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Short steel columns progress report, 1939

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SHORT STEEL COLUMNS PROGRESS REPORT

by Lloyd T. Cheney*

This is a report of the progress made in the first year of a two-year investigation of short steel columns sponsored by the American Institute of Steel Construction. The investigation is a regular research fellowship of the Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania.

INTRODUCTION

The original program outlining the scope of the investigation was submitted to the Technical Research Committee on November 13th in the form of a preliminary report. Following receipt of comments by the individual members, the program was brought up-to-date in accordance with these suggestions by the addition of an appendix on December 19th. The program as outlined then may be summarized briefly as follows:

Ia - Flange Buckling.

Two tests each of silicon steel columns with flanges planed to five different thicknesses, making ten tests in all.

Ib - Flange Crippling of Columns with Loaded Beam Connections.

Three different column sizes using three types of connections under three different beam loads, making twenty-seven tests in all.

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II - Columns with End Moments.

About twenty-five or thirty columns of various slenderness ratios tested under a variety of ratios of axial stress to bending stress.

GENERAL PROGRESS

Much of the progress of the first year of such an investigation is of a rather intangible nature. Studies in theory directly connected with the problems at hand were made and thus a working foundation for the investigation was obtained. Test methods were tried out and in some cases found deficient to a greater or less degree. Various alterations in details were necessary before a smoothly functioning test set-up was obtained. Although the number of tests reported are few this intangible progress will not only greatly facilitate the remaining work, but will be an invaluable aid in the interpretation and presentation of the experimental results.

PROGRAM

Ia - Program Ia calls for the use of structural silicon steel H-sections with flanges machined to specified thicknesses in order to obtain the desired ratio between flange width and flange thickness. Fabricators bidding on the machining of these sections frankly admitted they anticipated difficulty in executing this work and submitted bids accordingly. First bids received were so exorbitant that it was thought that this part of the program might possibly have to be side-stepped. However, a rational agreement was finally reached with the Bethlehem Steel Company whereby the fabrication cost of these specimens was reduced to within the limits of the investigation budget. Machining of the specimens lagged behind schedule, thus causing further delay. Upon delivery of this material it was found that some of the flanges were not machined to within reasonable tolerances and therefore necessitated rejection. Two pilot test columns were tested and testing technique was perfected while conducting these tests. Thus with the experience given in working these pilot tests it will be a relatively simple matter to complete the testing of the remaining columns in this series. As a result of the experience obtained in the testing of the two pilot test columns it seems desirable to slightly alter the method of testing these columns. As proposed originally in the preliminary report, the columns were to be loaded at the kern points as in Fig. 1(a) & (b). It now seems more desirable to load one specimen axially, and the other at the kern with the line of action paralled to the flanges as shown in Fig. 1(c) & (d). The type of loading as first proposed in Fig. 1(b) does not give a critical condition of loading. It seems more desirable to study the behavior of these columns with a centrally applied load.

Ib - Program Ib was an outgrowth of a previous investigation done at Fritz Laboratory dealing with semi-rigid beam-to-column connections[1]. In this investigation load was applied to the connections through cantilever arms but the column was not subjected to an axial load. It was thought advisable to extend this work and investigate the effect these connections would have upon the strength of the column as an axial load was applied. Three specimens as shown in Fig. 2(a), (b), and (c), were ordered from a structural steel fabricator. These specimens were fairly expensive so it was thought it might be wise in the interests of economy to curtail this portion of the program should the results of the first series warrant such action.

TEST PROGRAM

The test program in this work was guided principally by the results of the work of Inge Lyse and Edward Mount[1]. The design details of the three different specimens are shown in Fig. 2(a), (b), and (c). Using one of the most critical cases indicated in this report as a basis, the riveted connection and welded tie plate connection were designed in such a manner as to apply essentially the same load, per inch of connection, to the column flange. By so designing the specimens it was thought they were put on an equal basis since in each case the same line load was applied to the column.

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1. Inge Lyse and E. H. Mount, EFFECT OF RIGID BEAM-COLUMN CONNECTIONS ON COLUMN STRESSES. Research Supplement of American Welding Journal, pp. 25-31, Vol. 17, No. 10, October 1938

