1944

Report of tests of welded top-plate and seat building connections

J. L. Brandes

REPORT OF TESTS OF WELDED TOP-PLATE AND SEAT BUILDING CONNECTIONS

by J. L. Brandes°

I. Introduction.

1. Object and Scope of Tests.

In the past few years the question has been raised concerning the proper design of top plates to be used as parts of beam-to-column connections in buildings. Some tests have been made in which the welds were found to be faulty, and in others the particular design of top plate used in the connections did not give satisfactory results.

This series of specimens was designed and tested in the attempt to determine the structural behavior of some promising types of top-plate details. A total of seventeen specimens were tested, eleven of the "semi-rigid" type and six of the "flexible" type*.

Semi-rigid connections are of interest because in the range of fifty to one hundred per cent rigidity they theoretically permit the use of a lighter beam for the same load than either the fully rigid or the flexible connections. Furthermore, semi-rigid connections require lighter welds and less welding than fully rigid connections. The resulting decrease in weight of steel used and in welding required may effect considerable economy in the construction of some types of building.

- -

° American Welding Society Research Fellow
Fritz Engineering Laboratory, Lehigh University,
now with Carnegie Illinois Steel Company.

* For explanation of terms see Section II.

2. Acknowledgment.

This investigation was one of a series which has been sponsored by the American Welding Society in cooperation with Lehigh University and conducted in the Fritz Engineering Laboratory. Committee G on Structural Steel, of the Industrial Division of the Welding Research Committee of the Engineering Foundation, was responsible for general supervision of the work. A special sub-committee consisting of Mr. F. H. Dill of the American Bridge Company, Mr. H. Lawson of the Bethlehem Steel Company, Mr. C. L. Kreidler of the Lehigh Structural Steel Company, and Mr. B. G. Johnston, formerly of Fritz Engineering Laboratory, helped in designing and checking the design of the test specimens.

Special thanks are due to Mr. R. M. Mains, Assistant Director of Fritz Engineering Laboratory, for his assistance in almost every phase of this report. The graduate students and technicians of Fritz Engineering Laboratory rendered valuable aid in the conduct of the tests and compilation of the report.

II. Definition of Special Terms.

For convenience of discussion, the different types of building connections will be divided into three classes: flexible, rigid, and semi-rigid. Precise limits for the different classes can not be established, since one class passes gradually into the other without sharp demarcation.

The existence of the different types should be recognized, however, because the use of an inherently semi-rigid connection in a design which anticipated the use of flexible connections, with no provision made for moments developed at the joints, might lead to disastrous results.

3. "Flexible Connection" Defined.

A "flexible connection" is theoretically possible only if there is no restraint against relative rotation of beam and column. A pin or light angle connection approximates the truly flexible connection as illustrated in Fig. 1(c). Since it is considered to be impractical to use pin or light angle connections in some buildings, other types of connection, such as the top plate and seat, must be used if a flexible connection is desired. For the purpose of this report, any connection which develops beam restraint of less than twenty per cent of the fixed-end moment, thereby permitting eighty per cent or more of the beam rotation required for a theoretically flexible connection, will be called a flexible connection.

4. "Rigid Connection" Defined.

A "rigid connection" theoretically requires full restraint against relative rotation of the beam and column as shown in Fig. 1(a). This condition is impossible of complete realization in many practical cases, but can be rather closely approximated. Therefore, in this report, any connection which develops eighty per cent or more of

?

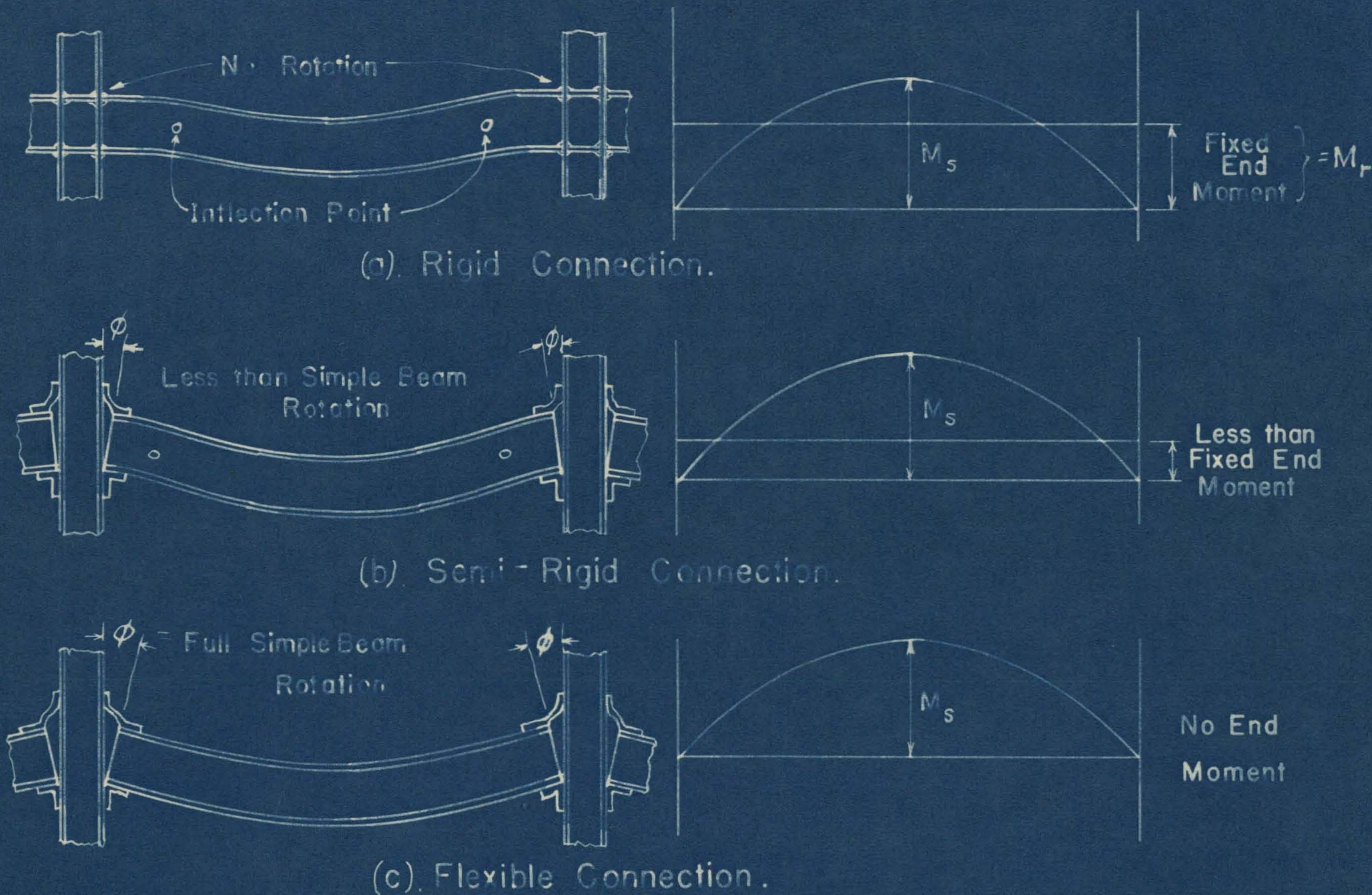


FIG.1. CLASSES OF BEAM-COLUMN CONNECTIONS.

the full fixed-end moment, thereby permitting no more than ten per cent of the beam rotation required for a theoretically flexible connection, will be called a rigid connection.

5. "Semi-Rigid Connection" Defined.

In consideration of the two preceding definitions, the term "semi-rigid connection" would refer to a connection whose behavior lay between the extremes of "flexible" and "rigid". The semi-rigid connection therefore is one capable of carrying from twenty to ninety per cent of the full fixed-end moment, thereby permitting from eighty to ten per cent of the beam rotation required for a theoretically flexible connection. Fig. 1(b) illustrates the action of a semi-rigid connection.

6. "Per Cent Rigidity" Defined.

When used in this report, the term "per cent rigidity" will mean the ratio of the moment developed by the connection with no column rotation, to the moment developed by a fully rigid connection under the same conditions, times one hundred.

7. "Beam-Line" Defined.

Fig. 2 shows a typical graph of the relation between moment at the connection and relative rotation of the beam and column. On the same graph is plotted a straight line whose equation is

$$M + \frac{2EI\phi}{l} = M_r \qquad (1)$$

Nd dun

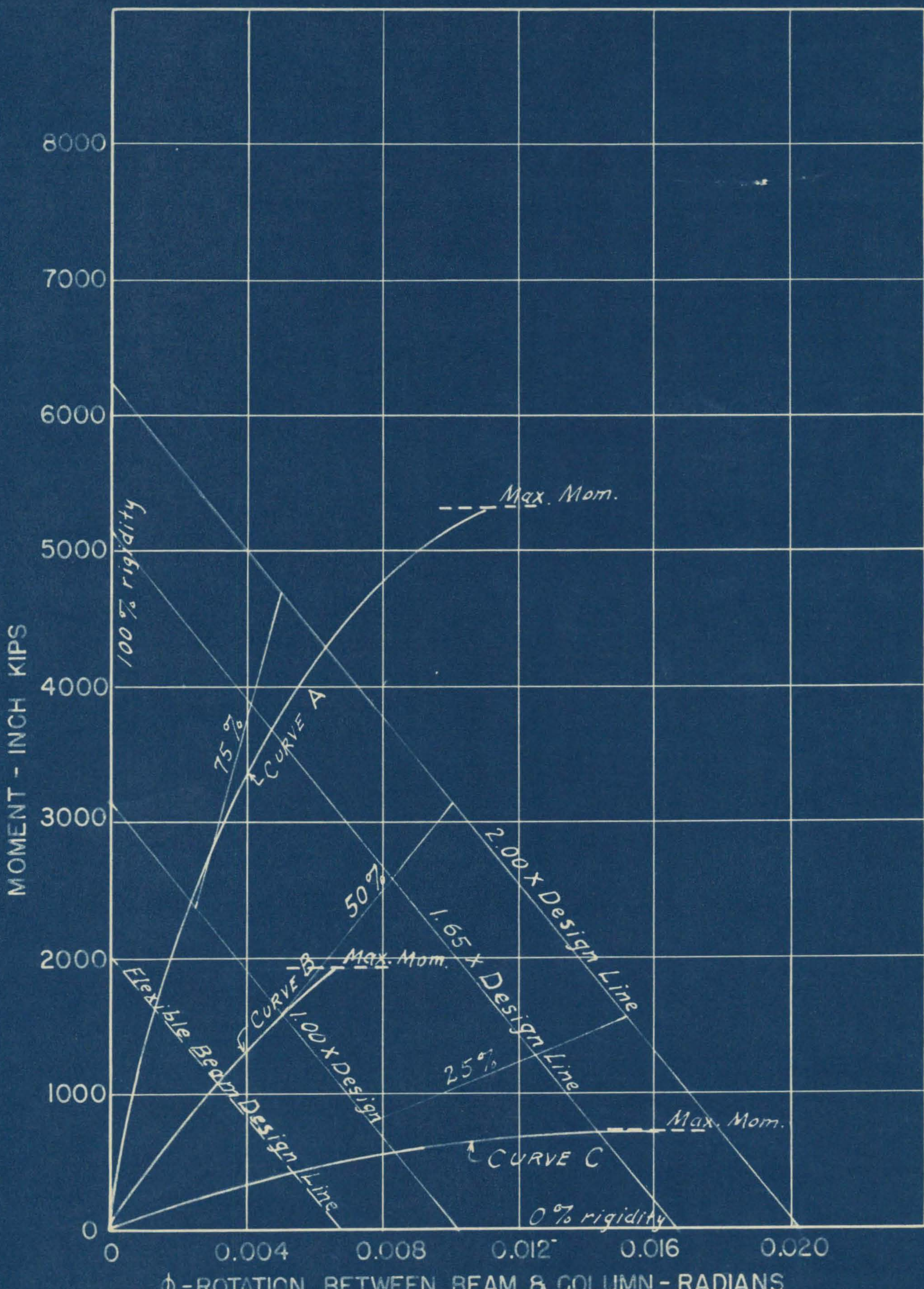


FIG. 2. TYPICAL MOMENT-ROTATION CURVES

in which

M = moment applied to connection

E = Young's modulus

I = moment of inertia of beam cross-section

l = assumed length of span

ϕ = angle of rotation in radians of the beam with respect to the column

M_r = fixed-end moment at the connection with zero rotation of the beam with respect to the column

According to C. Batho[5*], the line of equation (1) is the design requirement line for any single symmetrical loading of the beam if the columns do not rotate. If, for a particular connection, the moment-rotation curve crosses the line of equation (1) at any point, without previous failure, the connection will be able to carry the design load. In this report the graph of equation (1) will be called the "design requirement line for the beam", or, for brevity, "beam-line".

Fig. 2 also shows a dashed line which represents the design beam line for a flexible connection as determined for uniform load on the beam and a stress of 20,000 p.s.i. at the center of the beam. This line is theoretically correct for the standard assumptions mentioned.

- -

* Numbers refer to items in Bibliography.

III. Design of Test Specimens.

8. Specifications.

In general, the A.W.S. Building Code was followed in the design of the test specimens. When assumptions were made, the conservative estimate was chosen.

9. General Assumptions for All Specimens.

a. Allowable Stresses.

(1) Tensile stress in rolled sections = 20,000 p.s.i.

(2) Shear stress in welds = 13,700 p.s.i.

(3) Tensile stress in butt welds = 16,000 p.s.i.

b. Length-depth ratio for beams = 15.

c. Uniform load on beams.

10. Additional Assumptions for Semi-Rigid Connections.

a. The rigidity of the connection for use in designing the beam = 50 per cent.

b. The rigidity of the connection for use in designing the connection = 75 per cent.

c. Story height = 12 ft.

11. Method of Design of Semi-rigid Connections.

a. A beam size was selected and the length-depth ratio of fifteen then established the span automatically.

b. The lightest column section into which the beam could be framed was selected in order to place the connection at the greatest disadvantage.

c. The value of uniform load and the accompanying end shear on the beam for an assumed fifty per cent rigidity of connection was calculated. (For detailed procedure see reference 2).

d. Using the uniform load obtained in (c) the end moment for seventy-five per cent rigidity and no column rotation was calculated.

e. The seat detail was designed to resist the shear by a standard procedure.

f. The weld joining the beam to the seat was designed to carry a thrust equal to the end moment from (d) divided by the depth of the beam.

g. The top plate was designed to carry the same force as in (f), but at a stress of 16,000 p.s.i. This stress was used because the top plate was joined to the column with an unreinforced full penetration butt weld, for which the allowable stress was only 16,000 p.s.i. A full penetration weld was obtained by the use of a back-up strip tacked in place before assembly of the top plate. A detail of the butt weld is shown in Fig. 3. Reinforcement on the butt weld was eliminated since pilot tests indicated that the extra weld metal was not fully effective in increasing strength to the extent permitted by the specifications.

h. The fille weld for the top plate was designed in the usual manner to resist the same force as in (f).

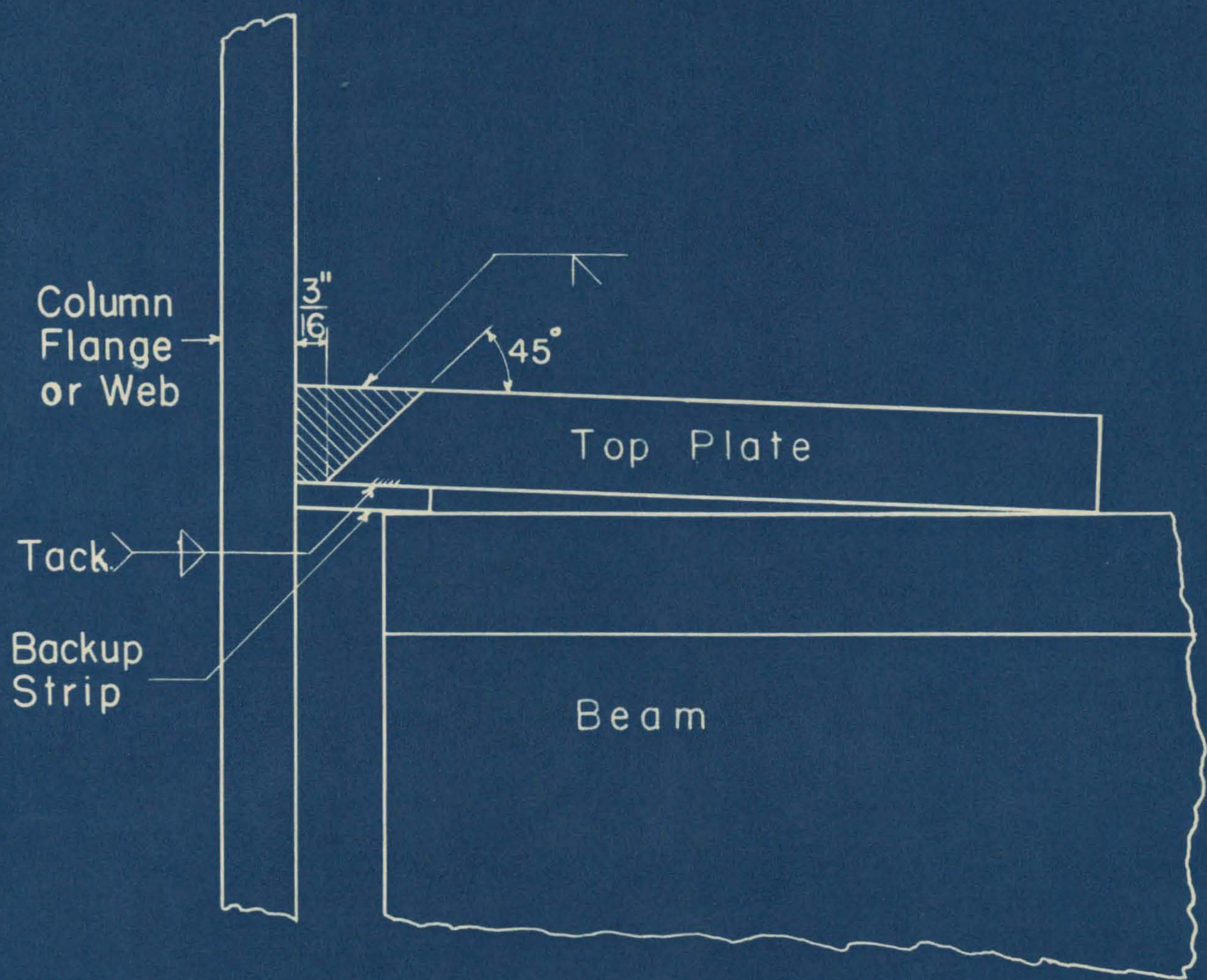


Fig. 3. BUTT WELD DETAIL

i. Table I lists the sizes of beam stubs, column stubs, top plates, seat details, and design shear and moment for each specimen, as well as moment arms for the test loads.

The procedure for the design of the top plate details were the result of a series of pilot tests made on several different top plates. The tests, reported previously in a progress report by Johnston and Brandes, were simple tension tests in which load-deflection data were obtained. From the elongation of these pilot plates, the rigidity of joints in which they might be used could be estimated. The unsatisfactory designs were eliminated and details were refined in this manner.

12. Explanation of Rigidity Assumptions.

In the design of the semi-rigid specimens, the apparent discrepancies of rigidity assumptions in the design of beams and connections may be explained as follows:

a. The top-plate connections were calculated from the pilot tests to have a rigidity in the neighborhood of seventy-five per cent. However, if the beam were to be designed assuming seventy-five per cent rigidity in connection, the beam would be working at the design stress only if the actual connection rigidity were seventy-five per cent. Any decrease of actual connection rigidity below seventy-five per cent would overstress the beam at the center, and any increase of rigidity would overstress the beam at the end.

TABLE I

DESIGN SIZES AND VALUES FOR SPECIMENS

Specimen No.	Specimen Type*	Sizes Of: Beam Stub	Sizes Of: Column Stub	Sizes Of: Seat	Sizes Of: Top Plate	Design Shear kips	Design Moment kip-in	Moment Arm For Test
1	WD	12 WF 25	10 WF 49	ST 7 WF 21.5 x 8-1/2"	5-1/2 x 3/8 x 7	18.2	405	22.5
2	WD	12 WF 50	12 WF 65	ST13 WF 45.5 x 6	7 x 5/8 x 9	36.6	840	23.0
3	WD	12 WF 47	12 WF 65	ST 9 WF 32 x 9	6 x 5/8 x 8	31.3	1045	33.25
4	WD	12 WF 85	12 WF 65	ST15 WF 54 x 7-1/2	7-1/2 x 7/8 x 12	54.0	1860	34.5
6	WD	12 WF 85	12 WF 65	ST15 WF 54 x 7-1/2	7-1/2 x 7/8 x 12	54.0	1860	34.5
7	WD	12 WF 85	12 WF 65	ST15 WF 54 x 7-1/2	7-1/2 x 7/8 x 12	54.0	1860	34.5
5	FD	12 WF 85	12 WF 65	8 x 8 x 7/8 - 10"	7-1/2 x 7/8 x 12	60.4	2070	28.5
9	FD	12 WF 85	12 WF 65	ST15 WF 54 x 8-1/2	7-1/4 x 1 x 12	60.4	2070	28.25
10	FD	12 WF 85	12 WF 65	8 x 8 x 1 - 10-1/2	7-1/2 x 1 x 12	60.4	2070	28.25
16	WD	12 WF 85	12 WF 65	ST15 WF 54 x 7-1/2	10-1/2 x 3/4 x 15	54.0	1860	34.5
17	WS	12 WF 85	12 WF 65	ST15 WF 54 x 7-1/2	7-1/2 x 7/8 x 12	54.0	1860	34.5
11	WD	12 WF 85	12 WF 65	Tee made of plates	6 & 3 x 5/16 x 12	45.4	--	(20.0 (13.0
15	WD	24 WF 74	14 WF 61	Tee made of plates	6 & 3 x 3/8 x 12	38.1	--	24.0
13	FD	18 WF 70	12 WF 65	6 x 3-1/2 x 3/4 - 10	6 & 3 x 5/16 x 12	38.1	--	(18.0 (15.0
14	FD	18 WF 45	12 WF 65	6 x 3-1/2 x 5/8 - 9	6 & 3 x 5/16 x 12	24.0	--	18.0
12	WS	18 WF 85	12 WF 65	Tee made of plates	6 & 3 x 5/16 x 12	45.4	--	20.0
18	WS	18 WF 85	12 WF 65	Tee made of plates	6 & 3 x 3/8 x 12	45.4	--	16.0

* FS = flange connection, one side only
FD = flange connection, both sides
WS = web connection, one side only
WD = web connection, both sides

b. On the other hand, if the beam were to be designed assuming fifty per cent rigidity of connection, then an actual connection rigidity from fifty to one hundred per cent would neither overstress the beam at the center nor at the end. An actual connection rigidity of less than fifty per cent would overstress the beam at the center, but rigidities of less than fifty per cent were not considered likely. Hence the beam was designed assuming a connection rigidity of fifty per cent.

c. The connections were designed to develop seventy-five per cent rigidity because the pilot tests indicated that the type of top plate connection under consideration would have a rigidity of at least seventy-five per cent. Therefore the connection would actually produce a moment at the end greater than that obtained by using an assumption of fifty per cent rigidity, which would bring about an early failure. Seventy-five per cent rigidity was therefore assumed in determining the maximum or design moment that the connection must resist.

d. Table II illustrates the reasoning behind the above discussion, and an examination of it will bear out the rationality of the design procedure used.

13. Method of Design of Flexible Connections.

a. Beam size, column size, and seat detail were determined in the same manner as for semi-rigid connections.

