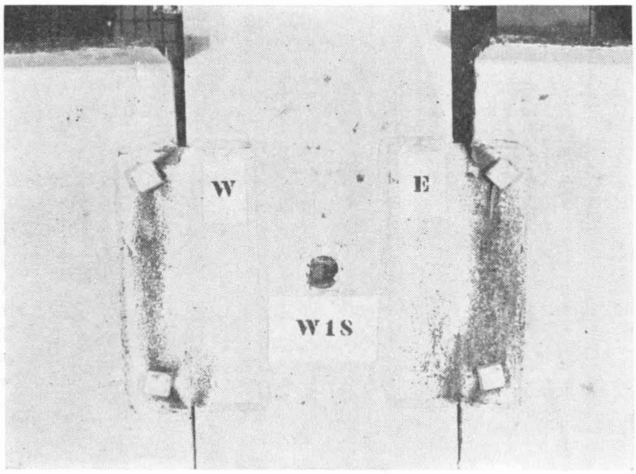


Fig. 57—Test W18

Conclusions

The tests on any one type of connection are so limited in number that complete and final conclusions on all questions are impossible, a fact foreseen in laying out

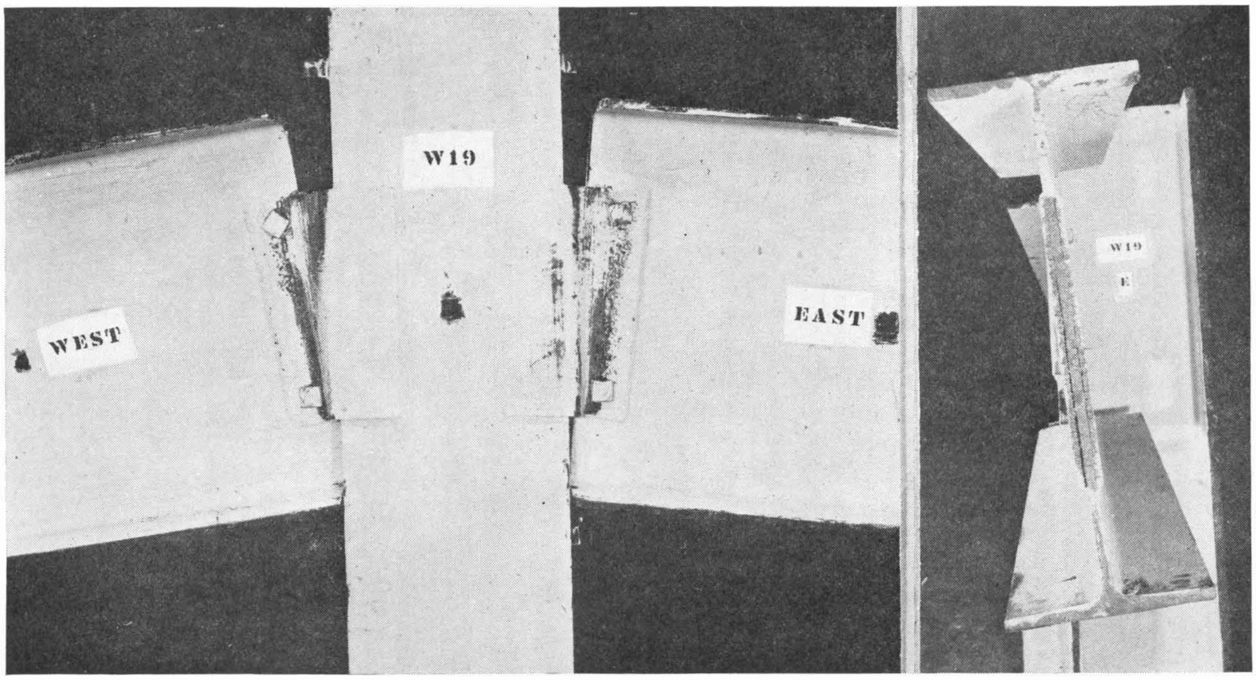


Fig. 58—Test W19

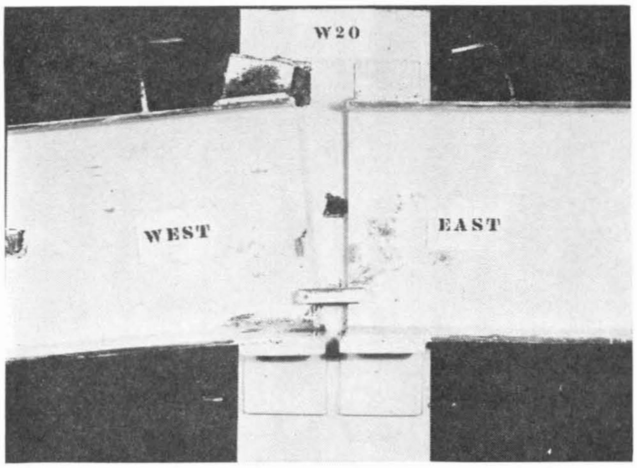


Fig. 59—Test W20

the program. General conclusions which appear obvious will be discussed by groups and this will be followed by a general summary. Reference should also be made to more detailed remarks in the body of the report.