TEST PROCEDURE

In the testing of these specimens great pains were exercised in order to eliminate any avoidable errors. Bearing blocks fifteen inches square and five inches thick having a four-inch length of 10 by 10 in. by 140 lb. H-section welded to them were used in an effort to obtain as near a uniform distribution of load as possible. A knife edge of heat-treated alloy steel fifteen inches in length was used to apply the load to the upper bearing block. The lower bearing block was used without a knife-edge for it was feared that a knife-edge under the lower block would result in an unstable and perhaps dangerous set-up. The specimens were set up in the testing machine and a trial direct load applied. Huggenberger tensometers were attached to the flanges midway between the top of the specimen and the beam connection. This position was selected in order to eliminate the interference of local stress concentrations near the ends of the columns, near rivet holes, adjacent to welds, etc. Increments of load within the elastic range were applied and the strains on all four flanges noted. Adjustments of the knife edge with shims were made until the strains in all four flanges were uniformly equal for the load increments. By taking these precautions a very uniform distribution of load was obtained. When the column was thus properly centered the cantilever arms (Fig.3) were attached by means of the splice. Dead weight was then applied to the loading beams in increments up to the design load of the connection. Displacements of the column flange, deflections of the loading beams, rotations of the loading beams, were measured as the bending load was applied. An axial load of fifty thousand pounds had previously been applied to the column to insure against any movement of the whole set-up due to the process of applying the bending load. A diagram was then drawn from the flange displacement data showing the shape of the flange as it had been distorted by the connection. When the dead weight had been removed from the loading pans a Huggenberger tensometer was attached to the upper and lower critical points of each flange as indicated by the shape of the distorted flange as locations of sharpest curvature. Again dead loads were applied in increments and the strains due to the bending of the flange by the connection were observed on the tensometer. With the full dead load on the cantilever arms, increments of axial load were applied to the column. Strains in the flanges were measured with a Whittemore strain gage having a twenty-inch gage length. Gage points were selected so that the gage length covered the portion of the flange most affected by the connection. With each increment of direct load beam deflections, beam rotations, tensometer flange bending strains, longitudinal flange strains and flange displacements were measured. Axial load was applied until failure of the columns occurred. Throughout the test to destruction the full design load of the connection remained on the cantilever loading beam.

TEST RESULTS

The main problem arising in attempting to analyze the results of these tests is the establishment of the load beyond which the structure may be considered as no longer useful. After considerable thought it was decided to use a combination of the load beam deflection curve and the longitudinal strain diagram as the criteria. Using the longitudinal strain diagram (Fig.4) it may be seen that a general yielding of the material has taken place at about 395,000 lb. for the welded angle and 435,000 lb. for the welded plate. Locating these points on the beam deflection curve (Fig.5) it is seen that the corresponding points fall in the immediate vicinity of the line corresponding to a 0.250 deflection. In the case of the riveted connection longitudinal strains were not taken but it is believed that in view of supplementary data presented herein that the load as indicated by this 1/4-in. deflection limit is a reasonable value for the load at general yield. Although a quarter of an inch deflection as a criteria of yielding appears on the surface to be of considerable magnitude, it should be remembered that this deflection is measured at the end of a twelve-foot cantilever and is therefore sensitive to very small rotations at the connection. The deflection as measured by the gage at the end of the cantilever may be resolved into three component parts: deflection due to rotation of the semi-rigid joint under moment alone, deflection as a result of the elastic bending of the cantilever assuming a fixed end, and deflection due to additional joint rotation caused by flange displacement as a result of increased direct load. Considering the case of the riveted connection, for example, it is found by calculation that beam deflection due to joint rotation, when the dead weight is applied, amounts to 1-1/8 in. while the elastic bending of the cantilever itself contributes an additional 5/16 in. deflection. Thus it is seen by comparison with the magnitude of these quantities that the selected limiting value of deflection as a result of direct load is in reality a relatively small quantity.

Fig. 6, 7, and 8, illustrate the type of failure produced by axial load in the column. It will be noted that the failure is very similar in the cases of the welded angle and riveted angle connections. In the case of the riveted angle the flange buckled the greatest amount slightly below the rivets (Fig.6a) while the welded angle caused the flange to buckle the greatest just below the top weld (Fig.7a). In Fig. 8(a) and (b) it may be seen that the buckled wave occurs above the top of the beam, the tie plate connection apparently having no effect upon the failure of the member as a short column. As evidence of the correct centering of the load the uniform strain pattern as indicated by the scaling or flaking of the thin whitewash coating will be noticed.

Table 1 (Fig.9) is a tabulation of the results of tension tests to determine the physical properties of the column material used for this series. It is apparent that the steel is of satisfactory quality as determined by the A.S.T.M. specifications for structural steel for buildings.

A summary of test results are presented in Table 2 (Fig.10). The first column of this table lists the types of connections tested in this investigation. For details of these connections and the specimen in general see Fig. 2(a), (b), and (c). In designing welded connections it is customary to refer to the working load of the joint in terms of pounds per inch of the connecting material[2]. In this investigation the connections were subjected to the design bending load for the connection as listed in the second column of Table 2. Column three of this table presents the general yield of the column as a whole as obtained from Fig. 4. The relation between longitudinal deformation of the column in a length of twenty inches taken across the connection and direct load is shown in Fig. 4. The method used in obtaining the yield point is one which gives some weight to the ultimate strength of the structure and which has been found to give good results in other tests of structural steel members. A straight line tangent to the initial, elastic, load-deformation curve is extended to the intersection with a horizontal

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2. Inge Lyse and G. J. Gibson, WELDED BEAM-COLUMN CONNECTIONS American Welding Society Journal Supplement, Vol. 15, No. 10, October 1936

line tangent with the ultimate load. The actual load at the deformation given by this intersection is taken as the general yield of the member. The yield points as listed in this column constitute the more important results of this work.