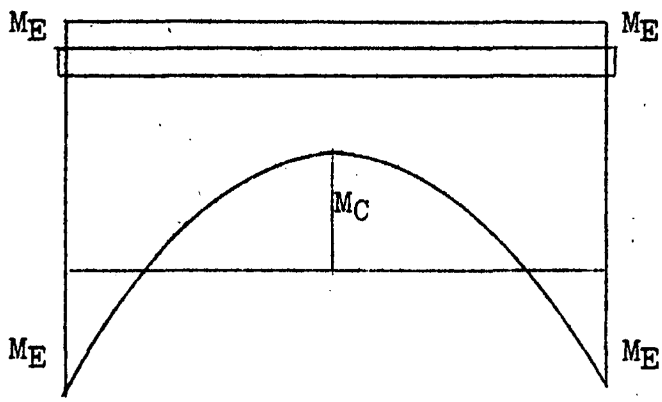
TABLE II

RELATION OF MAXIMUM MOMENTS IN A BEAM

AS RIGIDITY OF THE CONNECTION VARIES

Connection Rigidity	Design Based On 75 per cent Rigidity		Design Based On 50 per cent Rigidity	
	M_c	M_E	M_c	M_E
per cent	in kips		in kips	
0	40.0S	0	30S	0
25	33.3S	6.7S	25S	5S
50	26.7S	13.3S	20S	10S
75	20.0S	20.0S	15S	15S
100	13.3S	26.7S	10S	20S

S = Section Modulus



b. The end shear on the beam was calculated from the uniform load required to stress the beam to 20,000 p.s.i. assuming a flexible connection.

c. The 2-1/2 in. length of 1/4-in. fillet weld was used in accordance with standard practice as the equivalent of a 3/4-in. rivet.

d. The top plate detail was designed so that the wide portion of the plate and the butt weld would be working at a stress of 16,000 p.s.i. and the fillet weld at 13,700 p.s.i. when the narrow portion of the plate was under 32,000 p.s.i. The narrow portion of the plate should then have been able to yield enough to provide flexibility without overstressing the welds.

IV. Fabrication of Test Specimens.

14. Welding Operators.

The seventeen specimens were made by four qualified welders. Each welder used slightly different technique, but the sequence of welding was the same in all cases. Eleven specimens were made by the laboratory welder and four by a welder from a production shop working in the laboratory. Of the two remaining specimens, duplicates of one of the laboratory specimens, one was made in a large fabricating shop and the other in a small fabricating shop using their standard procedure.

15. Materials.

The rolled sections and plates were bought under A.S.T.M. Specification A-7-39 for building and bridge steel. The electrodes used were bought under A.S.T.M. Specification A-233-40T. Only the top-plate material was checked for conformance with specifications.

16. Welds.

All welds except butt welds larger than 5/8-in. were made with E 6010 electrode, and the butt welds larger than 5/8-in. were made with E 6020 electrode. As much as possible, all welds were made as they would be made in the shop or in the field. For field welds no electrodes larger than 3/16-in. were used. D.C. arc welding was used in all cases with flat positions.

V. Method of Test.

The requirements for building connections in general are two-fold. The connection must be able to withstand the shear reaction, and it must also be able to withstand the actual moment developed at the column. The shear requirement may be checked by the application of load as close to the connection as possible, but the moment requirement is not so simple an item to check. The method for checking moment requirements used herein consists of plotting the moment-rotation curve for loads applied at the theoretical inflection point for fifty per cent rigidity, and then noting how much farther than the 1.00 X design beam line the curve extends.

17. Arrangement of Tests.

The specimens were arranged for test as shown in Fig. 4 and 5. The connection was upside down in the testing machine because it was much more convenient for test purposes.

For semi-rigid connections, the distance "a" was made such that design shear was developed when design moment on the connection was reached, i.e., the test support was placed at the theoretical point of inflection. For flexible connections, the distance "a" was made some convenient value. It was noted that a change in distance "a" had little effect on the shape of the moment-rotation curves. Fig. 43 shows how smoothly the curve continued after the distance "a" had been reduced.

18. Measurements Taken.

a. Ames dials were located as shown in Fig. 4 to measure the horizontal movement of the top and bottom flanges. These measurements could be used to check rotation measurements and to locate the center of rotation, but in actual performance the gages did not give the accuracy of measurement desired.

b. The rotation bars shown in Fig. 4 were used in conjunction with a portable bubble gage, or "level bar", to measure the relative rotation of beam and column stubs. The portable bubble gage consisted of a level bubble mounted on a bar. The rear end of the bar carried two conical

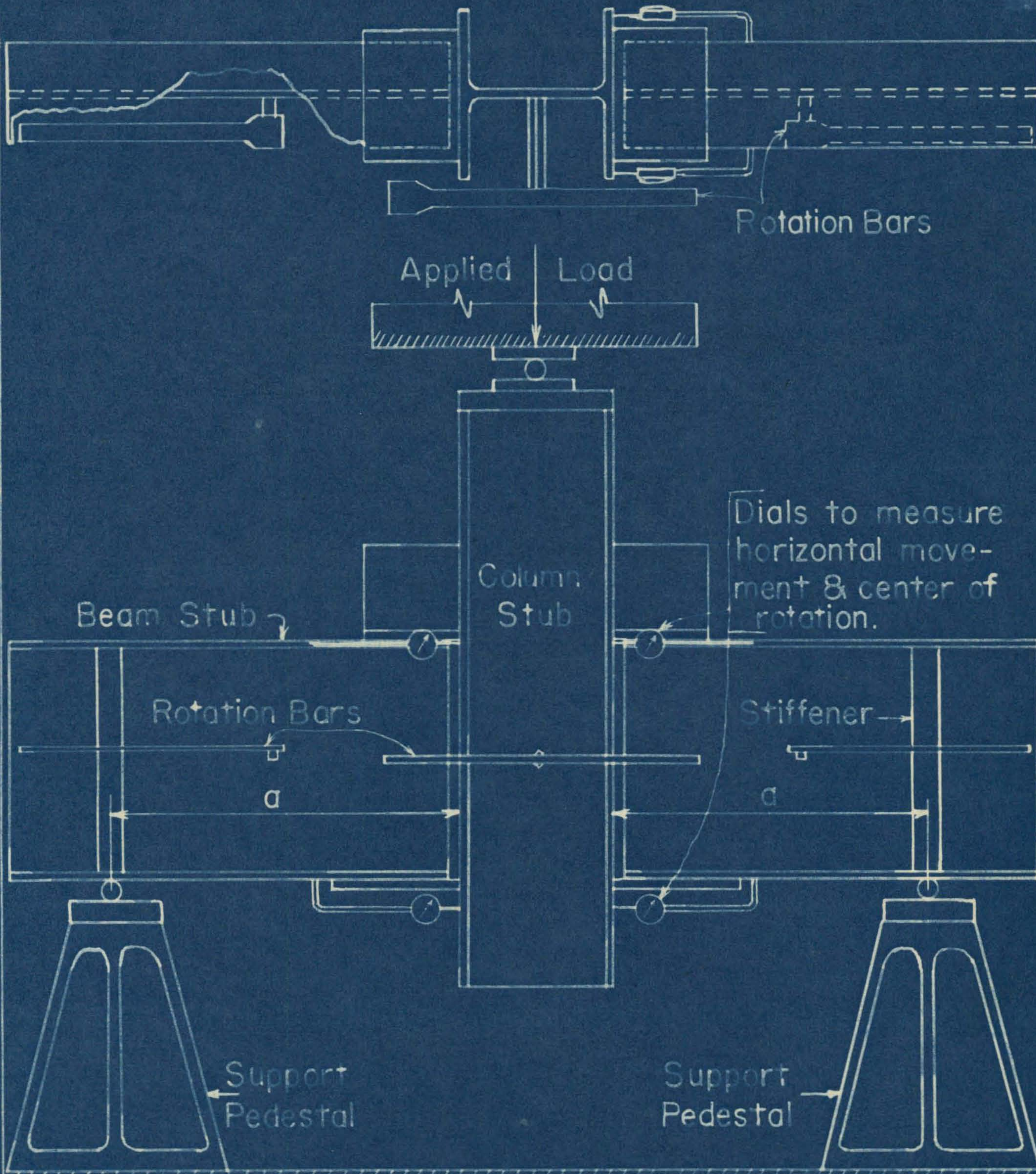


FIG. 4. DIAGRAM OF APPARATUS FOR TESTS.

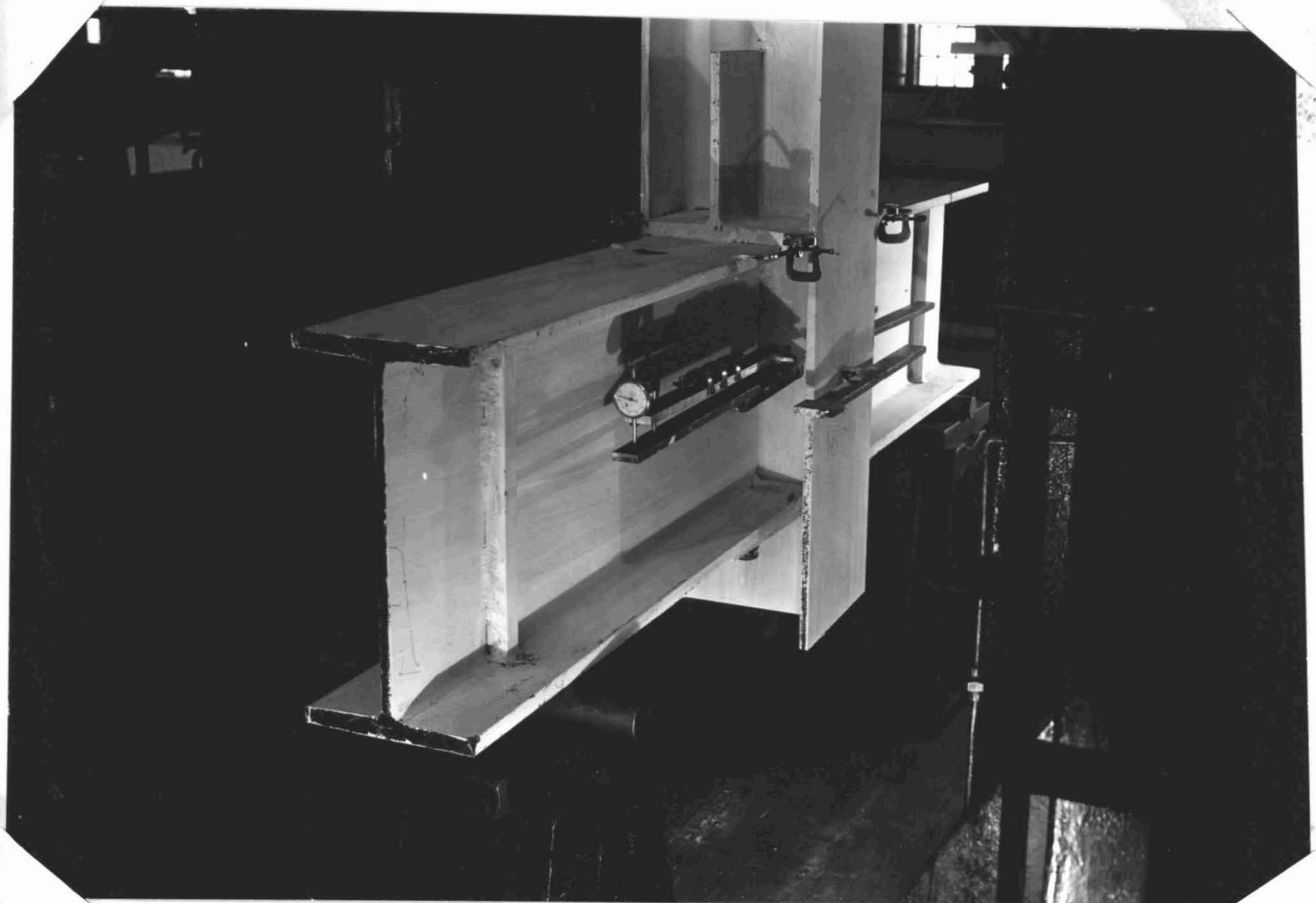


Fig. 5 - Typical Test Set-Up

points and the forward end a micrometer adjustment for raising and lowering the end of the bar, and an Ames dial to indicate the amount of raising or lowering. In tests the gage was standardized on an immovable gage line, then placed on each rotation bar in turn. The micrometer adjustment was used to make the bubble level, and a reading of the Ames dial was taken. This process made it possible to get rotation values by simple division of the dial differences with an accuracy of ±10 seconds. The level bar is shown in place for a reading in Fig. 5.

19. Application of Load.

In general, load was applied to the column stub in five increments up to the design load. Load was then removed and the permanent set measured. The specimen was reloaded and five more increments taken up to twice the design load. The load was once more removed and permanent set measured. Ames dials were removed, the load was brought back to twice the design load, and rotation readings were continued at increments until failure occurred.

As the specimen was tested, observations of the "strain lines" which formed as the mill scale chipped off were made. Whitewash was used to provide a contrasting background so that the flaking of the mill scale could be more easily observed. The formation of "strain lines" by flaking of the mill scale usually indicates stresses in the yield strength range in the region where the lines occur.

VI. Results of Tests.

20. Tests 1, 2, 3, 4.

The first four tests were intended to show the difference in top plate behavior for various sized beams. Details of specimens are shown in Fig. 6 to 9; graphs of the moment-rotation characteristics under test conditions are shown in Fig. 10 to 13; and photographs of the different specimens after test are shown in Fig. 14 to 17. The moments at which strain lines first appeared at various places in the specimen are listed in Table III.

a. Test 1.

At V = 34 kips, M = 757 kip-in., the top plate was pulled away from the beam flange by 1/16-in. When the load was dropped to zero, the plate returned to its original position, hence it would seem that up to this point the bending in the top plate was not serious.

At V = 58 kips, M = 1305 kip-in., the beam web buckled over the support pedestals, so that stiffeners were put in before the test was continued. The later buckling of the beam web over the seat at V = 76.4 kips, M = 1720 kip-in., may have been hastened by the earlier buckling over the support pedestals, but the reduction of maximum moment was probably not great.

The strain patterns developed in the tee web were particularly interesting, and a photograph of them is shown in Fig. 18. The strain lines shown were fairly typical for all the seat tees.

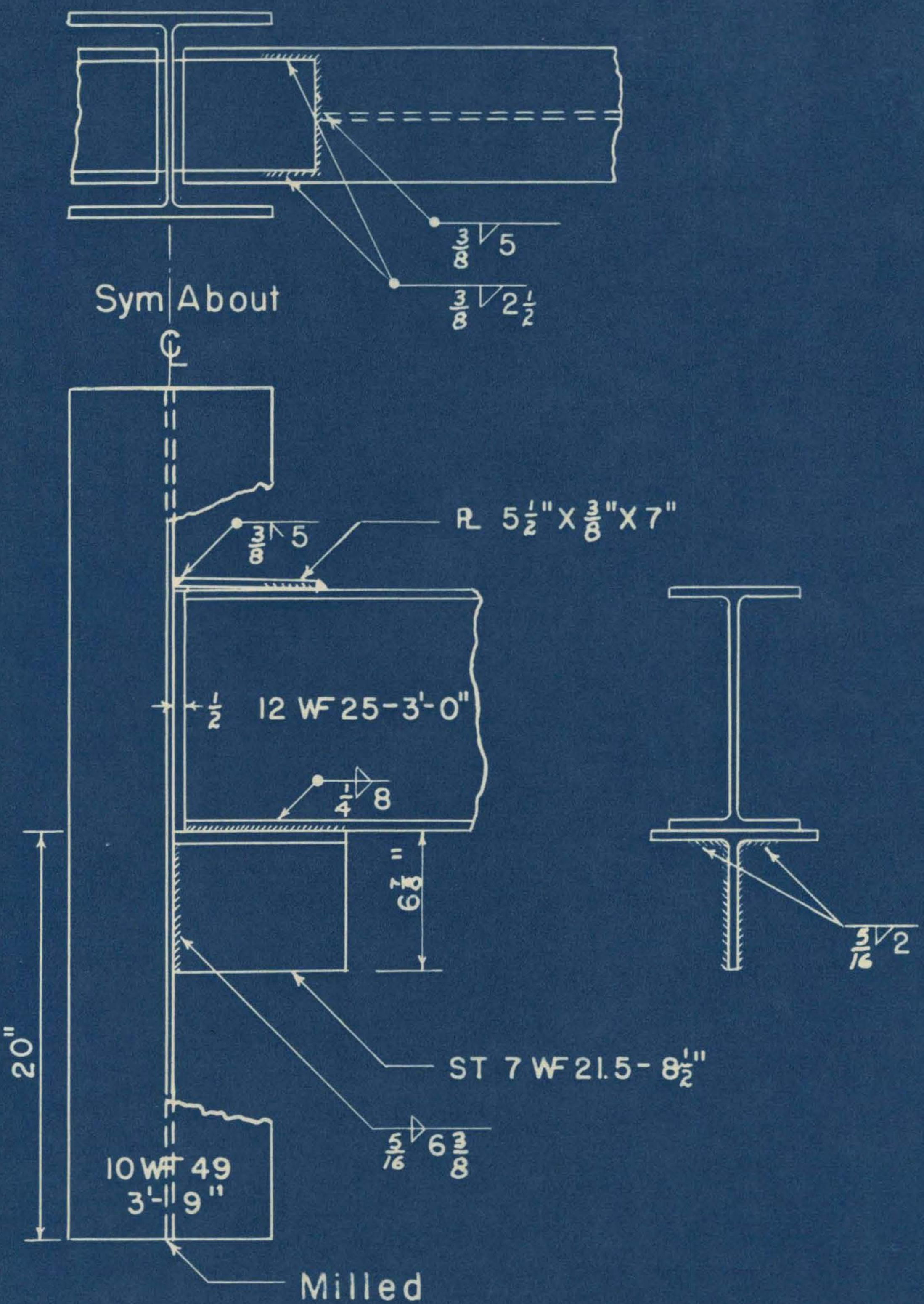
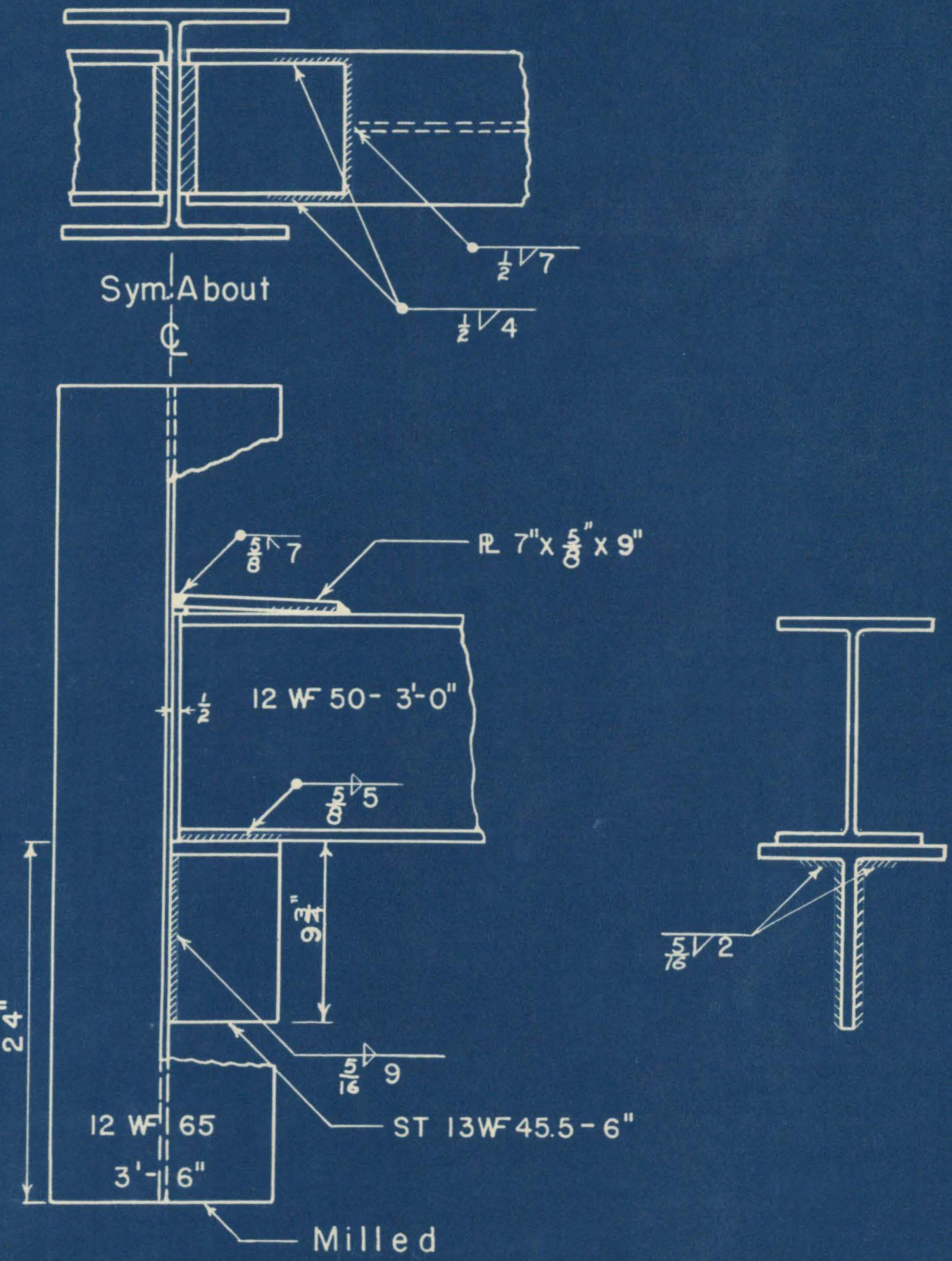
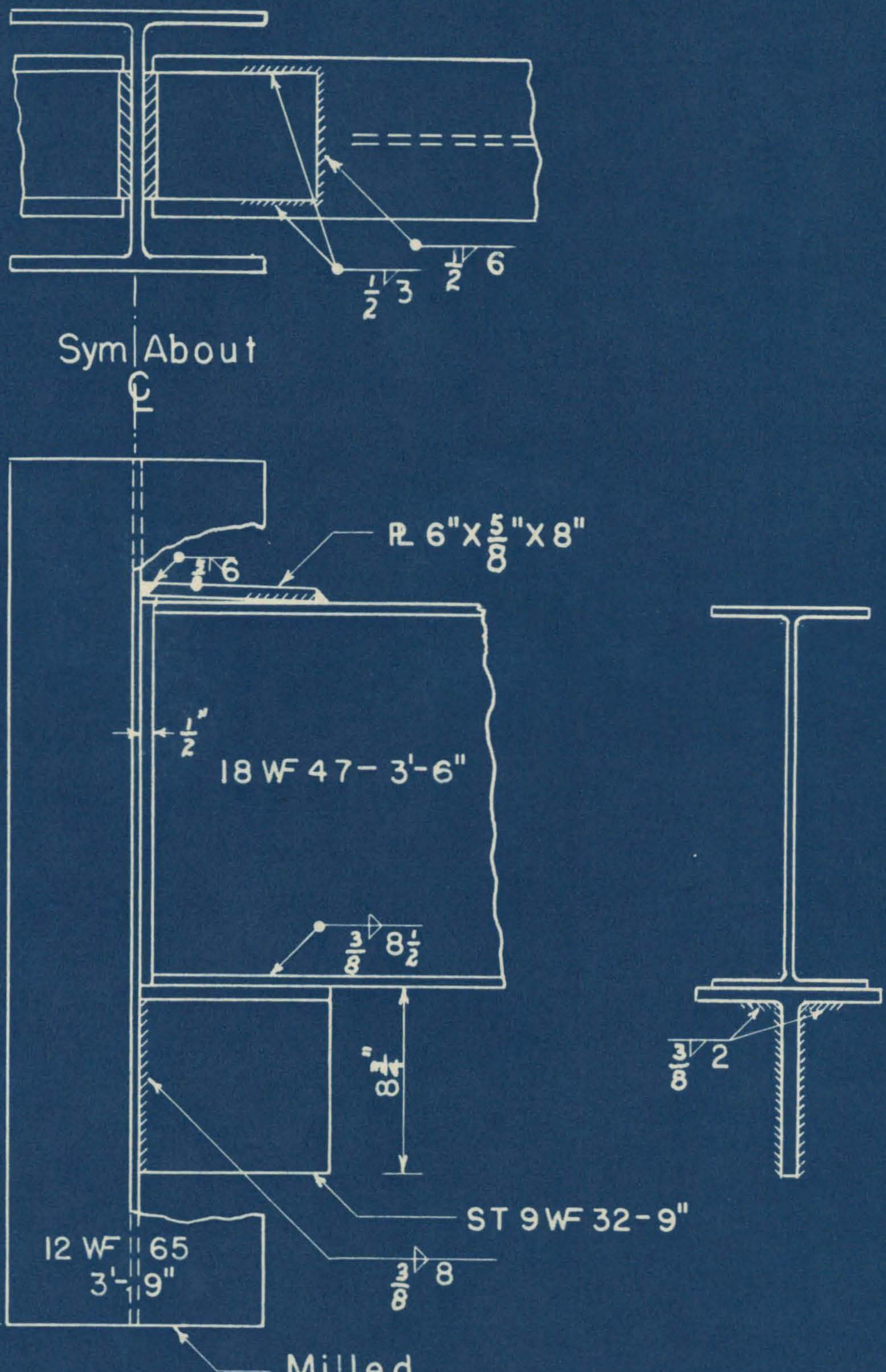