Group I.—The heavy 18WF85 beam connection tests in this group indicated a marked superiority, with respect to ultimate rotation reserve, for direct welding of the beam web to the column, as compared with the use of auxiliary plates as in W5. In the lighter series with 18WF47 beams, there was less difference in the ultimate rotation characteristics. All of the connections in this group showed high stiffness at low loads, but yielded at rotations near or below design requirements. This might be very undesirable if repeated loading and unloading were a design factor.

Group II.—Tests W7 and W8 showed very poor behavior in rotation, being "stiff" and at the same time "weak" with respect to moment. Such connections are not acceptable if weld tear is to be avoided. A favorable feature, however, is the fact that failure of the side lug or plate would not result in collapse, since the supporting seat angle would continue to carry the reaction load.

Tests W9 and W10 showed all-around good behavior, particularly in that the moment-rotation relation was essentially elastic considerably beyond the beam design requirements. This was not true in the case of W21, a similar connection with a heavier beam size.

The satisfactory behavior of the top-plate and seat-angle type of connection in tests W9 and W10 indicates the desirability of further laboratory tests on this type of connection. The complete success of the connection also depends on maintaining a high quality of butt weld between the top plate and column.

Group III.—The results indicate that with modifications of design, as discussed under individual test results, this type of connection has good possibilities for further development.

Group IV.—The tests on this type of connection cover a fairly wide range of beam size. The connections show a very gradual yielding throughout the range of the test In general the behavior of this group is poorer than comparable specimens of similar type, in Group I.

Group V.—These miscellaneous types of spandrel connections showed fairly satisfactory behavior. The crimped plate used in W19 seems objectionable because of the twist that is induced in the beam. Reference should be made to the individual test results.

General Remarks.—As emphasized early in the report, a satisfactory connection must perform two functions:

1. Carry the vertical reaction load with ample safety.
2. Permit the rotation which would be required between the beam and column in an actual frame and at the same time be strong enough to resist the end restraining moment which is developed.

With regard to (1), all of the connections showed an adequate shear factor of safety with respect to the design reaction. Two connections, W13 and W19, are excepted because they could not be tested in the supplementary shear test.