The criteria for usefulness of a column insofar as local failure at one end is concerned should be the longitudinal deformation. When the permanent longitudinal deformation has become excessive the life of the column as a useful structure is ended. The fifth column of this table of summarized results lists the experimental factors of safety of the columns. This factor of safety is the ratio between 17,000 lb per sq in. as the maximum allowable stress in short columns and the yield point of the column as a whole as determined from longitudinal deformation. Column five of the table presents the yield points as derived from Fig. 5. This is more a criteria of local lateral yielding of the column flanges at the point of connection than of the column as a whole. The reasoning used in arriving at these figures has been previously described. Column six of this table lists the observed ultimate loads carried by the three specimens. The final column of Table 2 is a result of a simple computation. Since it is generally thought that a short column should develop the yield strength of the material these figures are presented for comparison. The loads listed in the final column represent the load the specimen would have carried had it developed the full yield strength of the column material.

II - Part II of the investigation as set forth in the preliminary report of last fall concerned itself with columns having end moments. As originally proposed a number of columns having various slenderness ratios were to be tested with a variety of ratios between axial and bending loads. In view of the expense anticipated as a result of experience in purchasing previous specimens it is now proposed to conduct an extensive investigation using small sections. A quantity of three-inch I-beams having properties very closely approaching those of H-sections has been purchased for this purpose. A new, low capacity, precision hydraulic testing machine will be used for this purpose so that the tests will be of an accurate character. By using small, easily handled sections, a higher degree of precision is possible than in the case of tests using heavy full sixe sections requiring a heavy, cumbersome set-up and its consequent compromises. When this series of tests is complete a small number of tests involving full size sections will be made to demonstrate the similitude existing between the tests on large and small sections. The following program is submitted for the consideration of the committee.

Slenderness Ratio	Ratio of Eccentricity to X-X Kern Distance								Ratio of Eccentricity to Y-Y Kern Distance								
	Series -1		Series -2						Series -1		Series -3						
20	0		1						0		1						
30	0		1						0		1						
40	0	½	1	1½	2	3	5	7	0	½	1	1½	2	3	5	7	Series-4
50	0		1						0		1						
60	0		1						0		1						
80	0	½	1	1½	2	3	5	7	0	½	1	1½	2	3	5	7	Series-5
100	0		1						0		1						
120	0		1						0		1						

In considering the problem of columns with end moments there are two independent variables, namely the slenderness ratio and the eccentricity. This program in Series 1, 2, and 3 maintains a constant eccentricity and varies the slenderness ratio. In Series 4 and 5 the slenderness ratio is retained constant while the eccentricity is varied. This program calls for a total of forty-eight tests on the small sections. Should time permit, a series of tests involving oblique loading thus

24
32
44

putting the column into double curvature will be made. The large ratios of eccentricity to the kern distance were decided upon as a result of the trend to design for continuity. It has been found that semi-rigid connections in a building frame put moments into the columns equivalent to very large eccentricities.

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TABLE -1

PHYSICAL PROPERTIES				
Coupon Number	Yield Point Lbs. per Sq. In.	Ult. Strength Lbs. per Sq. In.	Young's Modulus Lbs. per Sq. In.	% Elongation in 2 Inches
1-F	35,480	62,140	30,000,000	46.5
2-F	38,220	63,670	30,000,000	40
1-W	40,570	62,010	30,520,000	36
A1-F	35,400	61,000	———	51.8
A2-F	39,400	60,800	29,300,000	39
A3-F	35,900	61,400	———	44.5
A4-F	35,600	61,500	29,000,000	44.5
Arith. Mean	37,200	61,800	29,760,000	43.2

TABLE-2

Type of Connection	Test Load of Connection Lbs. per Inch	Yield Point of Column by Longitudinal Deformation	Factor of Safety $\frac{Y.P.}{17,000}$	Local Yield Point of Column by Beam Deflection	Ultimate Load Sustained by Column	Yield Point Strength of Column
Riveted Angle	3,248	—	—	303,000#	486,600#	539,000#
		—		20,900#/□"		
Welded Angle	3,300	395,000#	1.6	366,000#	501,000#	539,000#
		27,250#/□"		25,250#/□"		
Welded Plate	3,300	435,000#	1.8	448,000#	511,000#	539,000#
		30,500#/□"		30,950#/□"		