3/8 5
3/8 2 1/2
Sym About ℄
PL 5 1/2" X 3/8" X 7"
3/8 5
1/2
12 WF 25-3'-0"
1/4 8
6 7/8"
20"
ST 7 WF 21.5 - 8 1/2"
5/16 6 3/8
10 WF 49
3'-1 9"
Milled
5/16 2

Fig. 6 TEST 1



$\frac{1}{2}$ 7
$\frac{1}{2}$ 4
Sym. About ℄
$\frac{5}{8}$ 7
PL 7" x $\frac{5}{8}$" x 9"
$\frac{1}{2}$
12 WF 50 - 3'-0"
$\frac{5}{8}$ 5
24"
$9\frac{3}{4}$"
$\frac{5}{16}$ 9
ST 13WF 45.5 - 6"
12 WF 65
3'-6"
Milled
$\frac{5}{16}$ 2

Fig. 7. Test 2



1/2 3
1/2 6
Sym About ℄
PL 6"X5/8"X8"
3/8 6
1/2"
18 WF 47 – 3'-6"
3/8 8 1/2
3/8 2
8 3/4"
24"
ST 9 WF 32 – 9"
12 WF 65
3'-9"
3/8 8
Milled

Fig. 8. TEST 3

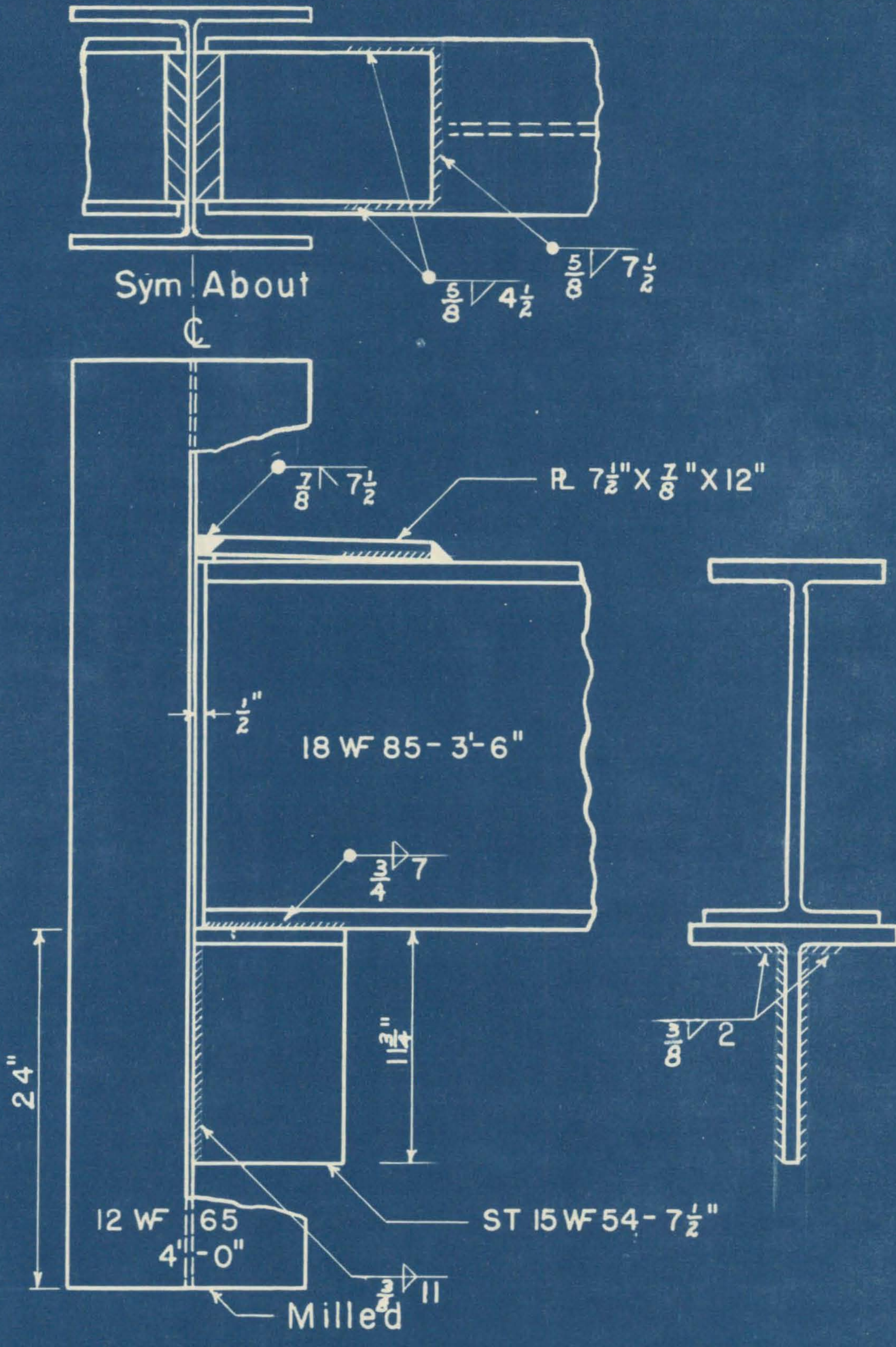


Fig. 9. TESTS 4, 6, & 7

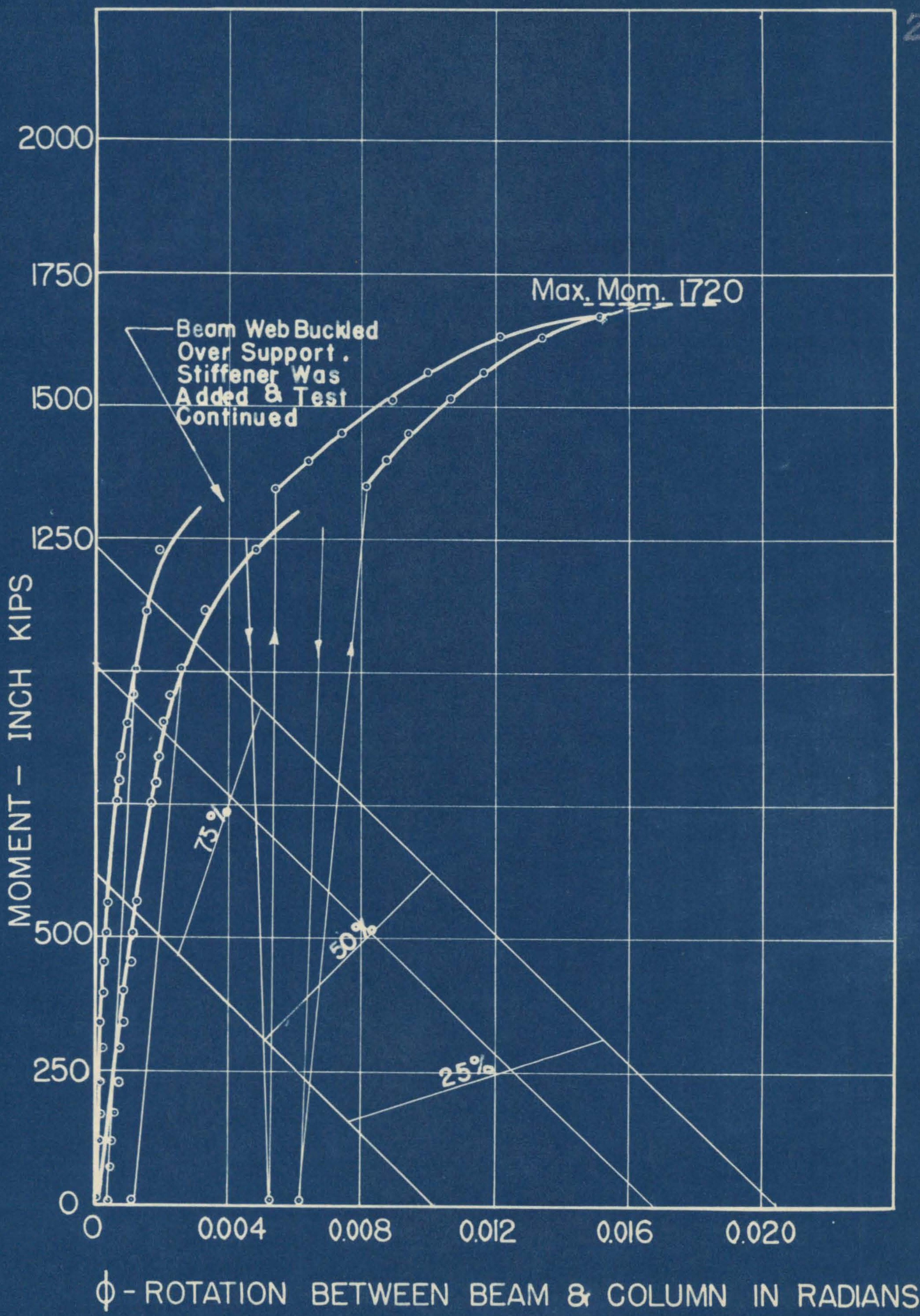


FIG. 10. MOMENT-ROTATION CURVES FOR TEST 1.