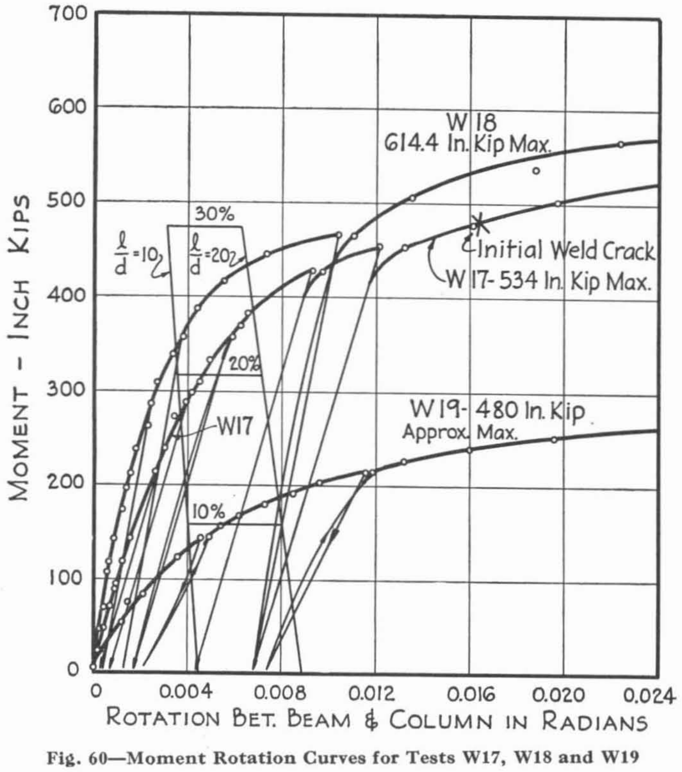


Fig. 60—Moment Rotation Curves for Tests W17, W18 and W19

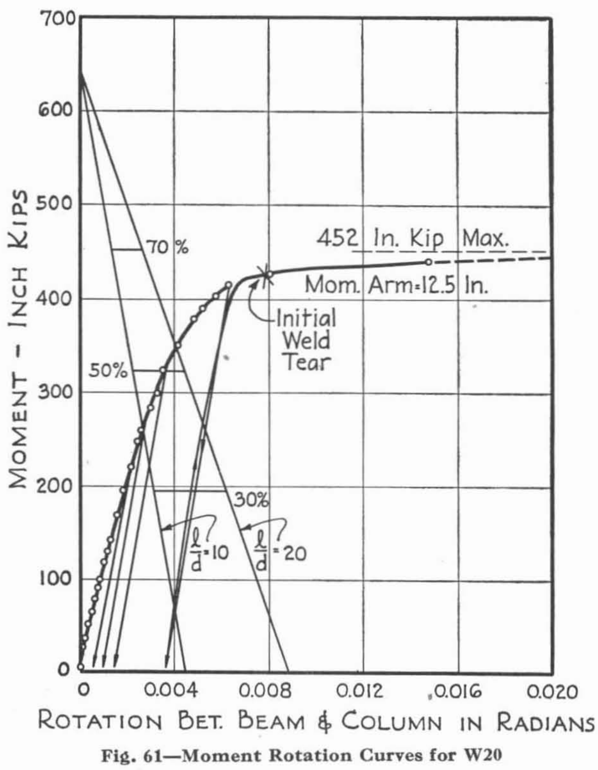


Fig. 61—Moment Rotation Curves for W20

Many of the connections yielded before developing a satisfactory rotation, but prior to initial weld or plate fracture all of the connections except W7 and W8 showed a rotation factor of 1.70 or more as compared with the design requirement for balanced reaction and moment design. Whether or not this rotation reserve is sufficiently safe in all cases may be a debatable question. Rotation failure of connections of the Group I or Group IV type would result in damage to the reaction-carrying welds if not collapse of the connection. Rotation failure of the Group II class, however, would not damage the reaction-carrying welds which are part of the independent seat angle. If it may be argued that the floor slab will be sufficient to support the top of the beam laterally this may be an important distinction.

An incorrect design assumption is frequently made in welded connection design, namely, that the inherent rigidity or end restraint of a connection is negligible. The designer presumes that he can assume that zero moment exists at any convenient point, usually at the center of gravity of a weld group. The welds at the assumed point of zero moment are designed for shear only and all other welds or groups are designed for shear plus the moment produced by the eccentricity of the weld or group from the point of assumed zero moment. Such an assumption is not valid because the restraining moments induced by frame action are maximum and certainly not zero at the connections. Reasonable design procedures may be difficult or impossible, and incorrect procedures may "get by" in some cases by virtue of the ductility of structural steel. Nevertheless, assumptions which are so contrary to fact are certain in some cases to lead to serious trouble. Certain types of connections, supported by seat angles, may have reasonable design assumptions, provided that the top flange or web connection is either flexible or strong enough to develop the tension accompanying the induced moment.

The erroneous assumption discussed in the foregoing paragraph is probably a hangover from riveted design, where the assumption may be relatively less dangerous. In a riveted connection the connecting medium is *less rigid* than the connected material. In a welded connection the weld, if adequately designed, is *more rigid* than the connected material. This is an important distinction between welded and riveted design that should always be considered.

This program of tests has included connections which were considered incorrect design by some engineers, as well as other connections with debatable qualities. Some types of connection, such as the seat and flexible top angle type, have been covered adequately in previous investigations[2] and are not included at all in the present program. Except in the obviously poor designs of W7, W8 and possibly W19, this program of tests should not be considered as providing the final answer to the acceptability of any type of connection. The tests throw light on some incorrect design procedures and may therefore prove of help in developing better designs. More laboratory tests of some of the connection types will undoubtedly be made in the future and these tests may help indicate the direction such programs should take.

References

1. Wilson, Wilbur M., "Tests to Determine the Feasibility of Welding the Steel Frames of Buildings for Complete Continuity," American Welding Society Journal, 15 (1), 28–38 (1936).

2. Johnston, Bruce, and Green, Lloyd F., "Flexible Welded Angle Connections," The Welding Journal, 19 (10), Research Suppl., 402-s to 408-s (1940).

3. Johnston, Bruce, and Hechtman, Robert A., "Design Economy by Connection Restraint," *Engineering News-Record*, 125 (15), 484–487 (Oct. 10, 1940).

4. Lyse, Inge, and Gibson, G. J., "Welded Beam-Column Connections," American Welding Society Journal, 15 (10), 34–40 (1936); and 16 (10) 2–9, Research Suppl., (1937).

5. Johnston, Bruce, and Mount, E. H., "Designing Welded Frames for Continuity," The Welding Journal, 18 (10), Research Suppl., 355-s to 374-s (1939).

6. Batho, C., and Rowan, H. C., "Investigations on Beam and Stanchion Connections" Second Report of the Steel Structures Research Committee, Dept. of Scientific and Industrial Research, Great Britain, pp. 61 to 137.

7. Lyse, Inge, and Schreiner, Norman G., "An Investigation of Welded Seat Angle Connections," American Welding Society Journal, 14 (2), Research Suppl., 1–15 (1935).