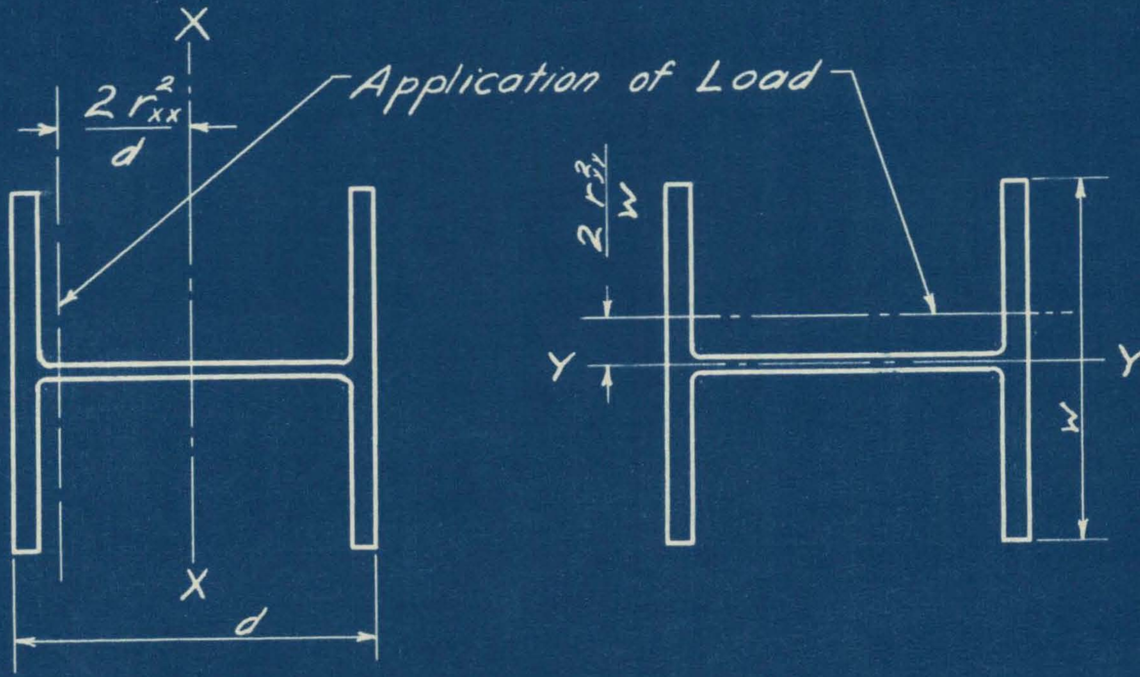


Fig.-1(a)

Fig.-1(b)