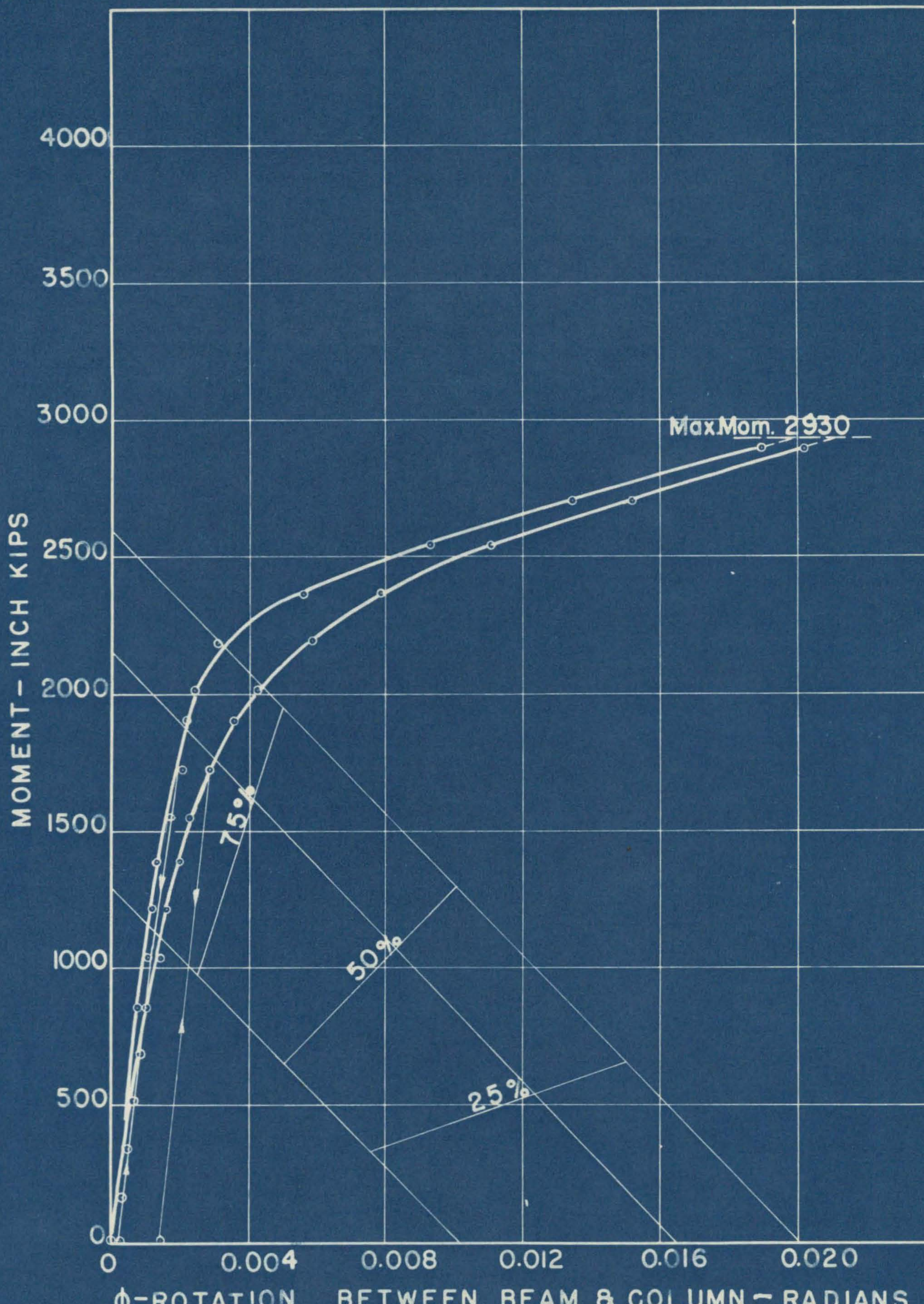


FIG. 11. MOMENT-ROTATION CURVES FOR TEST 2

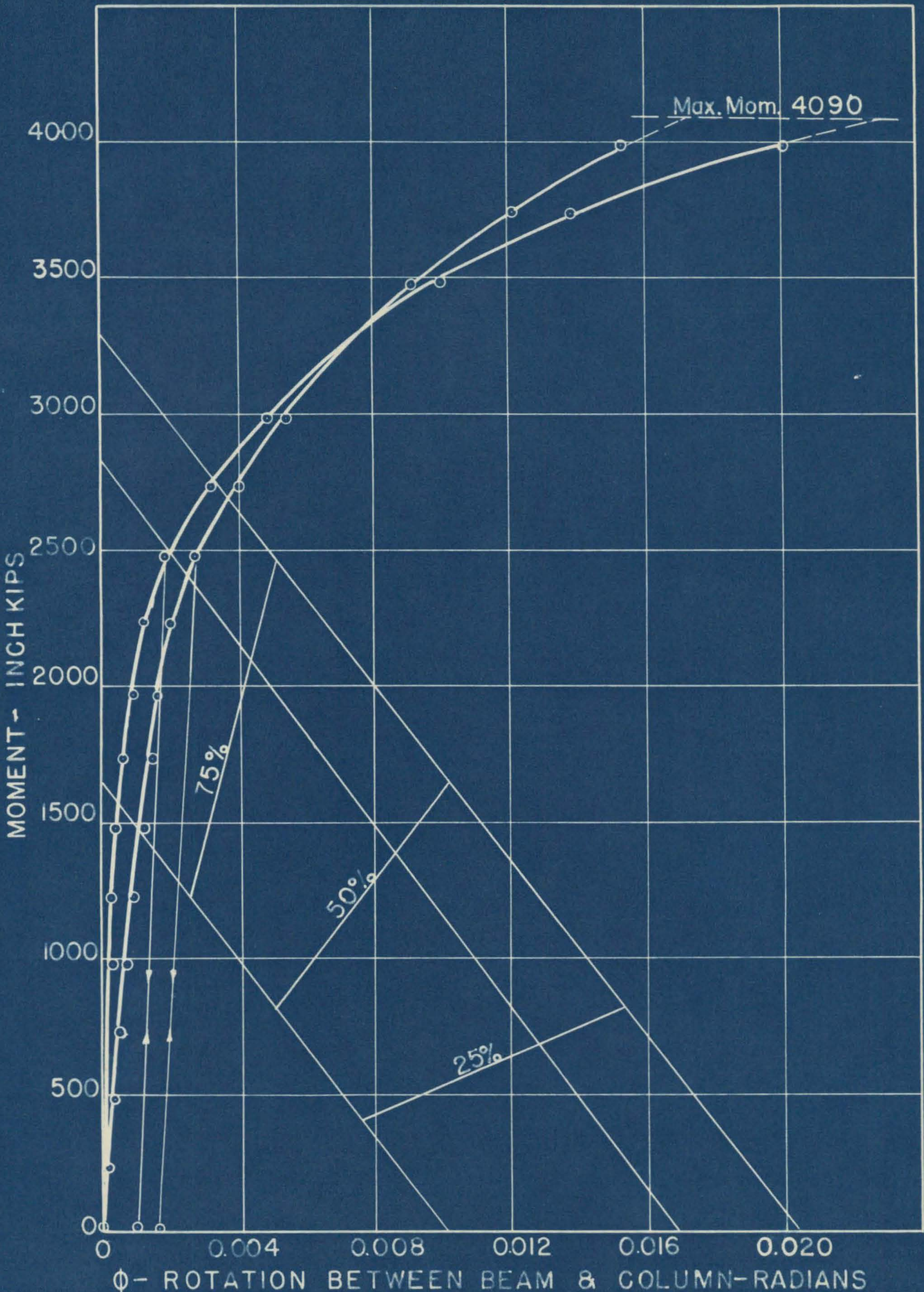


FIG. 12. MOMENT-ROTATION CURVES FOR TEST 3

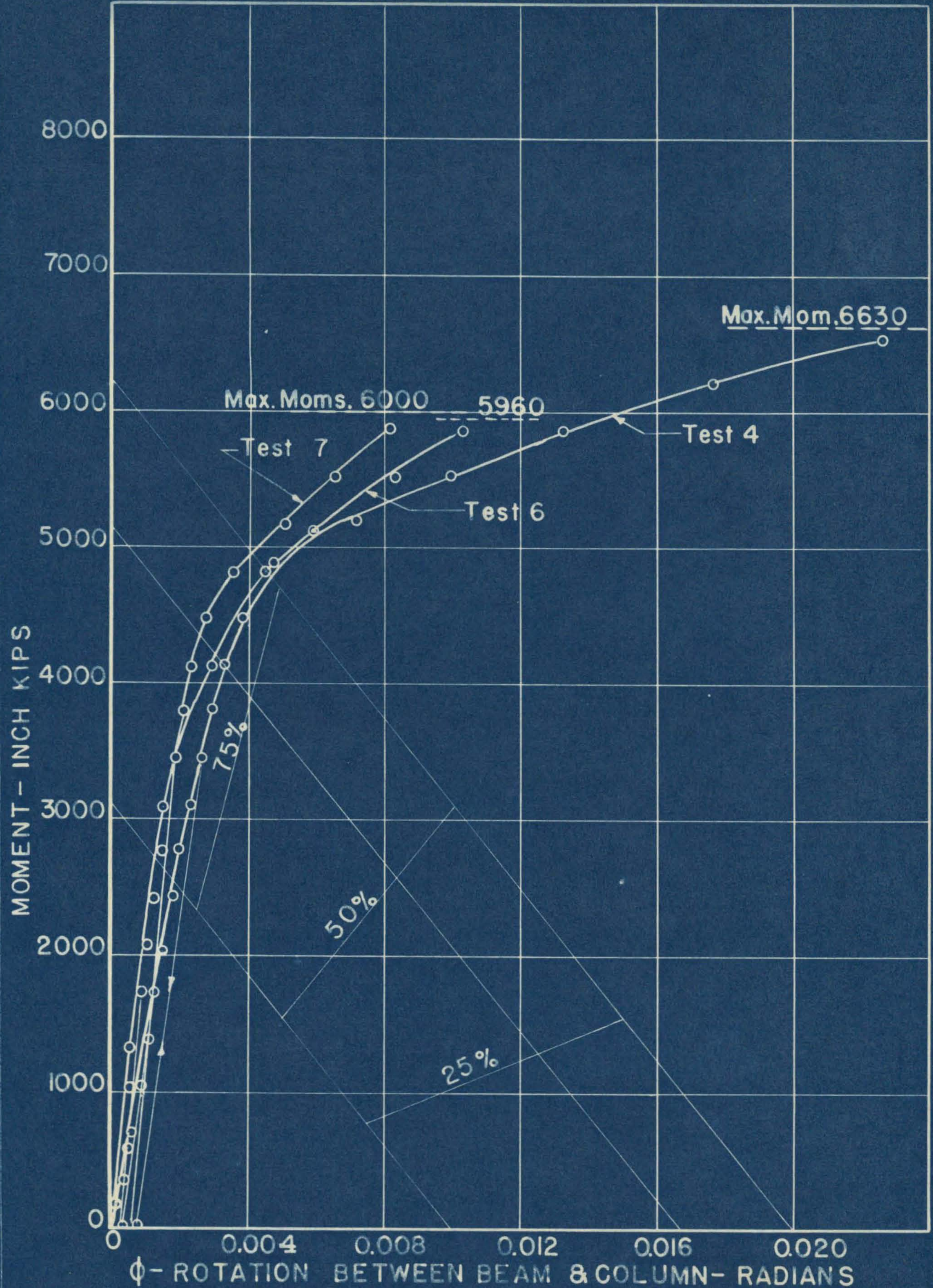


FIG. 13. MOMENT-ROTATION CURVES FOR TESTS 4,6,&7

Fig. 1(a) - Top Plate After Test

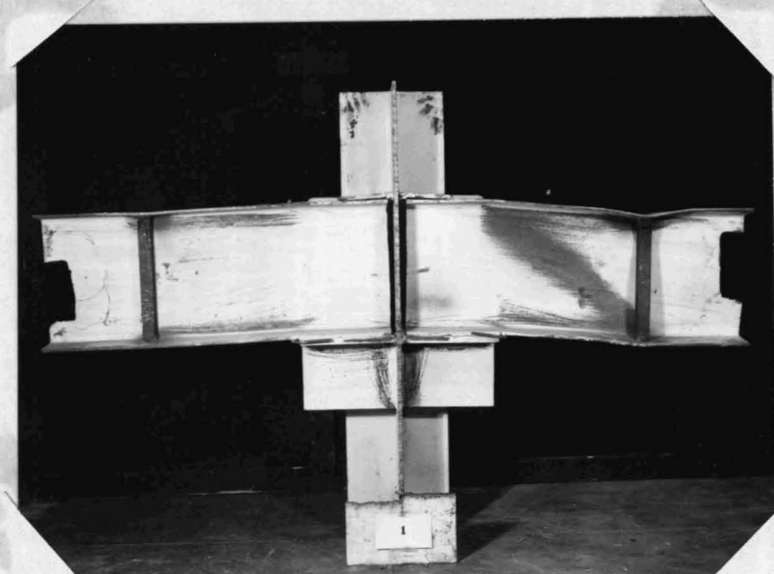


Fig. 1(b) - Test 1 After Failure

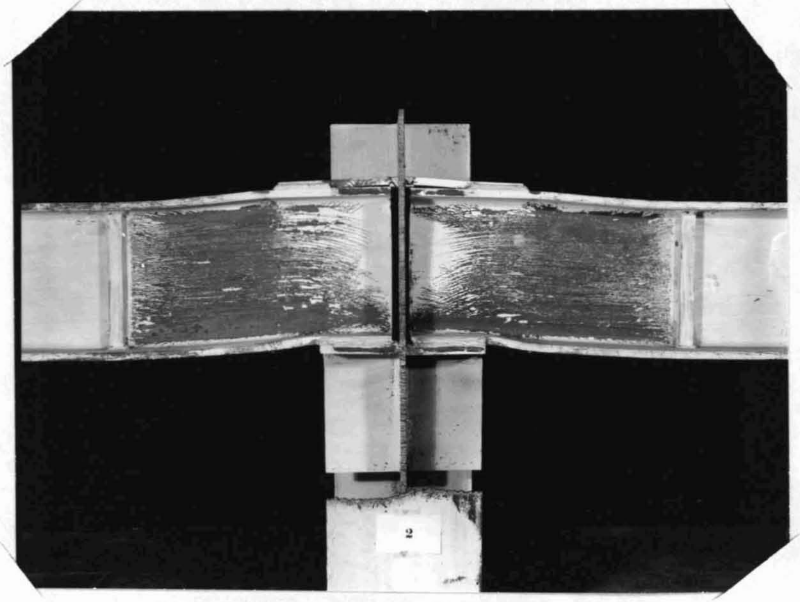


Fig. 15 - Test 2 After Failure

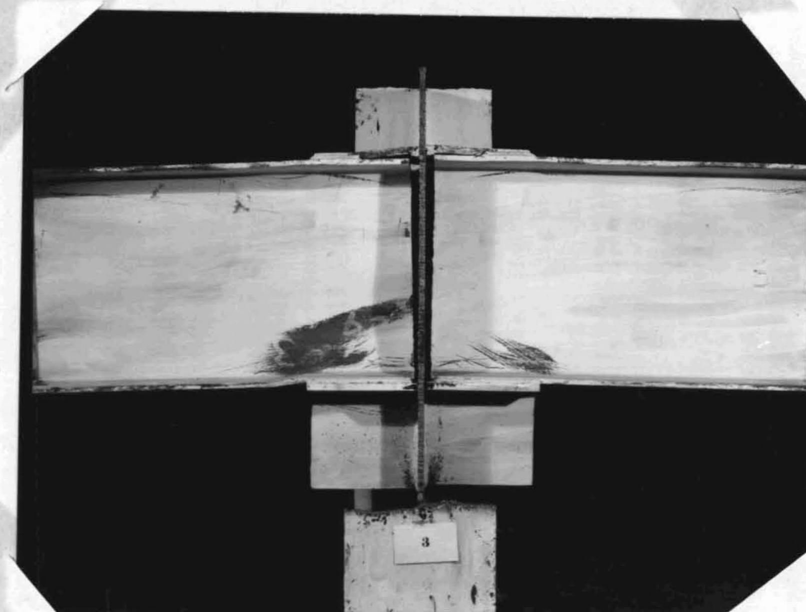


Fig. 16 - Test 3 After Failure

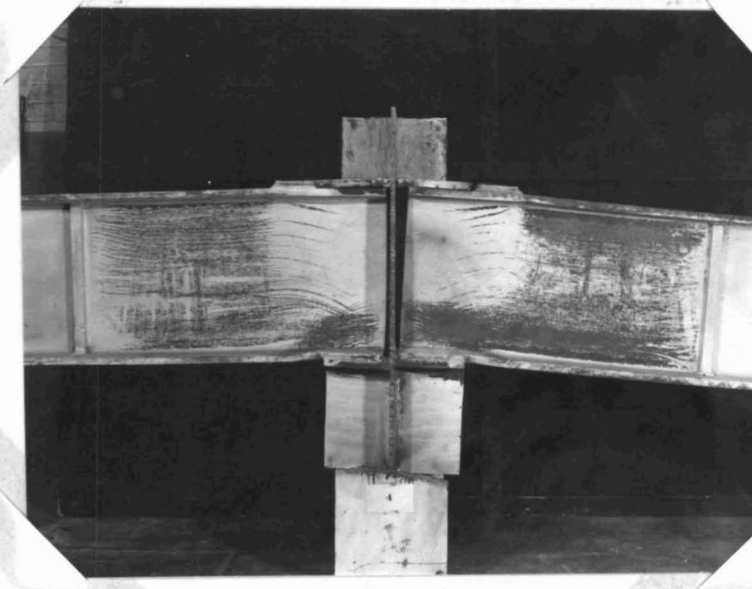


Fig. 17(a) - Test 4 After Failure

Fig. 17(b) - Fracture of Test 4 Top Plate

TABLE III

BEHAVIOR OF SPECIMENS DURING TEST

Specimen No.	Moments In Kip-Inches At Which Strain Lines Appeared: Beam Over Seat	Beam Web Shear Failure	Web Of Seat Tee	Flange Of Seat Tee	Lower Flange Of Beam	Top Flange Of Beam Near Top Plate	Top Plate	Web Of Beam Under Top Plate	Column At Seat	Column At Top Plate	Maximum Applied Moment kip-in.	Maximum Applied Shear kips	Type Of Failure
1	568	--	1305	--	--	--	902	--	--	--	1720	76.4	Beam Web Buckle Over Seat
2	863	2110	--	2700	2190	2010	1380	--	--	--	2930	127.4	Beam Web Buckle Over Seat
3	3240	--	3240	--	3240	2240	2740	--	--	--	4090	123.0	Beam Web Buckle Over Seat
4	2070	5879	3800	--	5170	4830	4830	--	--	--	6630	192.0	Top Plate Brittle Fracture
6	2760	4140	--	--	4840	--	--	--	--	--	5960	173.0	Top Plate Brittle Fracture
7	3800	--	3800	--	--	--	(5860)≠	3100	--	--	6000	174.0	Top Plate Brittle Fracture
5	1430	--	--	(2560)°	--	--	2850	--	2280 W/F	1430 W/F	3360	118.0	Pullout Flange Metal of Column at Top Plate
9	3960	--	4240	--	--	--	--	3680	849F	3110W 849F	4320	153.0	Pullout Flange Metal of Column at Top Plate
10	--	--	--	--	--	--	--	--	1410W	--	4270	151.0	Column Web Buckle at Seat
16	2070	--	5160	--	5160	--	3450	4830	--	--	5690	165.0	Top Plate Tear
17	2740	--	--	--	--	--	3450	--	1370	--	3760	109.0	Column Web Metal Tore Out
11	--	--	1100	600	--	--	600	--	--	--	1186 1520**	59.3 117.0	Top Plate Tear
15	1800	--	1620	--	--	--	1260	--	--	--	2064	86.0	Top Plate Tear
13	1010	--	(810)°°	--	--	--	675	--	--	810	1080 1360**	60.0 90.5	Top Plate Tear
14	450	630	(630)°°	--	--	--	630	--	720	630	1233	68.5	Top Plate Tear
12	--	--	750	--	--	--	700	--	750	600	936	46.8	Column Web Metal Tore Out
18	600	--	1560	--	--	--	600	--	720	--	1580	98.5	Top Plate Tear

/ W = web, F = flange

** reloaded with shorter moment arm

° fillet of angle rather than seat tee

≠ brittle cracks

°° vertical leg of angle

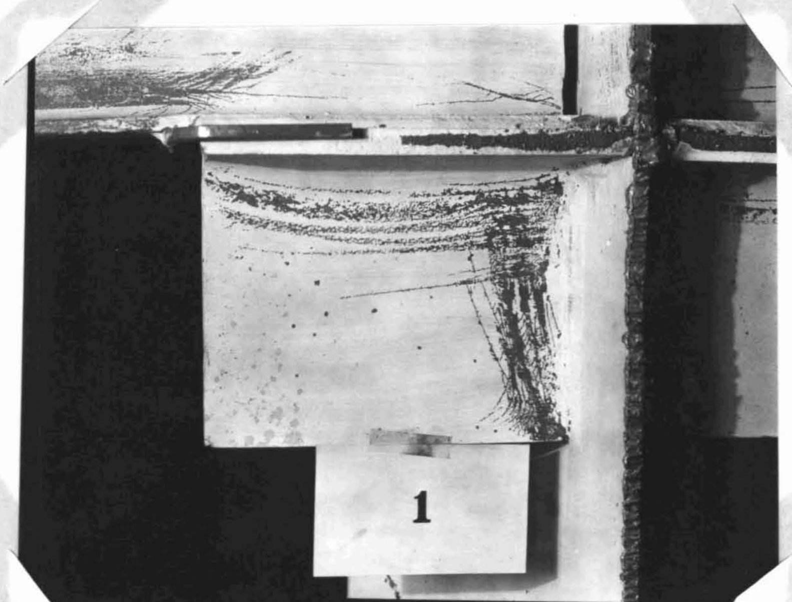


Fig. 18 - Strain Pattern On Tee

Stiffeners for the beam web over the support pedestals were used from the beginning of the test for all the other semi-rigid connections.

b. Test 2.

At V = 77.5 kips, M = 1780 kip-in., a weld ping was heard which is usually a dependable sign of weld failure, but no failure was visible in this case. The strain lines in the lower flange of the beam at the edge of the seat tee which appeared at V = 95.0 kips, M = 2190 kip-in., were caused by local bending of the beam at that point. This local bending is clearly shown in Fig. 15. At V = 127 kips, M = 2930 kip-in., failure occurred by the buckling of the beam web over the seat tee.

c. Test 3.

At V = 82.5 kips, M = 2740 kip-in., the top plate was 1/8-in. away from the beam flange. The strain lines in the beam over the seat noted at V = 97.5 kips, M = 3240 kip-in., are shown in Fig. 16. At V = 115.5 kips, M = 3840 kip-in., a weld ping was heard, but again no failure in the weld was visible. The sound may have been caused by adjustment taking place in the rollers rather than failure of the weld. At V = 120 kips, M = 4000 kip-in., the top plate was 3/8-in. away from the beam flange. At V = 123 kips, M = 4090 kip-in., the beam web buckled over the west tee and the specimen failed.

d. Test 4.

The strain lines noted for the lower flange of the beam at the edge of the tee at V = 150 kips, M = 5170 kip-in., were caused by local bending of the beam about the edge of the tee. Examination of Fig. 17(a) will show the extent of the local bending. At V = 150 kips, M = 5170 kip-in., the top plate was 1/16-in. away from the beam flange.

The top plate failed suddenly with a brittle fracture at V = 192 kips, M = 6630 kip-in. The fracture section is shown in Fig. 17(b). The rather coarse-grained character of the fracture surface and the lack of any necking down may be noted, also the fact that the fracture started at the end of the fillet return. Control tests of the plate material before the specimen was fabricated showed an elongation in 2 in. of 43 per cent, a reduction in area of 31 per cent, and an ultimate strength of 63,000 p.s.i., which should normally indicate a "ductile" steel.

21. Tests 6 and 7.

Since Test 4 failed in the connection and was also the most difficult to fabricate, Tests 6 and 7 were made as duplicates of Test 4. In this manner, a check could be made on the reproducibility of results and the laboratory fabrication could be compared with commercial fabrication. Specimen 6 was made in a large fabricator's shop and Specimen 7 was made in a small fabricator's shop, with the author present during the welding of one side of each specimen.

Details of the specimens are shown in Fig. 9; the moment-rotation curves are shown in Fig. 13, together with the curve for Specimen 4; and photographs of the failures are shown in Fig. 19 and 20. The observations of strain lines during test are noted as before in Table III.

a. Test 6.

Specimen 6 had top plates which were a little wider than the detail called for, thereby necessitating a reduction in size of the fillet weld return and a consequent increase in length of 3/4-in.

The top plate failed suddenly with a brittle fracture at V = 173 kips, M = 5960 kip-in. The failure, shown in Fig. 19, was similar to that for Specimen 4. Control tests of the plate material before fabrication showed an elongation in 2 in. of 55 per cent, a reduction of area of 50 per cent, and an ultimate strength of 58,500 p.s.i., which was again what would normally be called a "ductile" steel.

b. Test 7.

At V = 170 kips, M = 5860 kip-in., brittle cracks in the top plate appeared at the edge of the fillet weld return. The test was stopped long enough to take the photograph shown in Fig. 20(a). The specimen was then loaded further until failure occurred at V = 174 kips, M = 6000 kip-in., by tearing of the top plate. The first 1/2-in. of the test was similar to those of Tests 4 and 6,

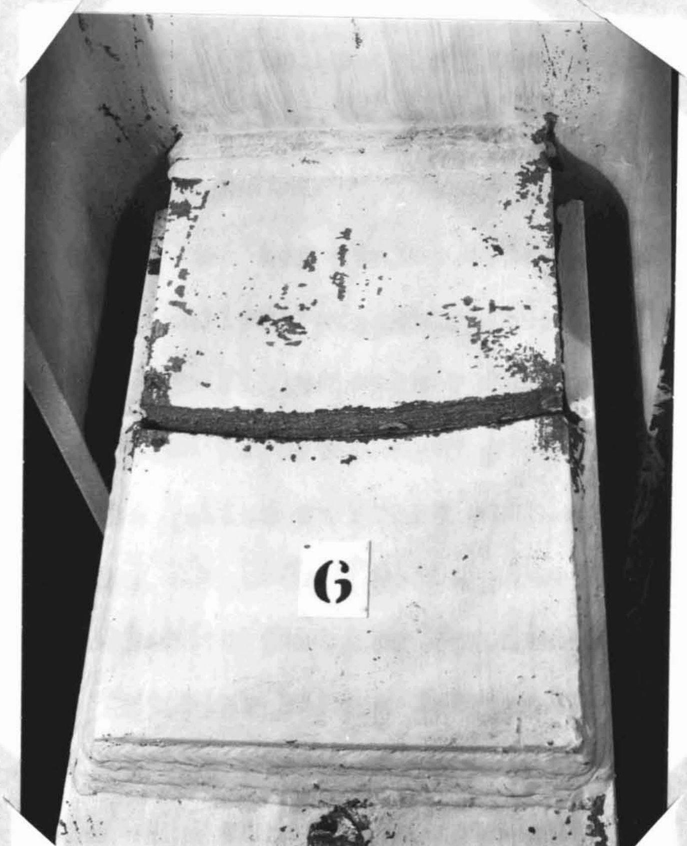


Fig. 19 - Top Plate Failure In Test 6

Fig. 20(a) - Top Plate at the Verge of Failure in Test 7

Fig. 20(b) - Top Plate Failure of Test 7 coarse-grained with no necking down, but from there on the fracture showed distinct necking down. An examination of Fig. 20(b) will bear out these statements.

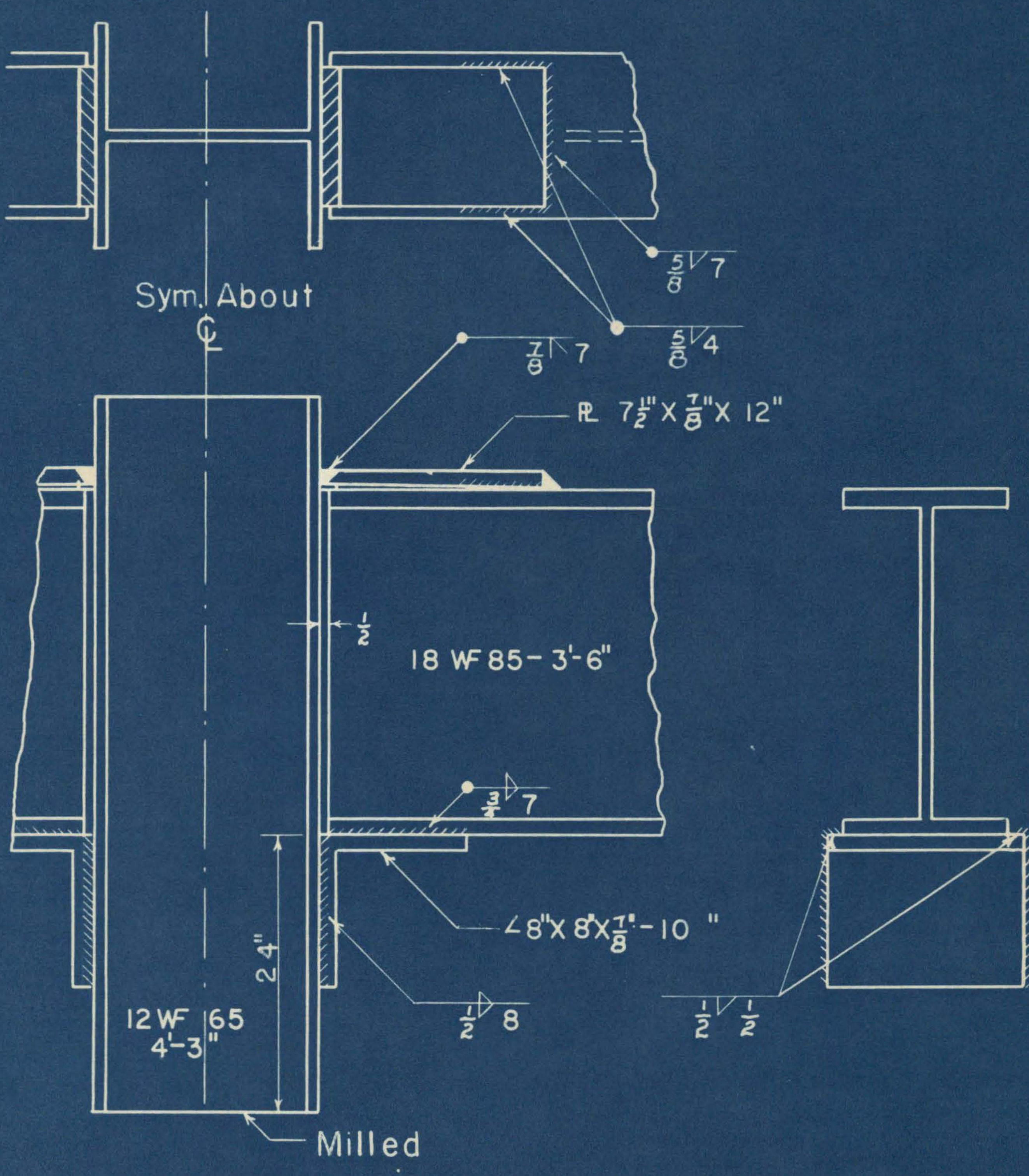
22. Tests 5, 9, and 10.

These specimens were designed to study the action of connections to the column flange. Details of specimens are shown in Fig. 21 to 23; graphs of the moment-rotation characteristics under test conditions are shown in Fig. 24 to 26; and photographs of the failures are shown in Fig. 27 to 29. The observations of strain lines during test are noted as before in Table III. Moments quoted refer to moment on the connection, NOT at the column center line.

a. Test 5.

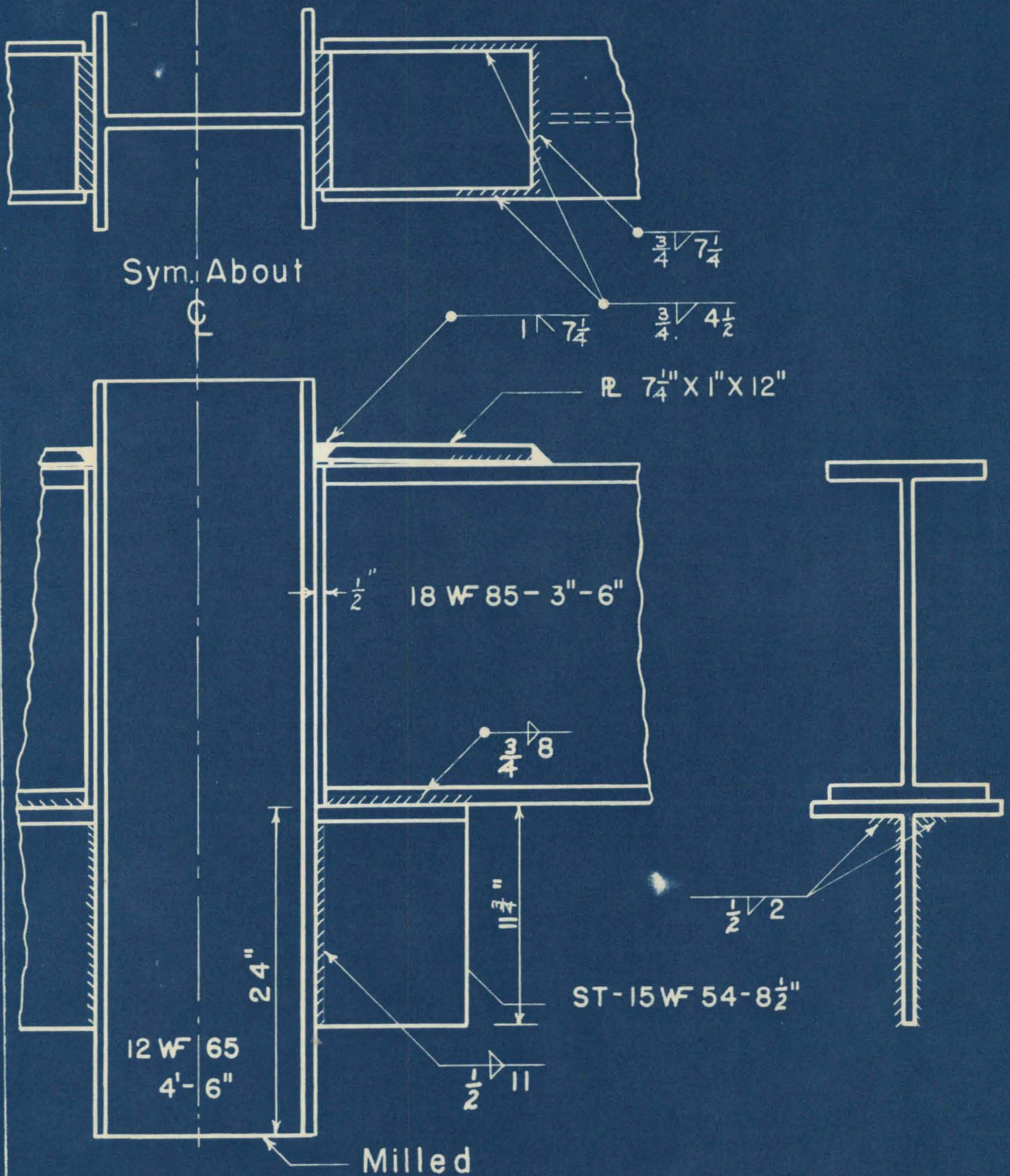
Specimen 5 was designed to compare the action of a flange connection with that of a web connection. It was, therefore, made the same as Specimen 4 except that a seat angle was used in place of a tee. For strict design procedure the top plate should have been heavier but this would have spoiled the attempted comparison. The effect of the lighter top plate was the same as if a slightly smaller length-depth ratio had been used in design.

The connection failed by tearing out the parent metal from the column flange at V - 118 kips, M = 3360 kip-in. Fig. 27(a) shows how the outside part of the flanges of the column were pulled out, while the portion near the web was torn.