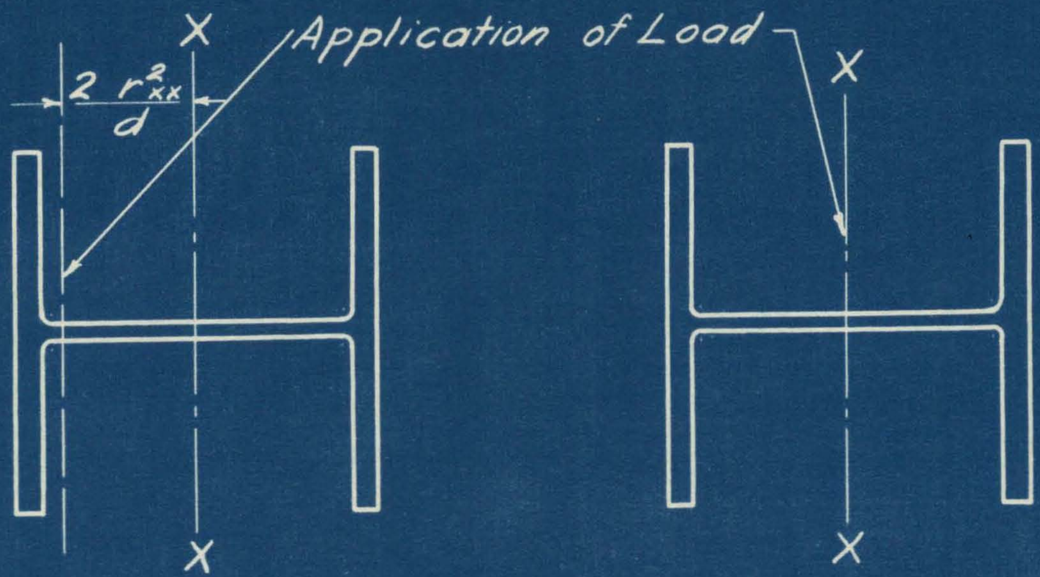
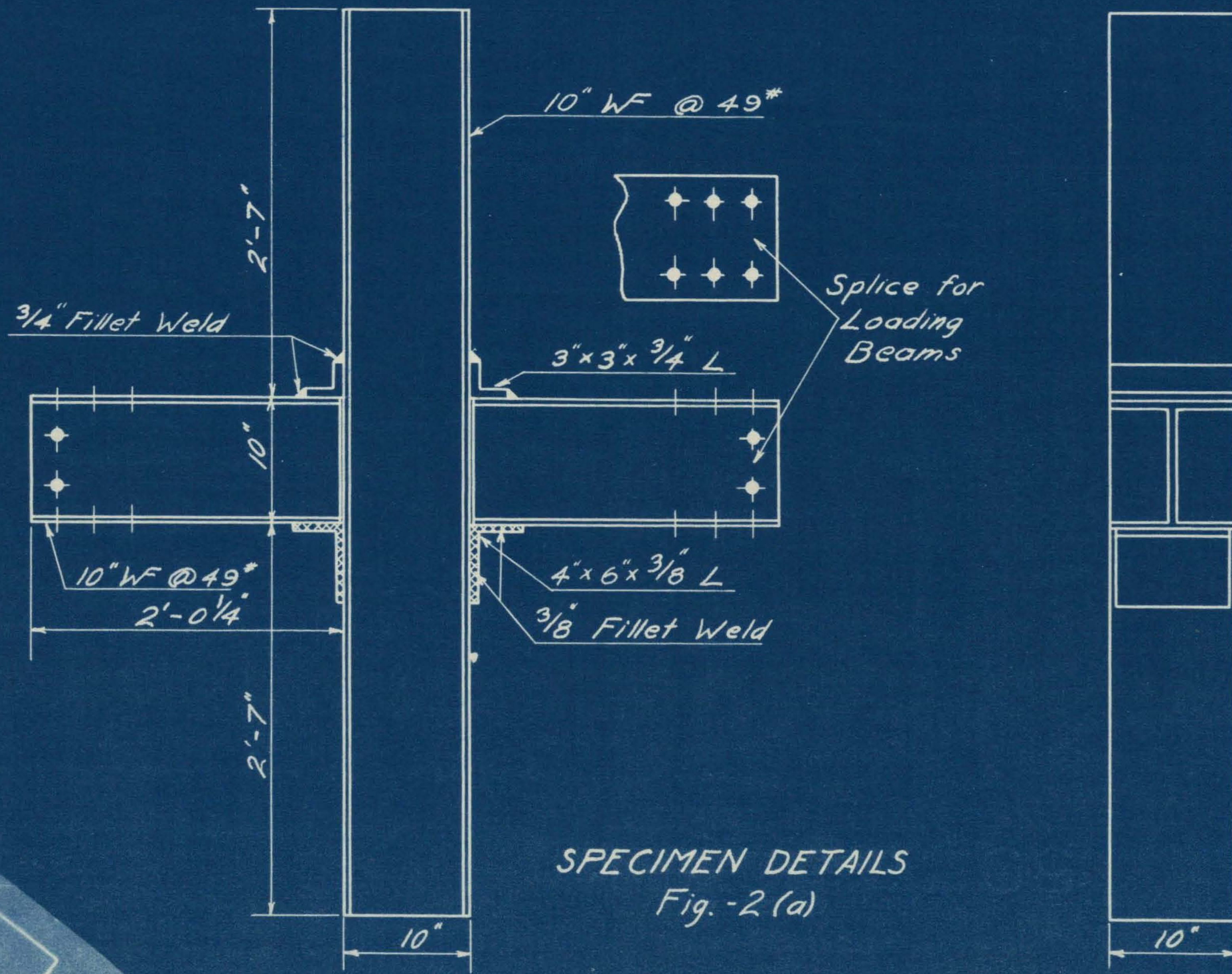