Sym. About ℄
$\frac{5}{8}$ 7
$\frac{5}{8}$ 4
$\frac{7}{8}$ 7
PL $7\frac{1}{2}$" X $\frac{7}{8}$" X 12"
$\frac{1}{2}$
18 WF 85 – 3'-6"
$\frac{3}{4}$ 7
∠ 8" X 8" X $\frac{7}{8}$" – 10 "
24"
$\frac{1}{2}$ 8
$\frac{1}{2}$ $\frac{1}{2}$
12 WF 65
4'-3"
Milled

Fig. 21. TEST 5



Sym. About
℄
3/4 7 1/4
1 7 1/4
3/4 4 1/2
PL 7 1/4" X 1" X 12"
1/2"
18 WF 85 - 3"-6"
3/4 8
24"
11 3/4"
1/2 2
ST-15 WF 54-8 1/2"
12 WF 65
4'-6"
1/2 11
Milled

Fig. 22. TEST 9

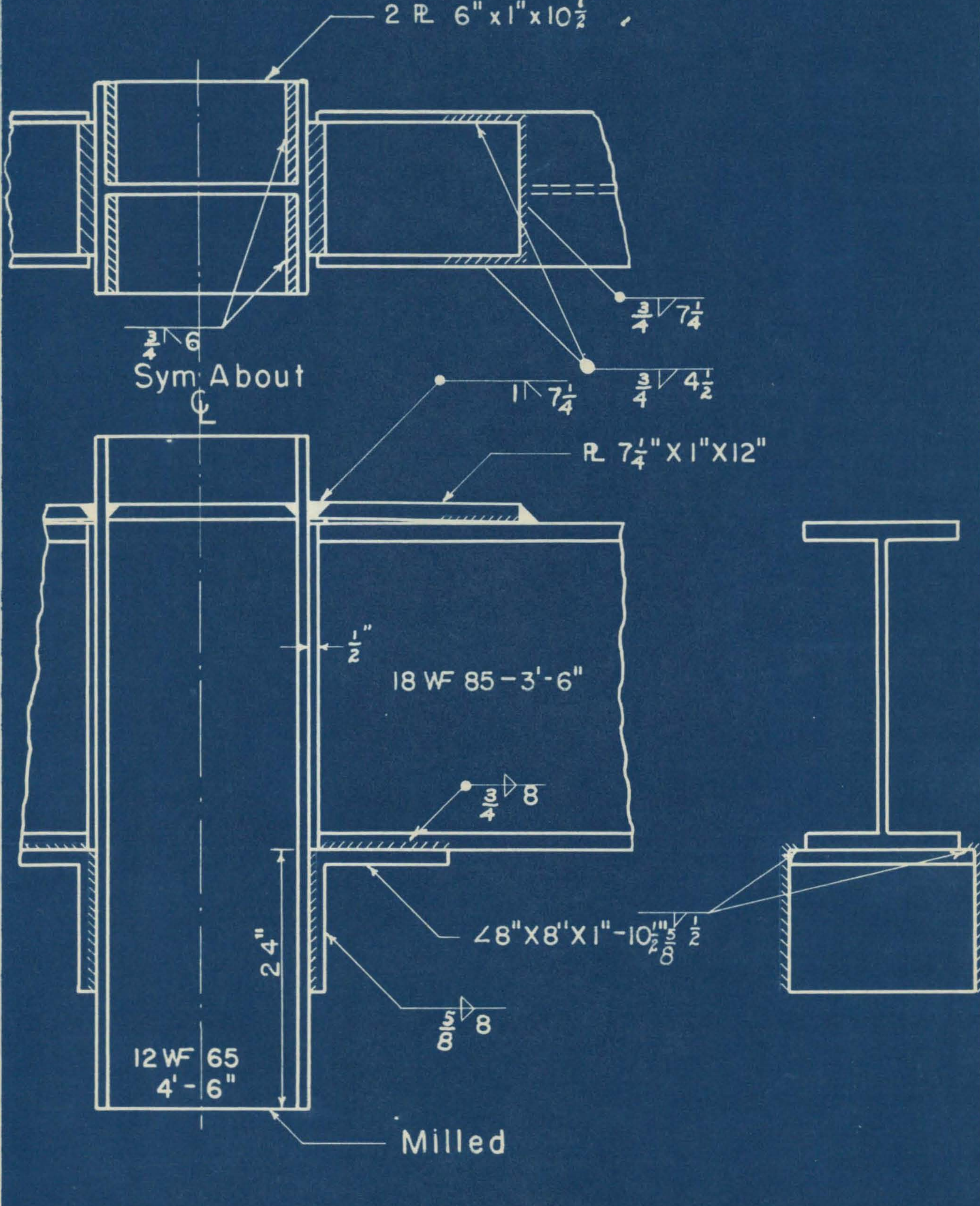


Fig. 23. Test 10

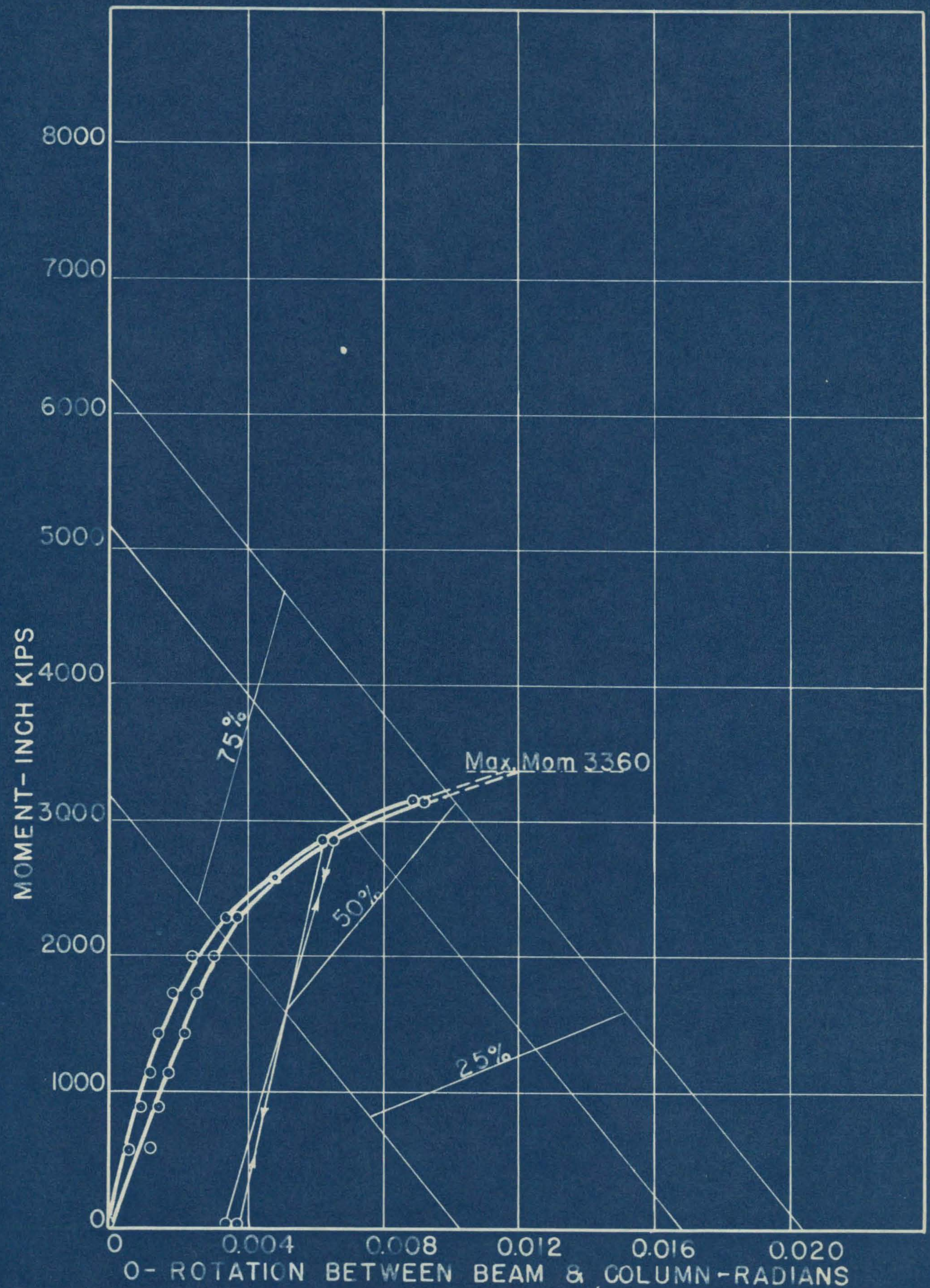


FIG. 24. MOMENT-ROTATION CURVES FOR TEST 5

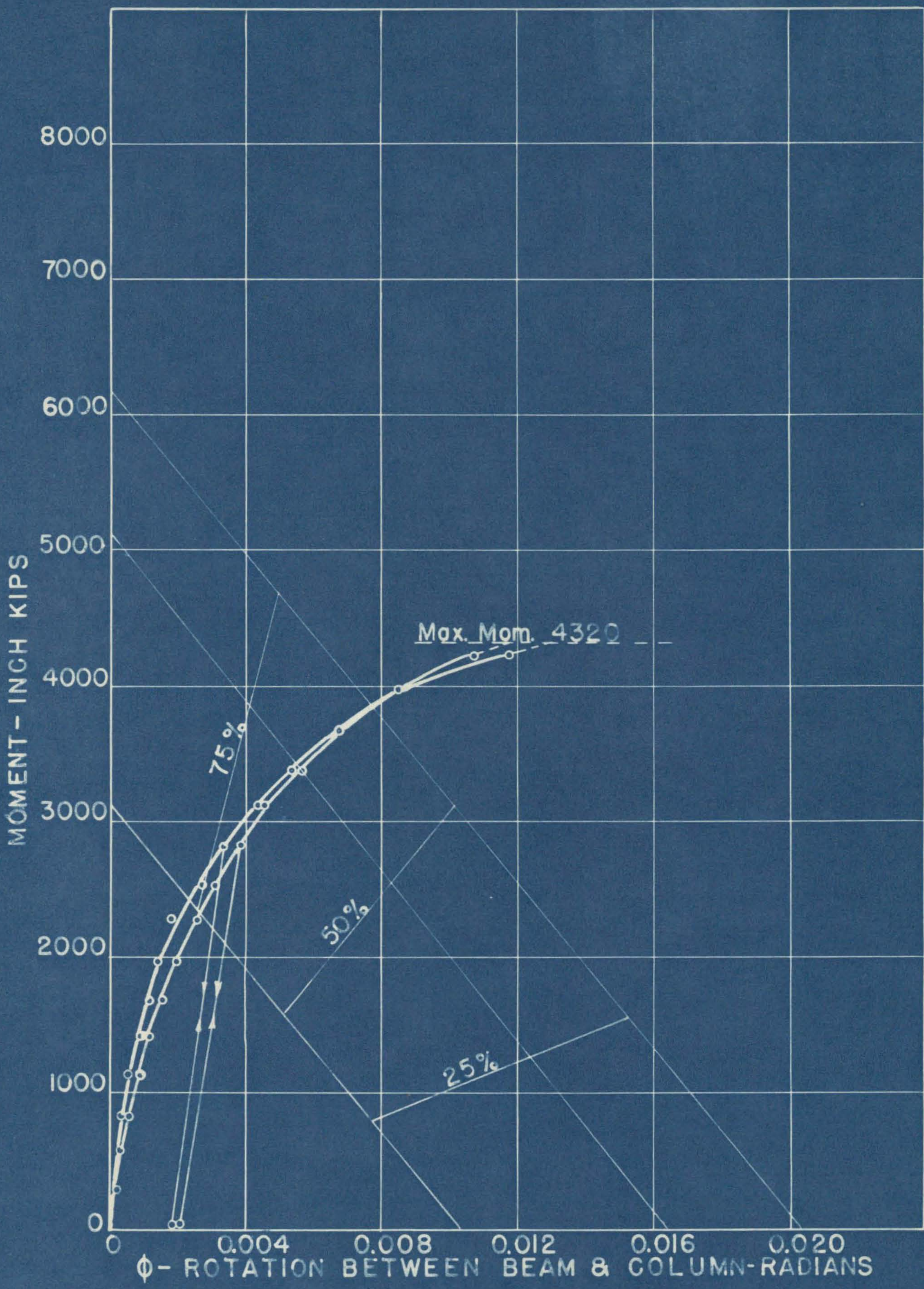


FIG. 25. MOMENT-ROTATION CURVES FOR TEST 9

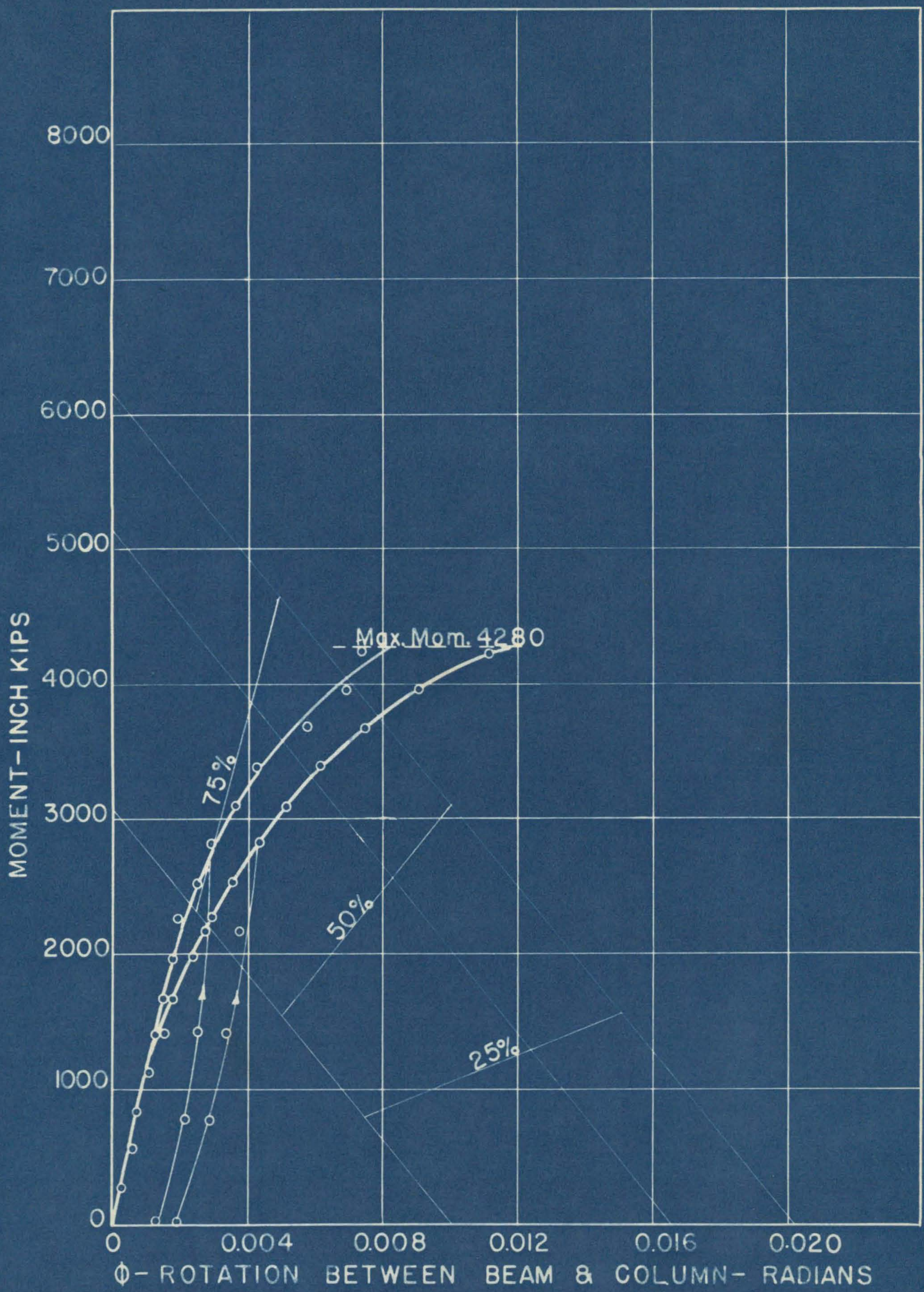


FIG. 26 MOMENT-ROTATION CURVES FOR TEST 10

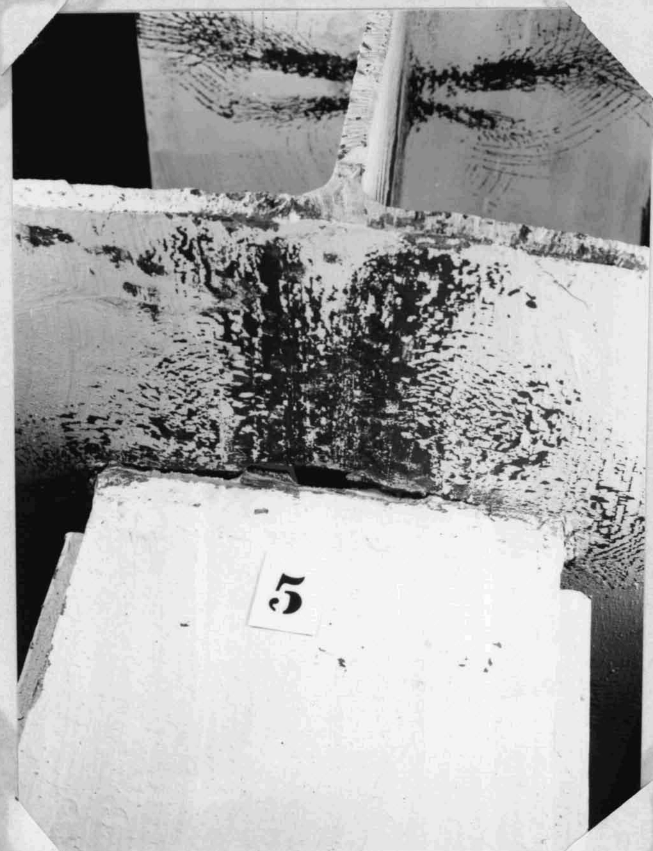


Fig. 27(a) - Failure Of Test 5

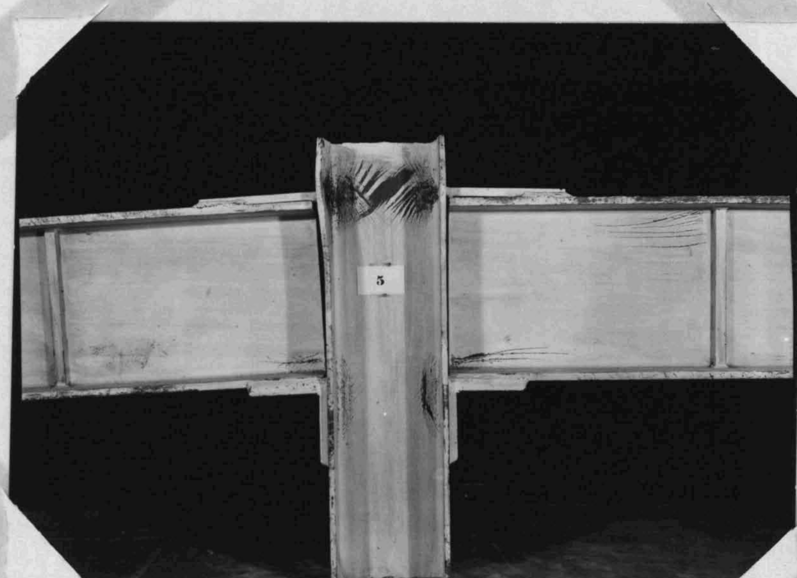


Fig. 27(b) - Test 5 After Failure

Fig. 28 - Failure In Test 9

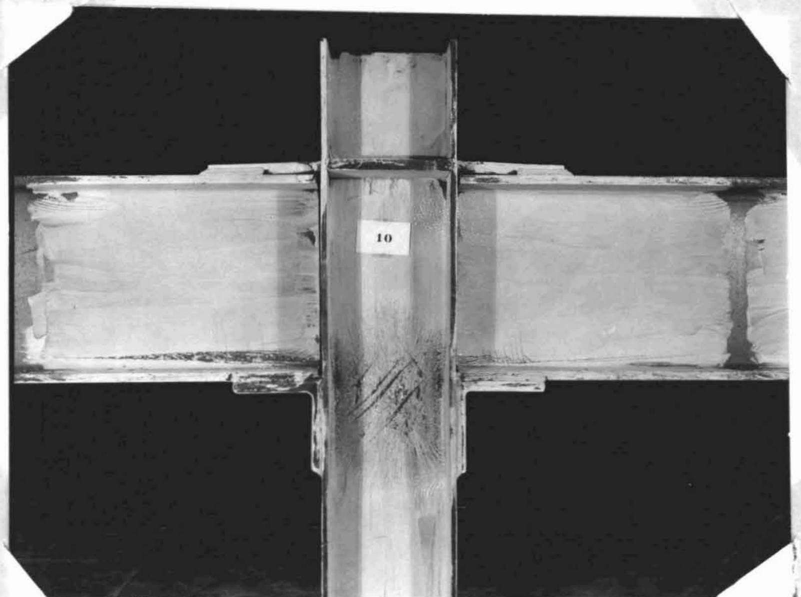


Fig. 29 - Test 10 After Failure

b. Test 9.

Specimen 9 had a seat tee similar to No. 4, but a heavier top plate. At V = 60 kips, M = 1695 kip-in., the top plates were 1/8-in. away from the beam flange. At V = 140 kips, M = 3960 kip-in., the column flanges were pulled out of line by the top plate, and at V = 150 kips, M = 4240 kip-in., the parent metal near the web in the column flange began to tear. At V = 153 kips, M = 4320 kip-in., maximum moment was reached and the tear of column metal increased. The photograph in Fig. 28 shows the test at the junction of the top plate and column flange.

c. Test 10.

Specimen 10 was similar to Specimen 5 except for heavier seat angles and top plates. A plate was added between the column flanges to prevent their being pulled out as in Test 5.

At V = 151 kips, M = 4270 kip-in., failure occurred by the buckling of the column web behind the seat angle. Apparently the extra plates between the column flanges cured the cause of failure at the top plate and shifted the difficulty to the seat.

23. Test 16.

Specimen 16 was designed after the behavior of Specimens 5, 9, and 10, had been observed. The top plate was not designed for connection to the column web, but was made wide enough to act as a stiffener for the column flanges in the same fashion as the extra plates in Specimen 10, with the butt welds acting in shear for the beam connection. The design details are shown in Fig. 30; the moment-rotation curve is shown in Fig. 31; and the top plate failure is shown in Fig. 32. Observations of strain lines during test are listed in Table III.

At V = 145 kips, M = 4990 kip-in., a weld ping was heard. Later examination of the specimen showed that the butt weld had broken away from the column web at the end of a weld pass which the welder had carried around to the web in finishing off the pass. The crack was parallel to the column web and in no way interferred with the structural action of the butt weld to the flange. At V = 165 kips, M = 5690 kip-in., failure occurred by the tearing of the top plate at the edge of the fillet return. Fig. 32 shows the tear on the left side of the top plate.

24. Test 17.

At the corners of buildings, beams are frequently framed into one side only of the column web. Specimen 17 was designed to simulate corner framing conditions. The design details are shown in Fig. 33; the moment-rotation curve in Fig. 34; and the general appearance of the specimen after test is shown in Fig. 35. Observations of strain lines during test are noted in Table III.