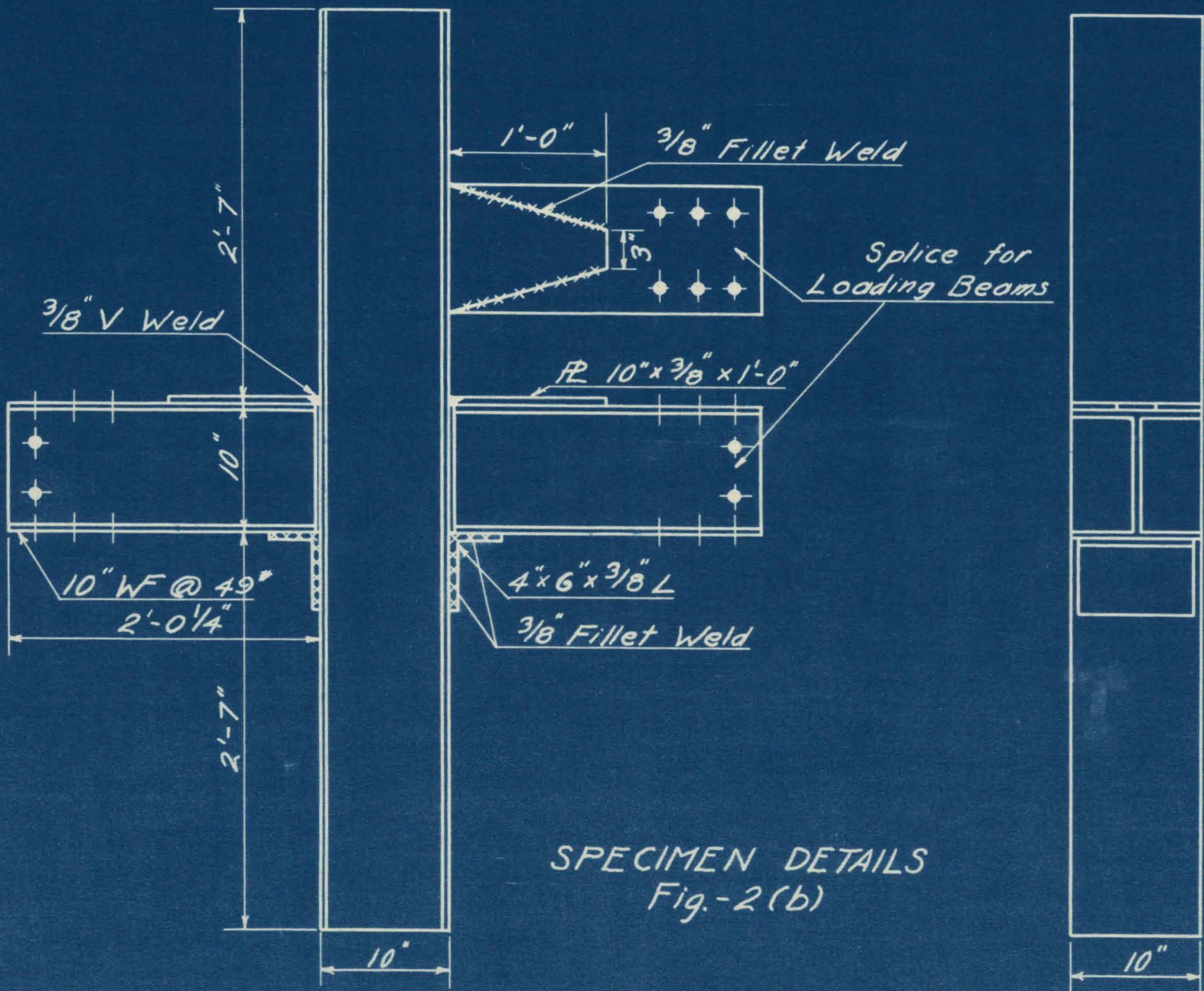