The strain lines noted in the column web at the base of the seat tee for V = 40 kips, M = 1370 kip-in., later spread until they completely encircled the tee. At

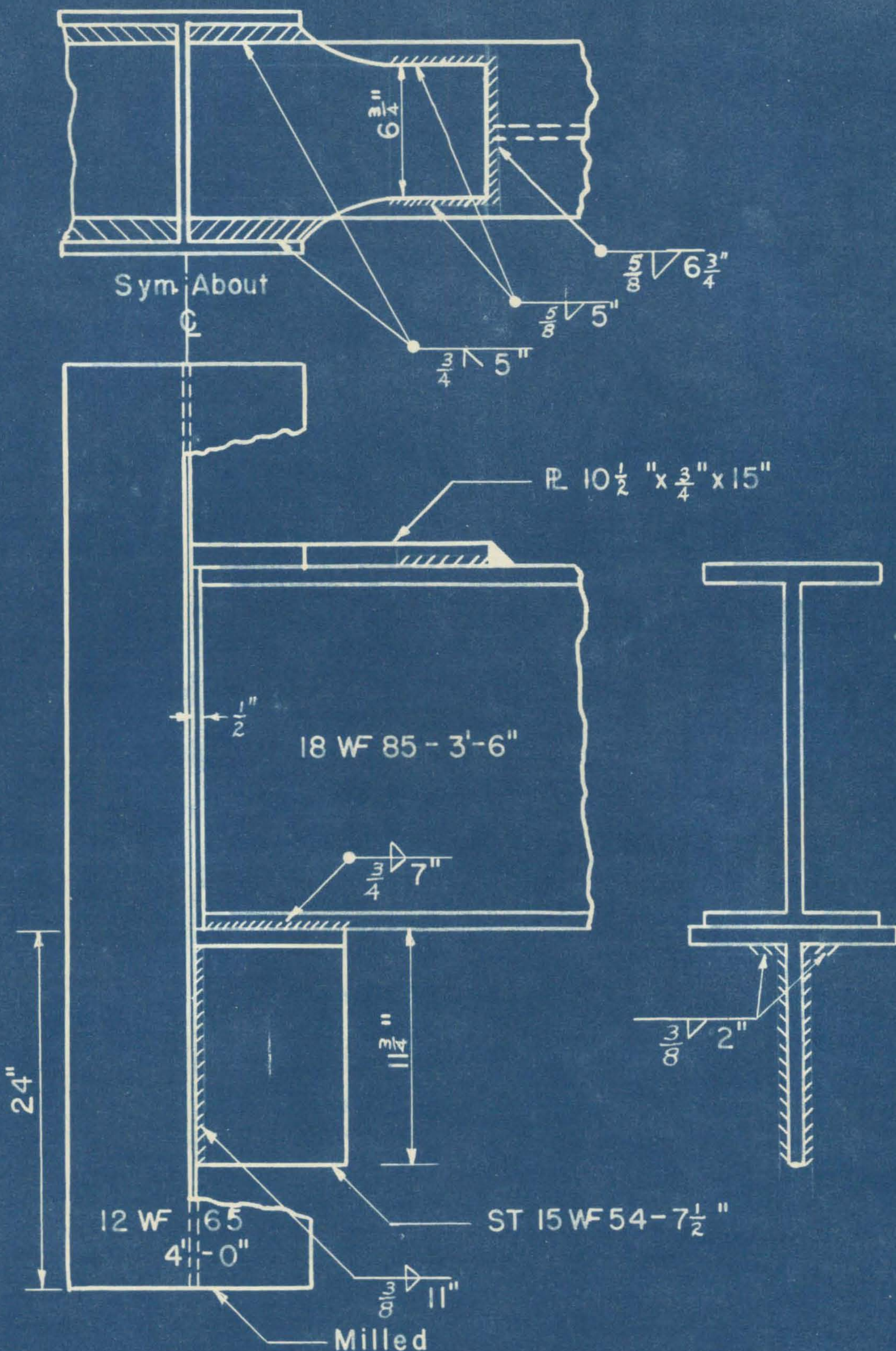


Fig. 30. Test 16

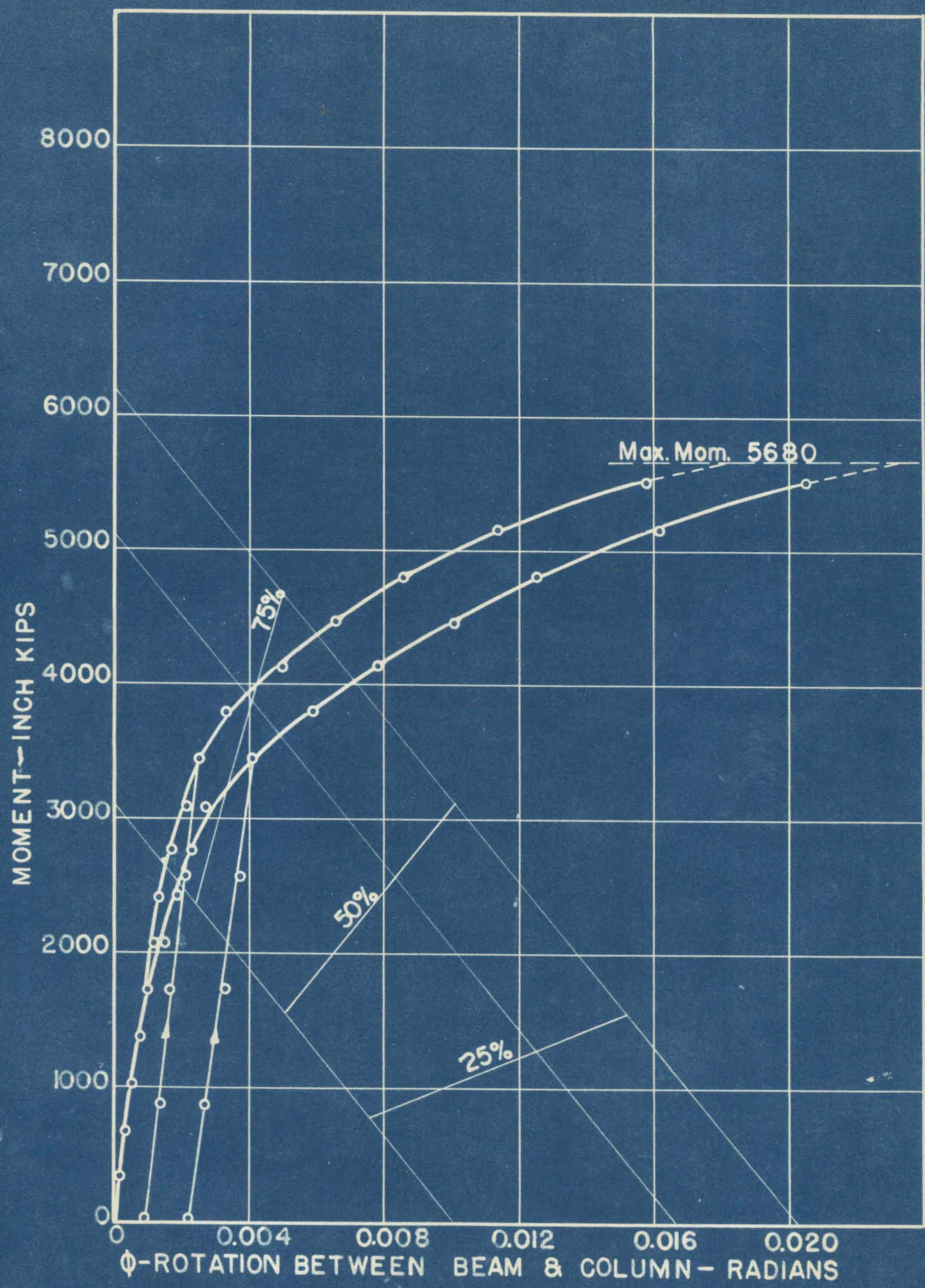


FIG. 31. MOMENT-ROTATION CURVES FOR TEST 16

Fig. 32 - Top Plate of Test 16
After Failure

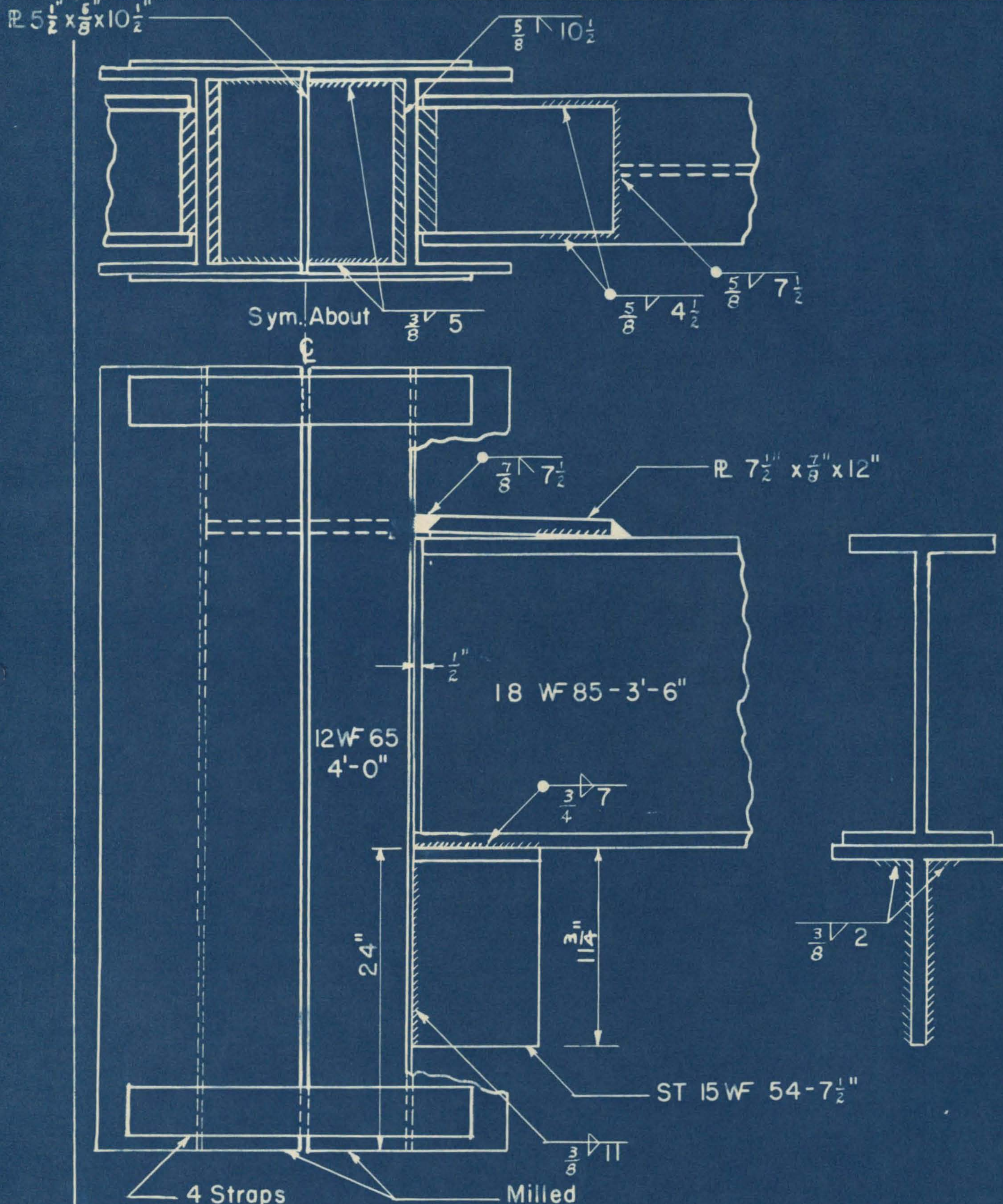


Fig. 33. TEST 17

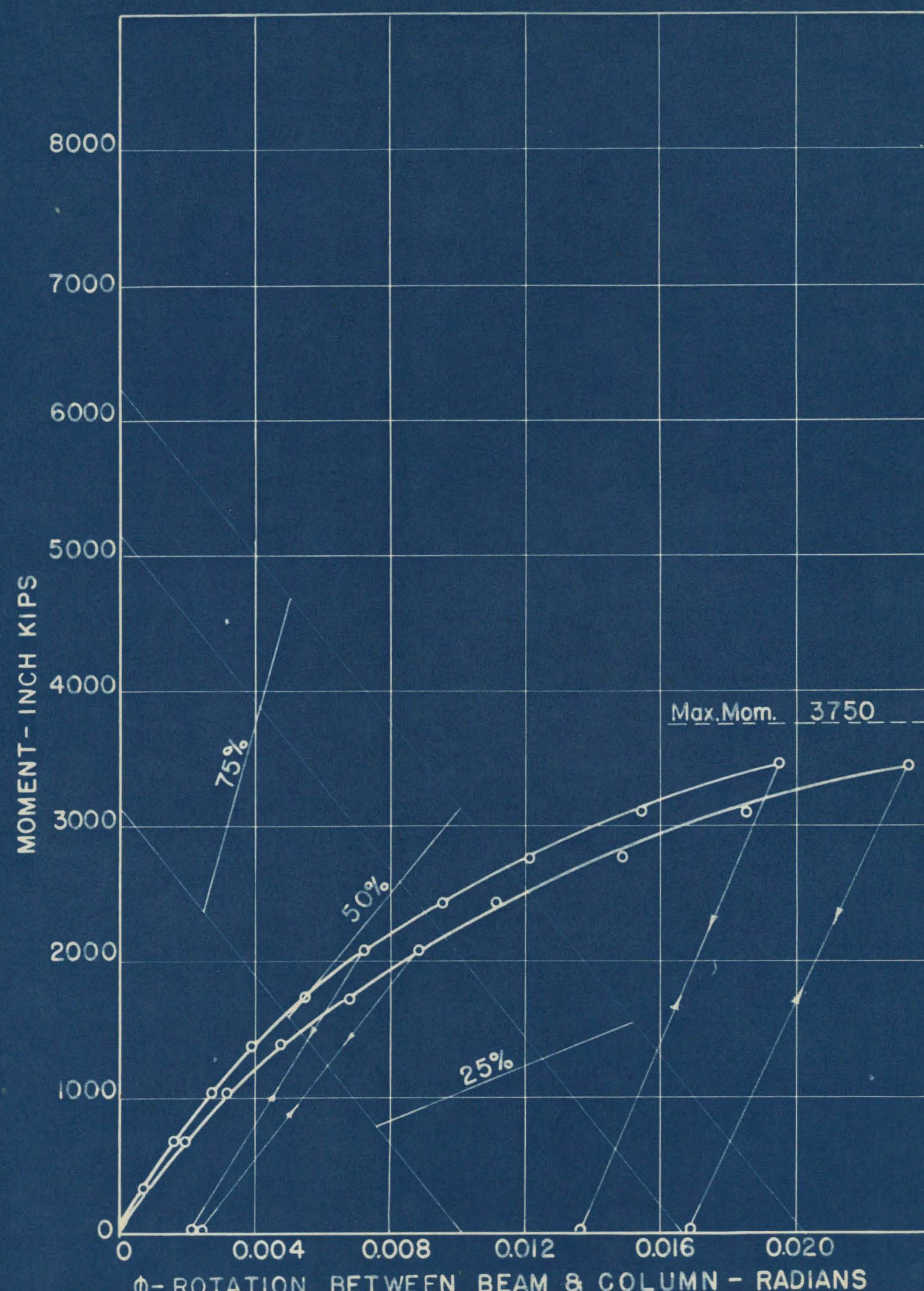


FIG. 34. MOMENT-ROTATION CURVES FOR TEST 17

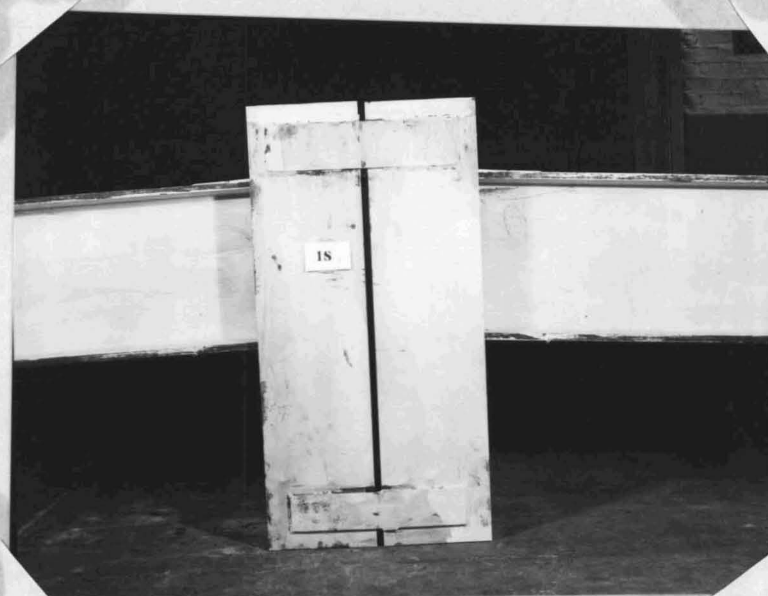


Fig. 35 - Test Specimen of Beam Welded to One Side of the Column Web

V = 109 kips, M = 3760 kip-in., the top plate suddenly pulled out half the thickness of the column web. Fig. 36 (a) shows this failure as it appeared after test, and Fig. 36(b) shows the oxidized surface at each end of the fracture after the plate had been bent down through 90 degrees.

25. Tests 11, 15, 13, 14.

A series of six specimens was made to determine whether or not a light top plate with a reduced or "throat" section would give a satisfactory flexible connection. Specimens 11 and 15 were web connections and Specimens 13 and 14 were flange connections. Design details are shown in Fig. 37 to 40; moment-rotation curves are plotted in Fig. 41 to 44; and a photograph of a typical failure is shown in Fig. 45. Observations of strain lines during test are listed in Table III.

a. Test 11.

Specimen 11 was intended to be a web connection similar to Specimen W-21 reported in reference (7) but with a reduced section top plate. At V = 59.3 kips, M = 1186 kip-in., a maximum load was reached and the top plate began to tear. The support pedestals were moved in to reduce the moment arm so that shear design requirements could be checked. This procedure was justifiable since the moment-rotation curve indicated a connection rigidity which would produce an inflection point closer to the column than the "a" distance used. The specimen was reloaded to V = 117 kips, M = 1520 kip-in., at which load the tear in the top plate became serious, preventing further application of load.

Fig. 36(a) - Failure of Test 17

Fig. 36(b) - Pipe In Column Web Of Test 17

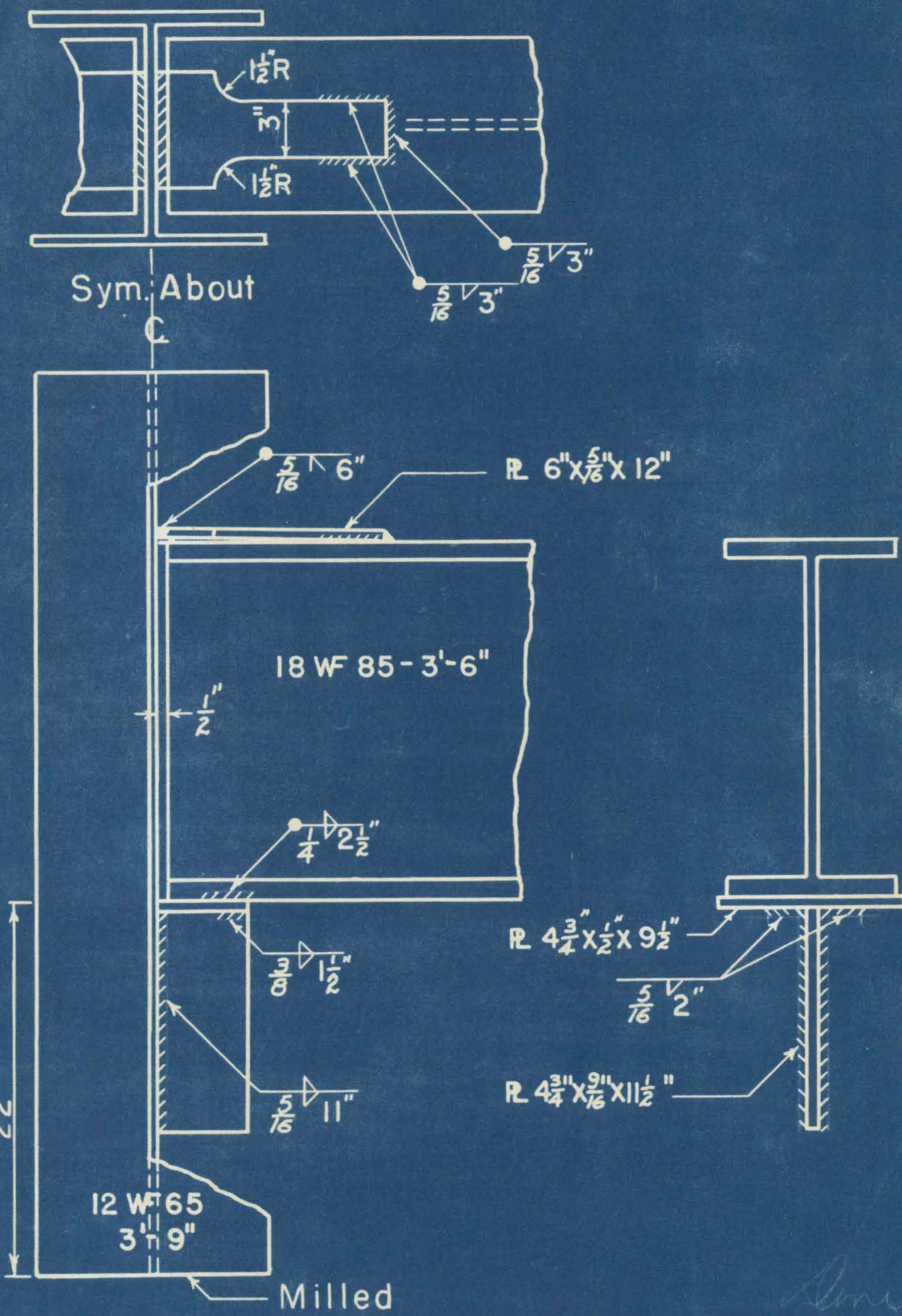


Fig. 37. Test 11

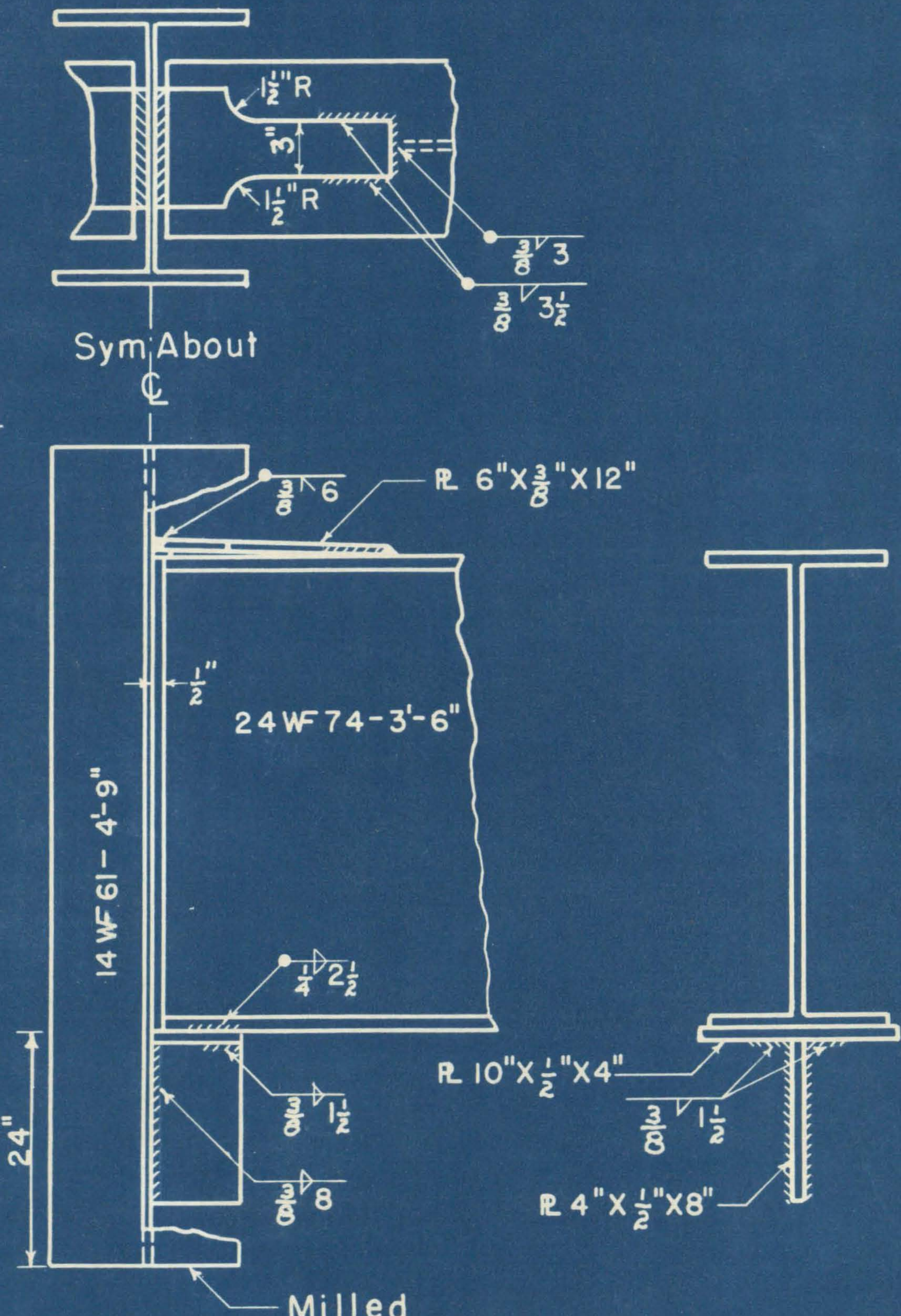


Fig. 38. TEST 15

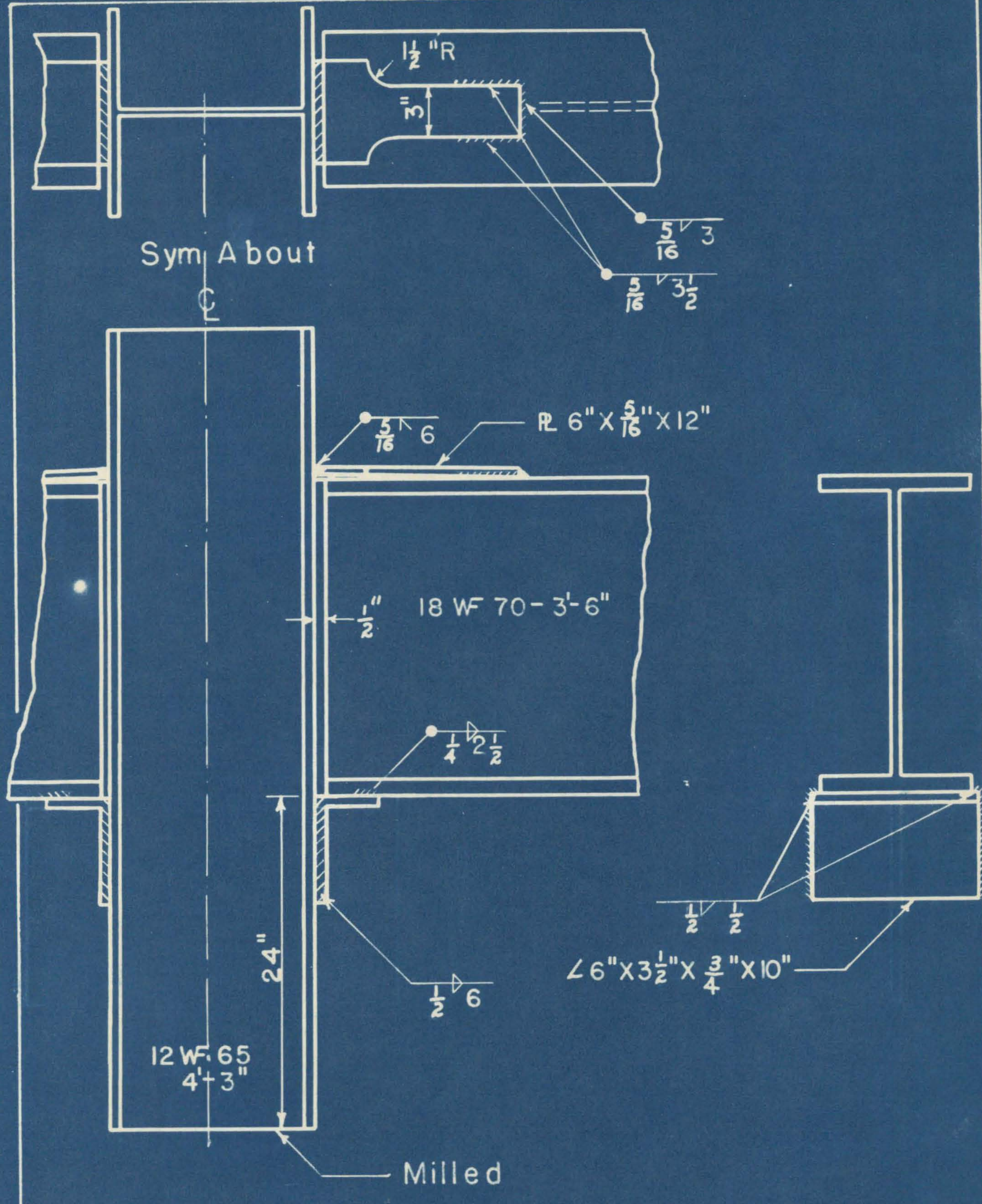


Fig. 39. TEST 13

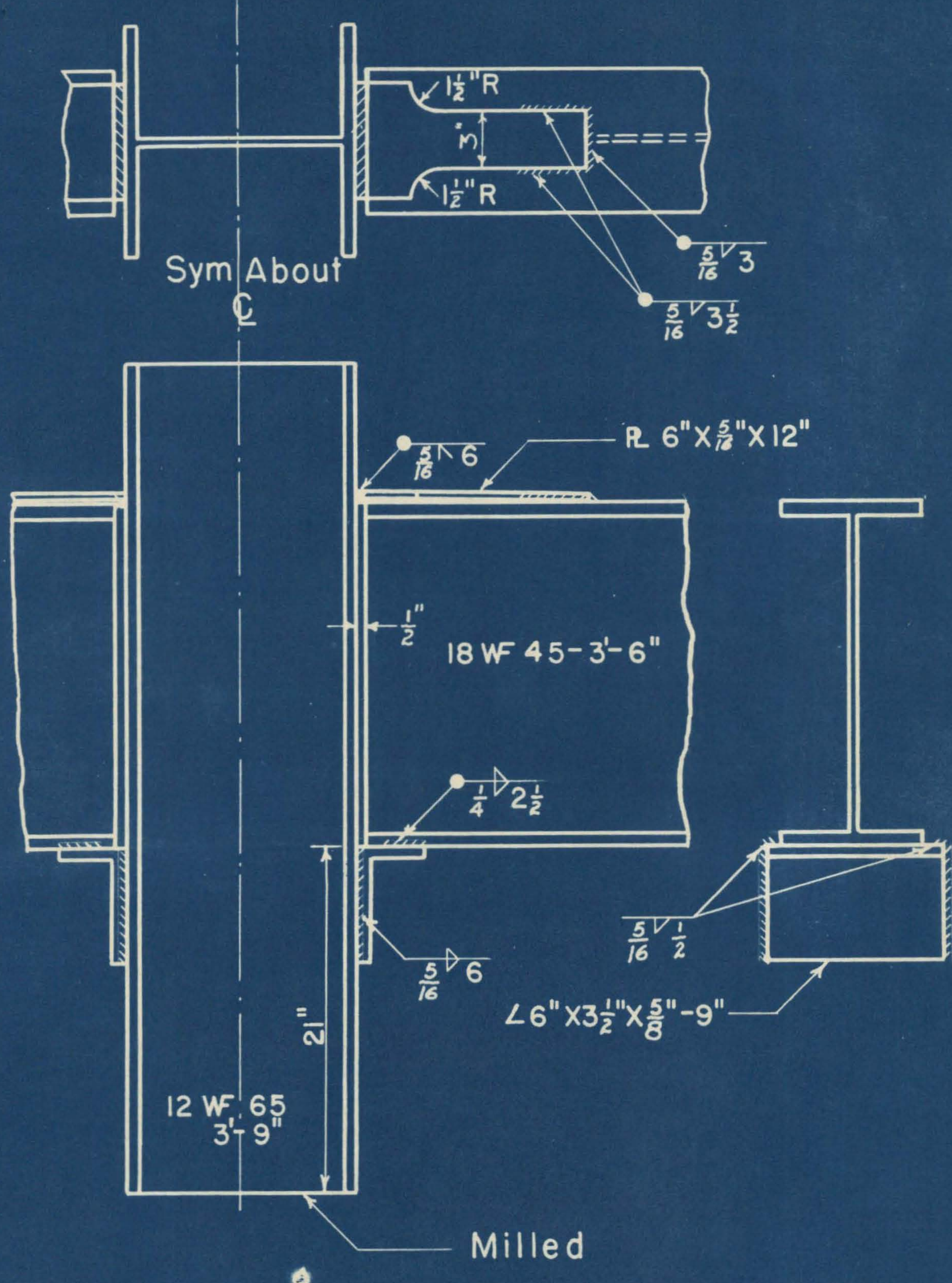


Fig. 40. TEST 14

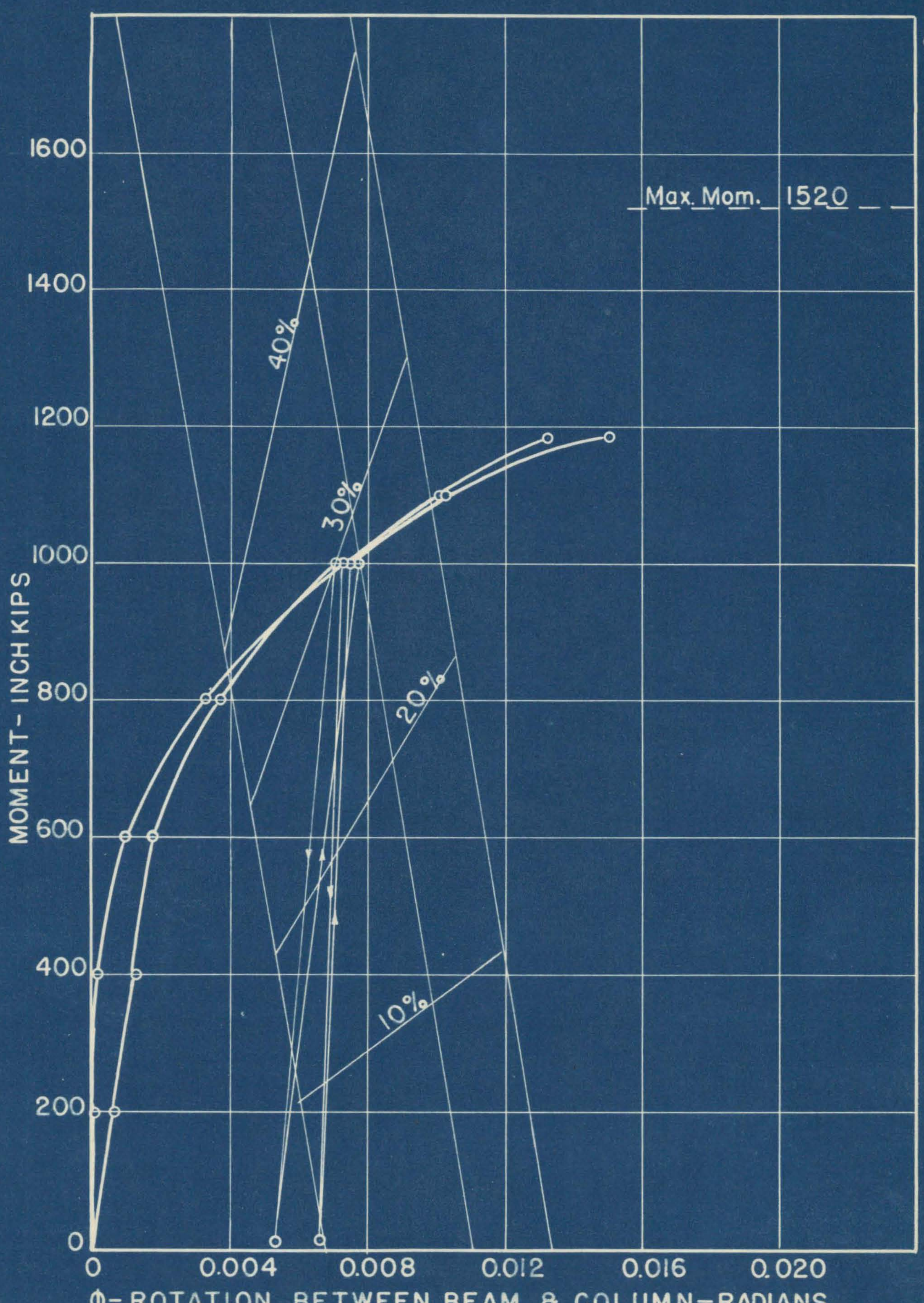


FIG. 41. MOMENT-ROTATION CURVES FOR TEST II

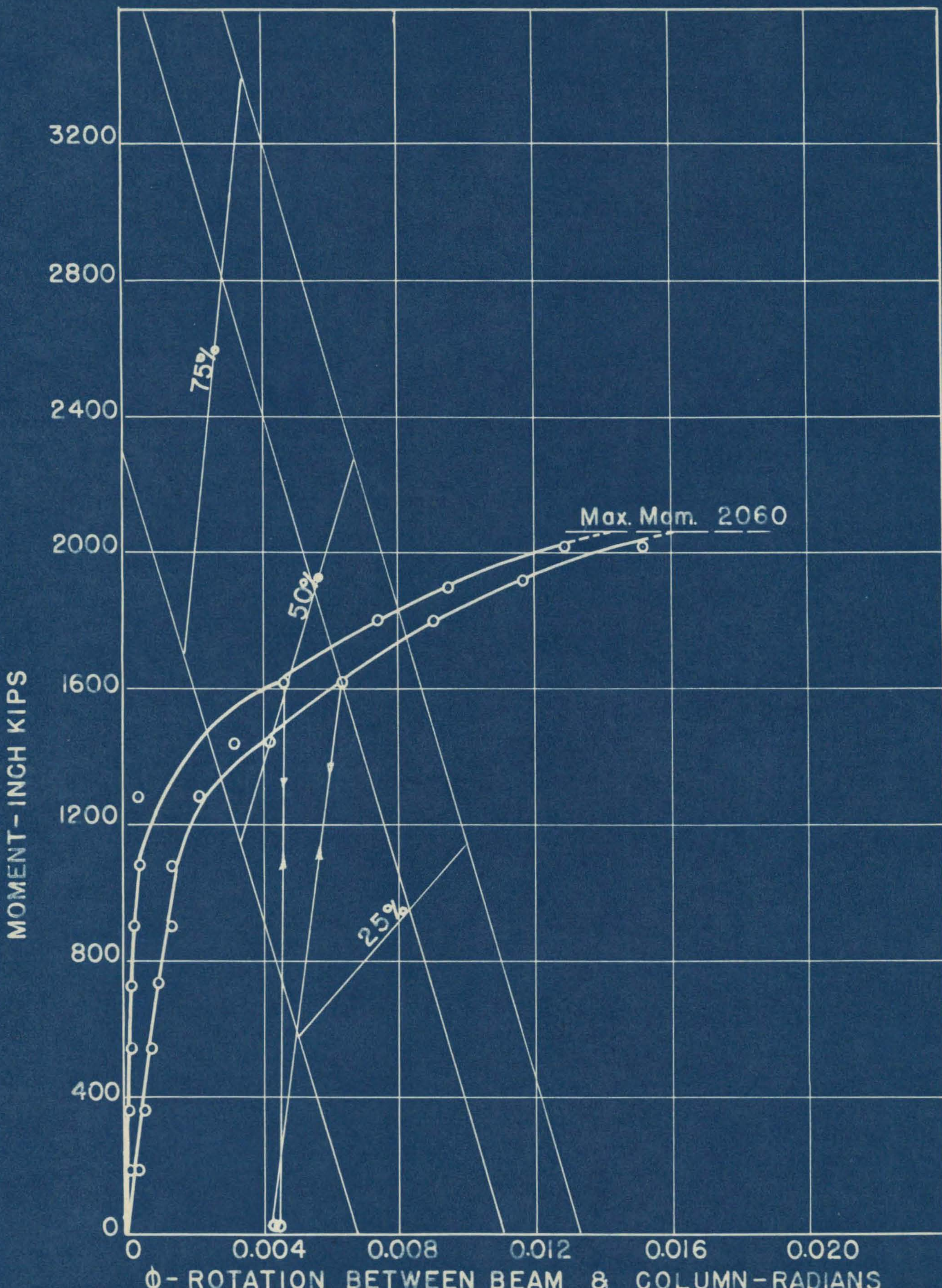


FIG.42.MOMENT-ROTATION CURVES FOR TEST 15

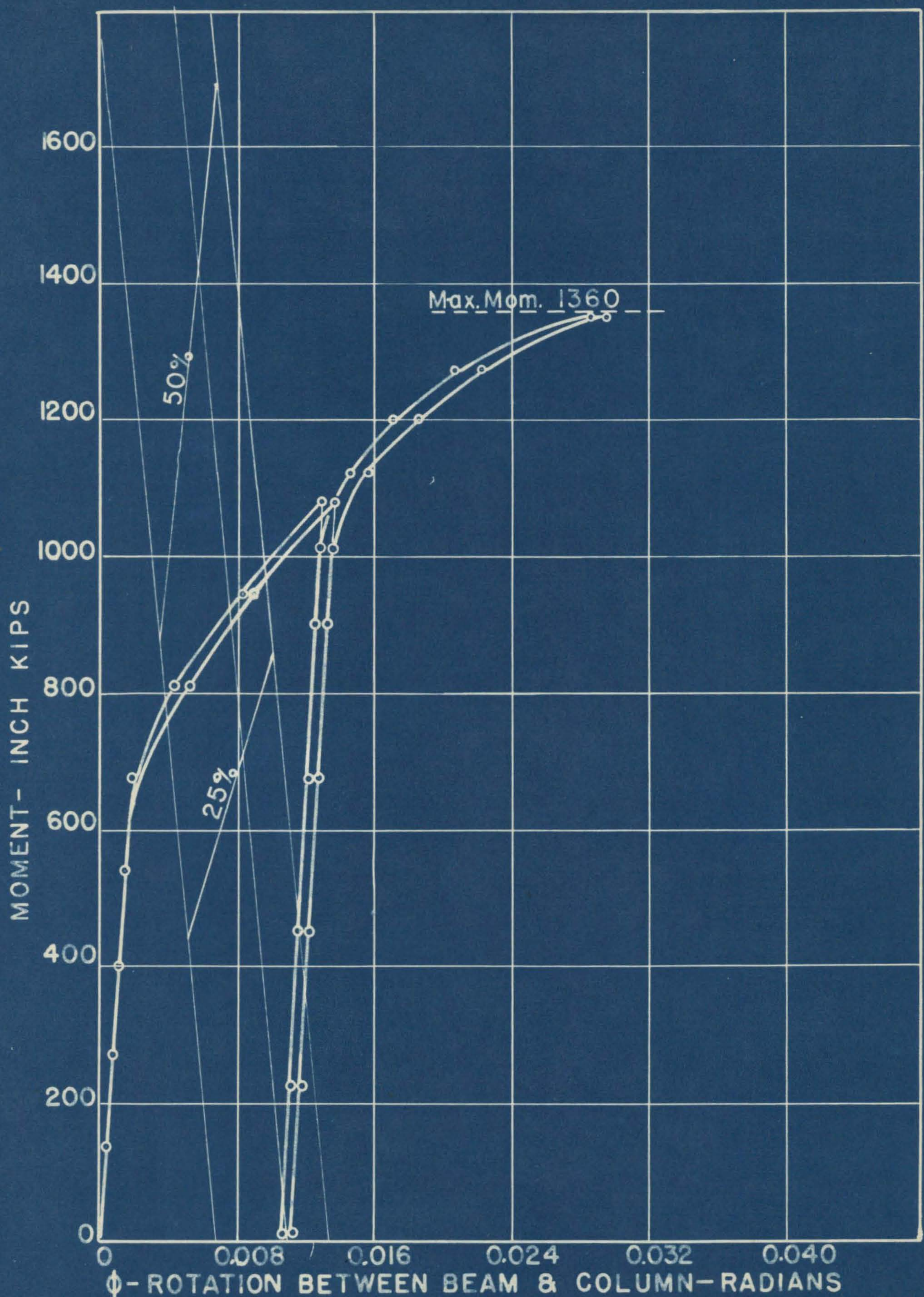


FIG. 43 MOMENT-ROTATION CURVES FOR TEST 13

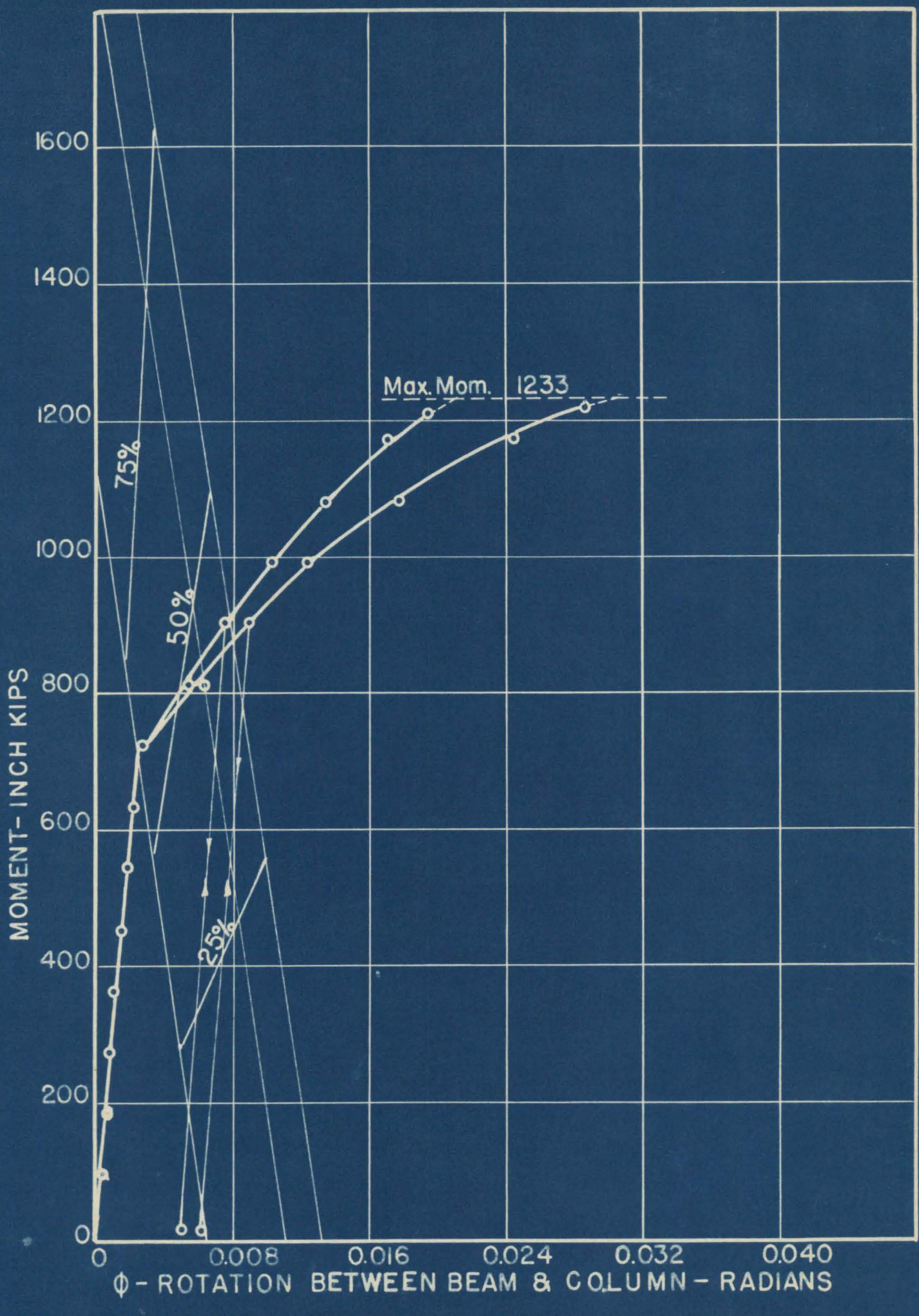


FIG. 44 MOMENT-ROTATION CURVES FOR TEST 14

Fig. 45 - Typical Failure
of Flexible Top Plate

b. Test 15.

Specimen 15 was a web connection with a light 24-in. beam. At V = 52.5 kips, M = 1260 kip-in., strain lines developed in the wide portion of the top plate, and later spread into the narrow portion of the plate. This spread of strain lines was typical of the plates with reduced sections. At V = 85 kips, M = 2020 kip-in., the top plate had pulled 1/8-in. away from the beam flange. The top plate tore at the end of the fillet weld return causing failure at V = 86 kips, M = 2064 kip-in.

c. Test 13.

Specimen 13 was a flange connection with a heavy 18-in. beam. The beams used were heavily rusted, hence the observations of strain lines were probably not as dependable as in the specimens with an intact coating of mill scale. As much rust as possible was removed with a wire brush and blow torch before fabrication.

At V = 60 kips, M = 1080 kip-in., the support pedestals were moved in to give a shorter moment arm. The unloading and reloading curves are shown in Fig. 43. At V = 90.5 kips, M = 1360 kip-in., the specimen failed with a tear through the top plate at the end of the fillet-weld return.

d. Test 14.

Specimen 14 was a flange connection intended to be similar to Specimen W-9 reported in reference (7), but with a reduced section top plate. At V = 68.5 kips, M = 1233 kip-in., the top plate failed by a tear at the end of the fillet weld return.

26. Tests 12 and 18.

The connections for Specimens 12 and 18 were nearly the same as for Specimen 11, but the specimens were made up as single-side connections similar to Specimen 17. Design details are shown in Fig. 46 and 47, and the moment-rotation curves are shown in Fig. 48 and 49. Observations of behavior during test are listed in Table III.

a. Test 12.

The column webs for Specimen 12 were unreinforced, hence they were pulled out of shape by the top plates at rather low loads. At V = 46.8 kips, M = 936 kip-in., the top plate pulled the parent metal out of the column web at its edges. This failure is shown in Fig. 50. In the moment-rotation curve, the normal beam rotation was increased by the bowing of the column web, as may be seen by comparing Fig. 48 and 49.

b. Test 18.