Fig.-1(c)

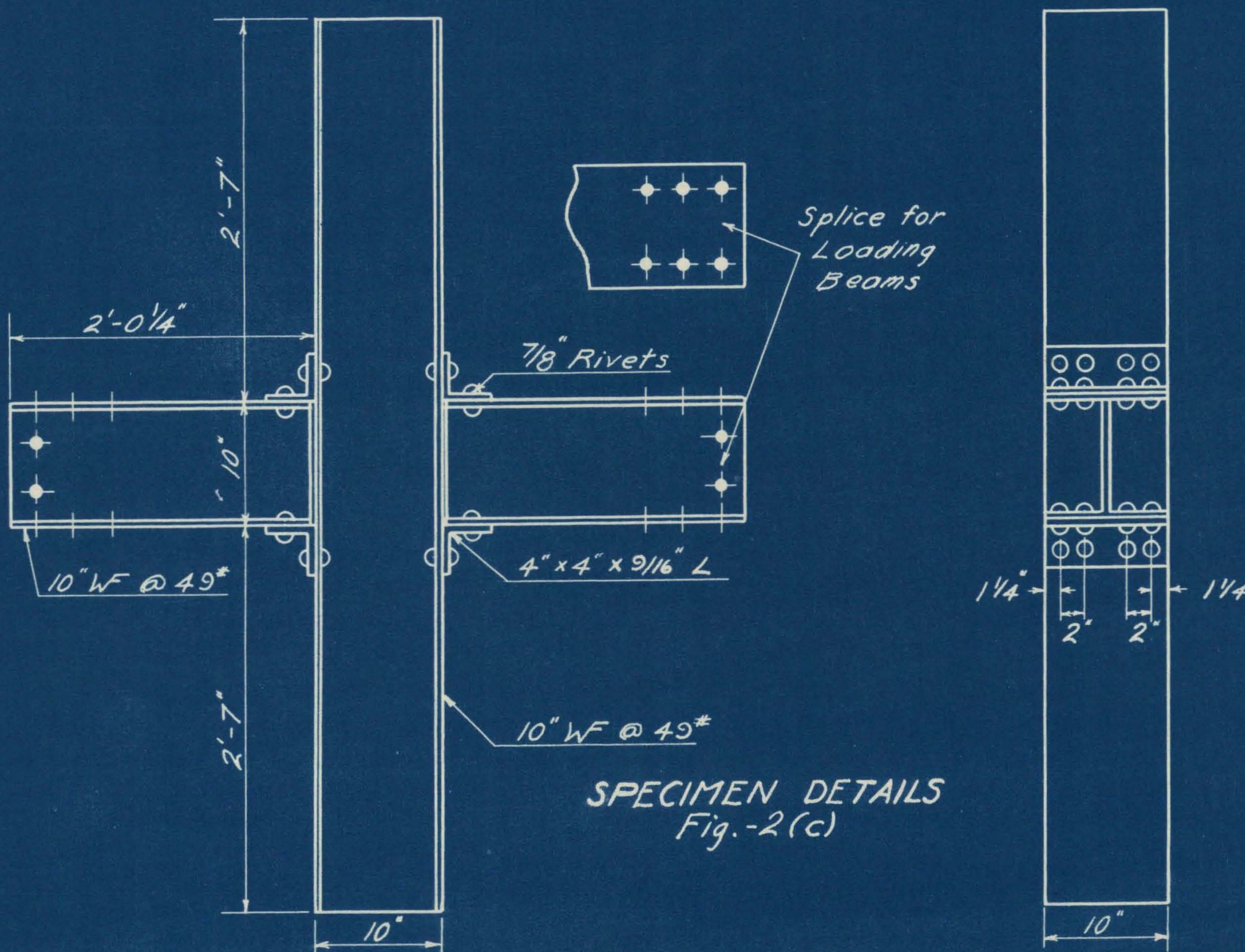
Fig.-1(d)