Specimen 18 was similar to Specimen 12 except for extra plates which stiffened the column webs at the top plates. Some other small differences were present because of the lack of identical materials.

At V = 98.5 kips, M = 1580 kip-in., the top plate failed by tearing at the edge of the fillet weld return in the same fashion as shown in Fig. 45. As in Specimen 10, the reinforcing plates on the column webs at the top plate showed the weakness of the column web at the seat, since the web buckled in and showed heavy strain lines around the tee.

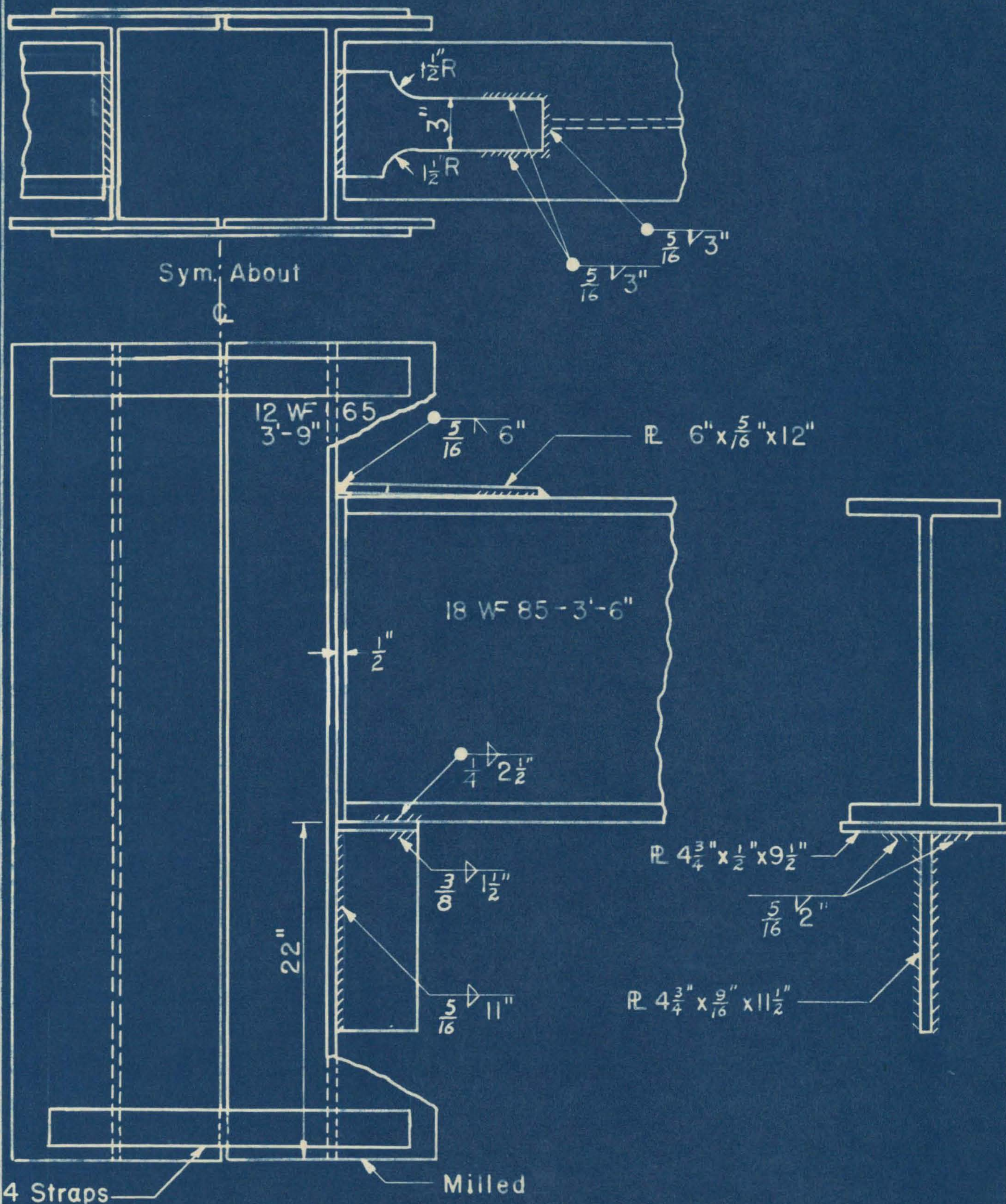


Fig. 46. TEST 12

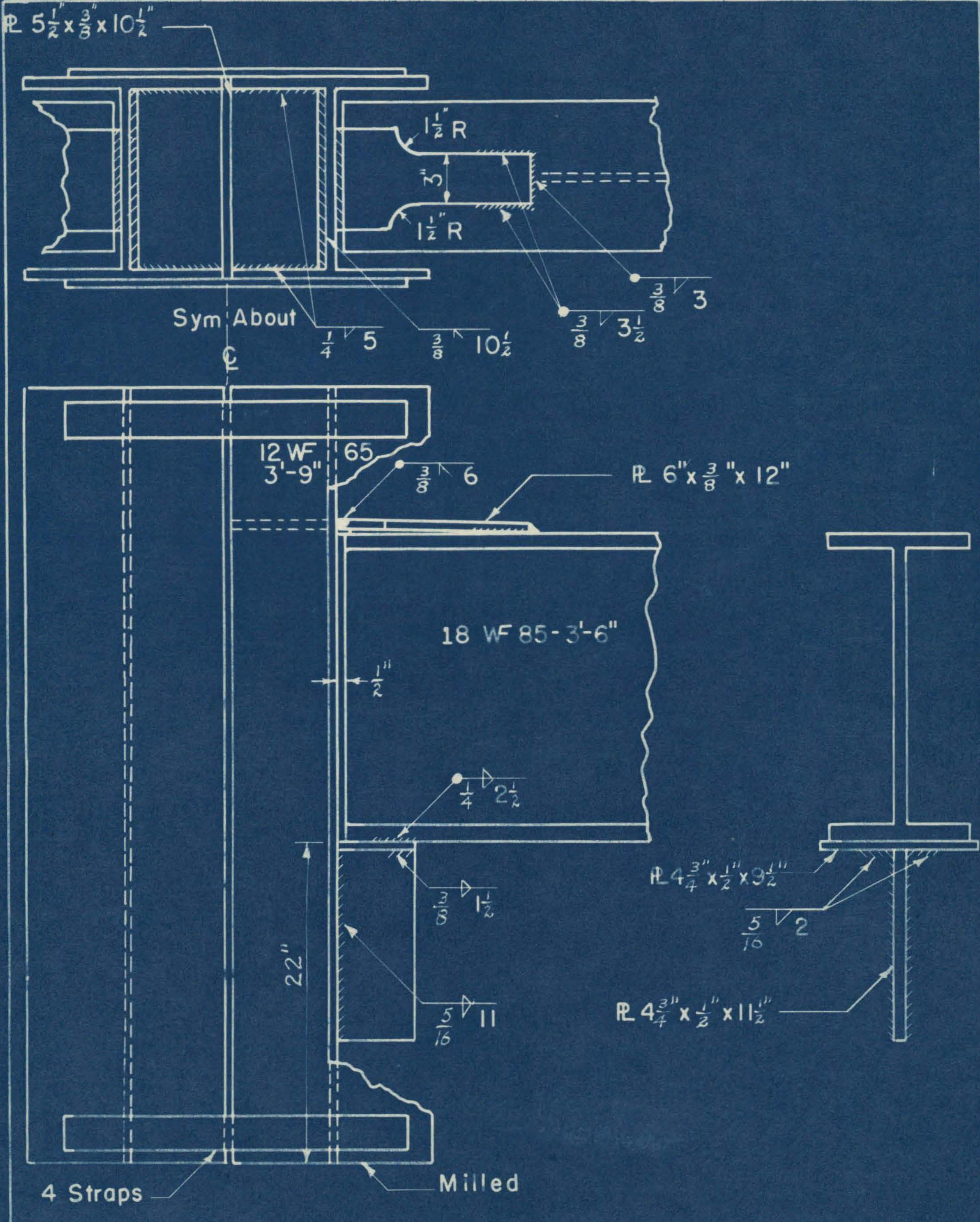


Fig. 47. TEST 18

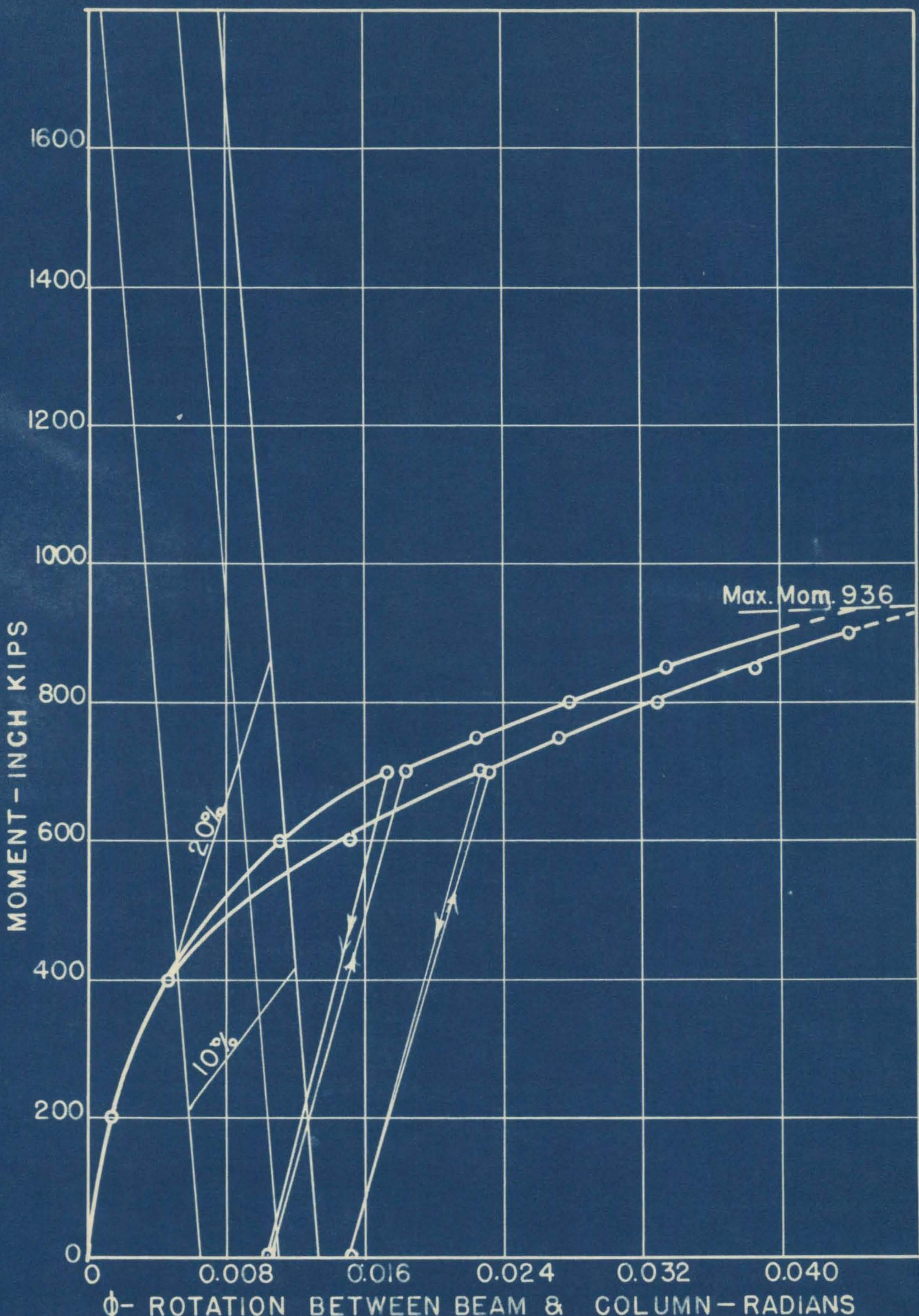


FIG.48.MOMENT-ROTATION CURVES FOR TEST 12

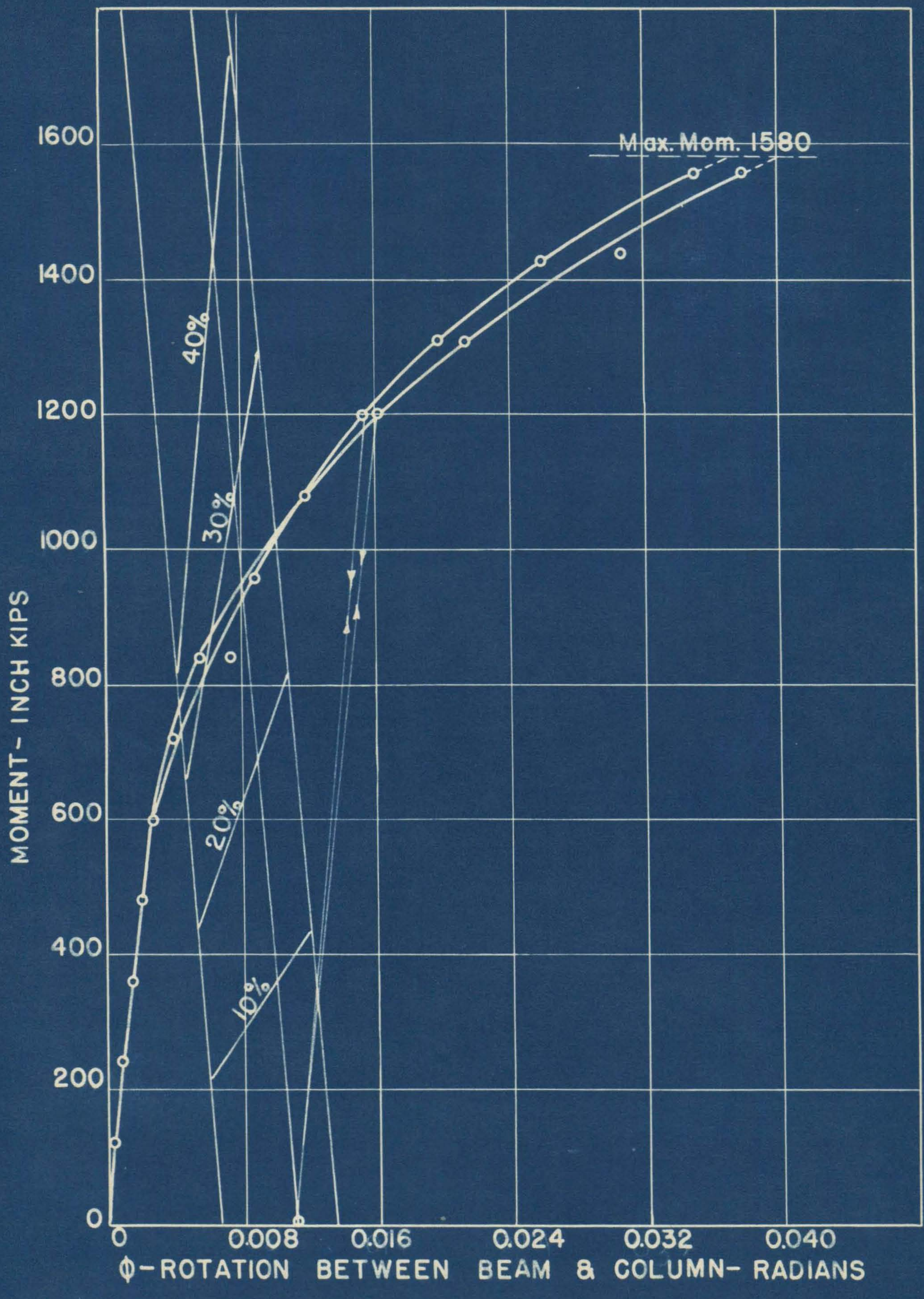


FIG.49.MOMENT-ROTATION CURVES FOR TEST 18

Fig. 50 - Failure of Test 12

27. Typical Strain Patterns.

Fig. 18 shows the strain lines which formed in the web of the seat tee for Specimen 1. These strain lines were typical of ones developed on seat tees. Fig. 51 shows the strain lines formed in the flange and web of the beam over the seat tee for Specimen 7. This was the usual form of strain lines over the seat tee. Fig. 52(a) shows the strain lines formed in the flange and web of the beam over the seat angle of Specimen 14. Fig. 52(b) shows the strain patterns in the column web around the seat tee of a single-side connection, Specimen 17.

VII. Conclusions.

28. Behavior of Test Specimens.

a. Tests 1, 2, 3, 4.

All specimens except No. 4 failed by buckling of the beam web over the seat tee. The development of such a failure should mean that the connection details at top and bottom of the beams had more strength than the beams were capable of developing, and might therefore be overdesigned.

The "brittle" fracture of the top plate of Specimen 4 in metal which should normally have been ductile, was unexpected and was thought to be quite unusual. Possible explanations of the brittle fracture might be:

(1) A tri-axial tension state of stress may have been developed at the edge of the fillet return, though this could not very well have been true at the center of the top plate where the fracture was also brittle in character.

Fig. 51 - Strain Pattern On Beam
Over Tee

Fig. 52(a) - Typical Strain Pattern
In Beam Over Angle

Fig. 52(b) - Strain Pattern
In Column Web
Around Tee

(2) The welding process may have caused a brittle structure to be formed near the fillet weld, but again this does not explain the brittle fracture exhibited at the center of the top plate. Also, the size of the weld and the accompanying high heat input during welding make the formation of a brittle structure rather improbable.

(3) A bi-axial tension state of stress probably existed in the top plate along the fracture line. If the "normally ductile steel" had some peculiar characteristics which would make it behave in a brittle fashion under bi-axial tension, then that characteristic would explain the failure observed.

(4) A brittle crack was formed in the top plate at the end of the fillet weld return as discussed in either item (1) or (2). The cross-sectional area under tension was thereby reduced considerably. If the crack had been formed rather quickly, the remaining area could have been subjected to a sudden increase of stress as the testing machine recovered its elastic strain. This sudden increase of stress on the plate and the notch effect of the cracks could cause a brittle failure in a sufficiently notch-sensitive material.

b. Tests 6 and 7.

The failures of these two specimens were almost identical with that of No. 4, with brittle behavior in what should normally have been ductile material. The fracture of

Specimen 7 was brittle near the weld and ductile in the center of the plate, perhaps because the loading was stopped about one-half hour while a picture was taken.

c. Tests 5, 9, 10.

The flange-connection specimens showed the weakness of light columns for two-way connections with heavy connection details. The pulling-out of the parent metal in the center of the column flange may have been a direct result of the use of column sections with light flanges. The use of columns with heavier flanges might prevent the type of failure observed.

The restraint of the column flanges afforded by the interior reinforcing plates as in Specimen 10 was sufficient to prevent failure at the top plates, but showed the weakness of the light column web at the seat. The tests afforded no information concerning behavior of a specimen in which the reinforcing plates between column flanges were top plates for beams framing into the column web with moment on all four connections.

The seat angle of Specimen 5 was less stiff than the tee of 9, as shown by the center of rotation figures in Table IV.

d. Test 16.

The behavior of the top plate in Specimen 16 showed the possible usefulness of such a detail to reinforce the column flanges in a four-way connection and at the same time to act as top plate for the web connections.

TABLE IV

SUMMARY OF TEST RESULTS

Specimen No.	Specimen Type*	Design Shear kips	Maximum Applied Shear kips	Ratio of Maximum Applied Shear to Design Shear	Design Moment kip-in.	Maximum Applied Moment kip-in.	Ratio of Maximum Applied Moment to Design Moment	Approximate Rigidity at 1.00 X Design Line	Center of Rotation in % of Depth from Bottom
1	2	3	4	5	6	7	8	9	10
1	WD	18.2	76.4	4.18	405	1720	4.24	90	60
2	WD	36.6	127.4	3.48	840	2930	3.49	90	55
3	WD	31.3	123.0	3.93	1045	4090	3.91	90	60
4	WD	54.0	192.0	3.55	1860	6630	3.57	80	--
6	WD	54.0	173.0	3.20	1860	5960	3.21	85	45
7	WD	54.0	174.0	3.22	1860	6000	3.23	80	60
5	FD	60.4	118.0	1.95	2070	3360	1.62	65	10
9	FD	60.4	153.0	2.53	2070	4320	2.08	75	40
10	FD	60.4	151.0	2.50	2070	4270	2.06	75	35
16	WD	54.0	165.0	3.05	1860	5690	3.06	80	50
17	WS	54.0	109.0	2.02	1860	3760	2.02	50	100
11	WD	45.4	117.0	2.57	--	1520	--	35	10
15	WD	38.1	86.0	2.26	--	2064	--	60	45
13	FD	38.1	90.5	2.38	--	1360	--	40	0
14	FD	24.0	68.5	2.85	--	1233	--	60	0
12	WS	45.4	46.8	1.03	--	936	--	20	--
18	WS	45.4	98.5	2.17	--	1580	--	35	80

* WD = web connection, both sides
WS = web connection, single side

FD = flange connection, both sides
FS = flange connection, single side

However, what this detail might accomplish in a four-way connection can not be predicted from the test.

e. Test 17.

The reinforcing of the column web on the single-side connections of Specimen 17 would probably have been successful had the column web itself been good. The frequency with which such a separated or "piped" column web might be expected to occur at a connection in actual design is a matter outside the scope of this report.

f. Tests 11, 15, 13, 14.

The top plates on these specimens permitted ample rotation of the beams with respect to the columns, but appreciable rigidities were developed. The rigidity at 2.00 X design load was from twenty to thirty per cent less than the rigidity at 1.00 X design load. Hence, though the connections were in the semi-rigid range at 1.00 X design load, at 2.00 X design load they were close to or within the flexible range.

Smaller values of rigidity and greater rotations could probably have been obtained with lighter and longer top plates.

g. Tests 12 and 18.

The flexible single-side connection of Specimen 12 showed that reinforcement of the column web at the top plate was required to prevent distortion and premature failure at that point. Specimen 18 had reinforcing at the top plate, and therefore developed a higher failure moment. There were signs of distress around the seat tee which indicated that reinforcement of the column web might be desirable at that point also.

29. Design Method.

a. The test results in general showed that the method of design of specimens gave figures which were reasonable. Moment-rotation behavior could be predicted with acceptable accuracy, except for unusual failures.

b. The weld joining the beam stub to the seat tee may have been oversized in terms of loads that the small welds on the flexible series were able to carry.

c. The test results indicated that care in detailing the connection is essential for good results.

30. Center of Rotation.

The figures given in Table IV for center of rotation are not considered to be more accurate than plus or minus five per cent, but they do serve to give some idea of the relative behavior of different connections.

31. Welds.

No failures in the welds occurred, hence it is believed that the tests represent tests of the structural action of the connections rather than of welder's skill.

32. Summary.

The results of the tests are summarized in Table IV. Column 3 lists the values of shear for which the specimens were designed and column 4 shows the maximum shear applied to the connection. Since there were no connection failures by shear, the values in column 4 do not represent ultimate shear strength. Column 5 shows the ratio of applied shear to design shear.

Column 6 lists the values of moment for which the specimens were designed, and column 7 shows the maximum moments which the connections withstood during test. Column 8 gives the ratio of maximum applied moment to design moment and represents something akin to a factor of safety for the connection in moment.

Column 9 lists the approximate rigidity of the connection as determined from the moment-rotation curves at the 1.00 X desigm beam line. The values of rigidity at 1.65 and 2.00 X design may be readily determined from the moment-rotation curves if such values are desired.

Column 10 lists the location of the center of rotation of the beam for all but two specimens. The values for these two specimens were considered too undependable to quote.

It should be noted that the moment arm for Specimen 12 was necessarily large, hence the applied shear was low in proportion to the applied moment. However, the connection of Specimen 11 was almost identical with that of No. 12, which indicates that the connection was capable of carrying at least 2.57 X design shear.

It seems fitting to interpolate a note of caution before closing this report. All of the tests reported herein were of beam stubs framing into one or two sides of a column, whereas in an actual building the connections would be to three or four sides of the column. Since it was impossible to perform four-way tests with the available testing machinery, the degree of correlation between two-way and four-way connections is unknown. Application of these test results to building designs should therefore be made with caution.