SPECIMEN DETAILS
Fig. -2 (a)



SPECIMEN DETAILS
Fig.-2(b)



SPECIMEN DETAILS
Fig.-2(c)

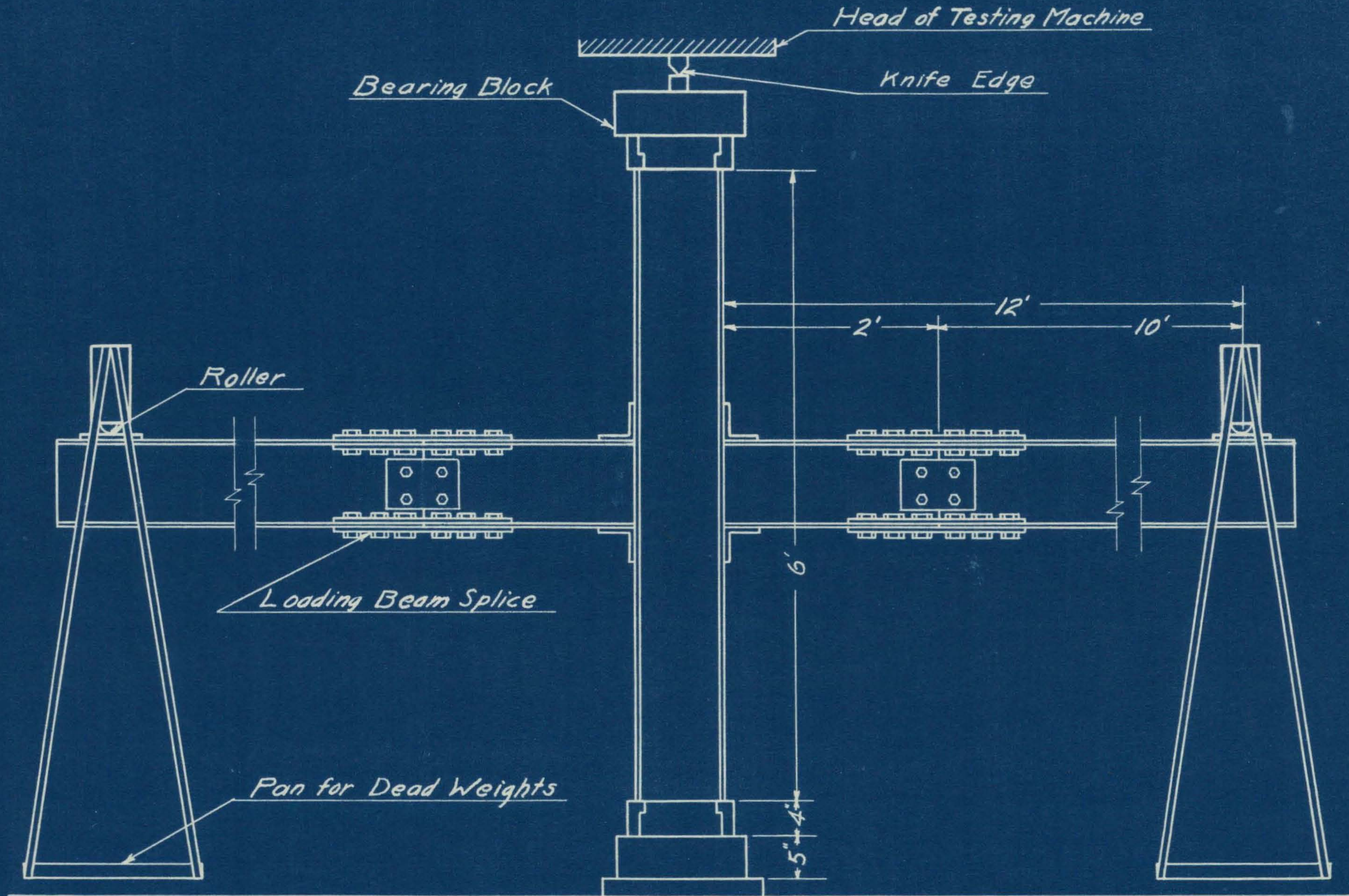


Fig. -3 TESTING ARRANGEMENT

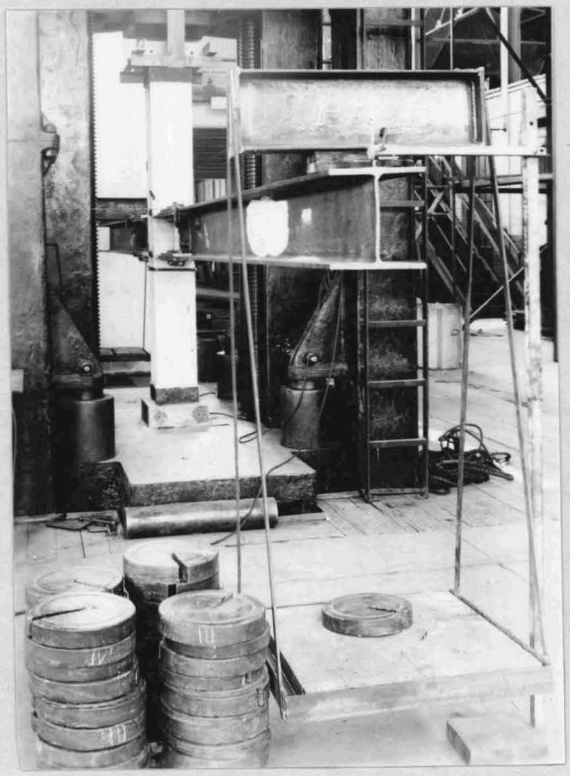


Fig. 3(a)

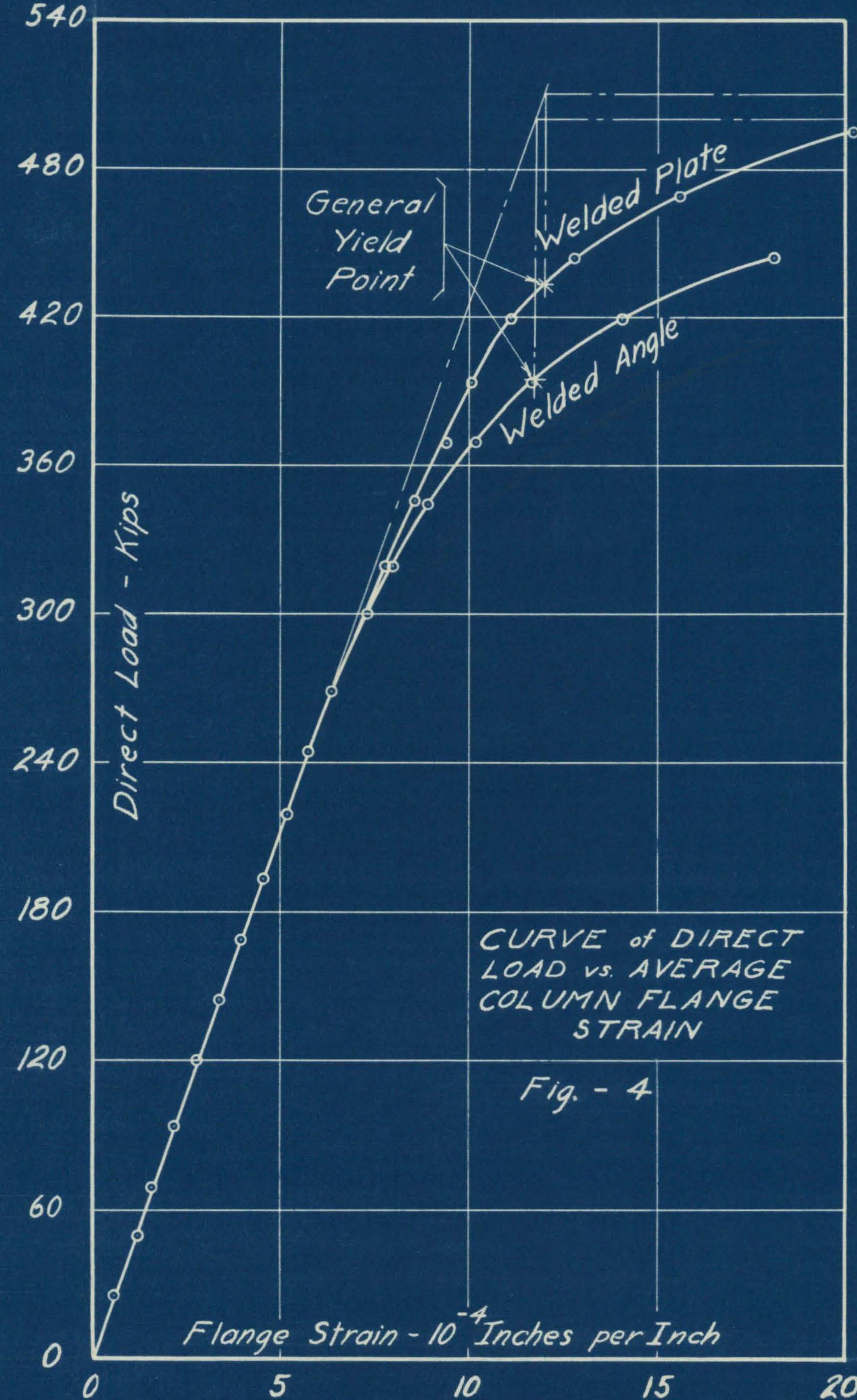
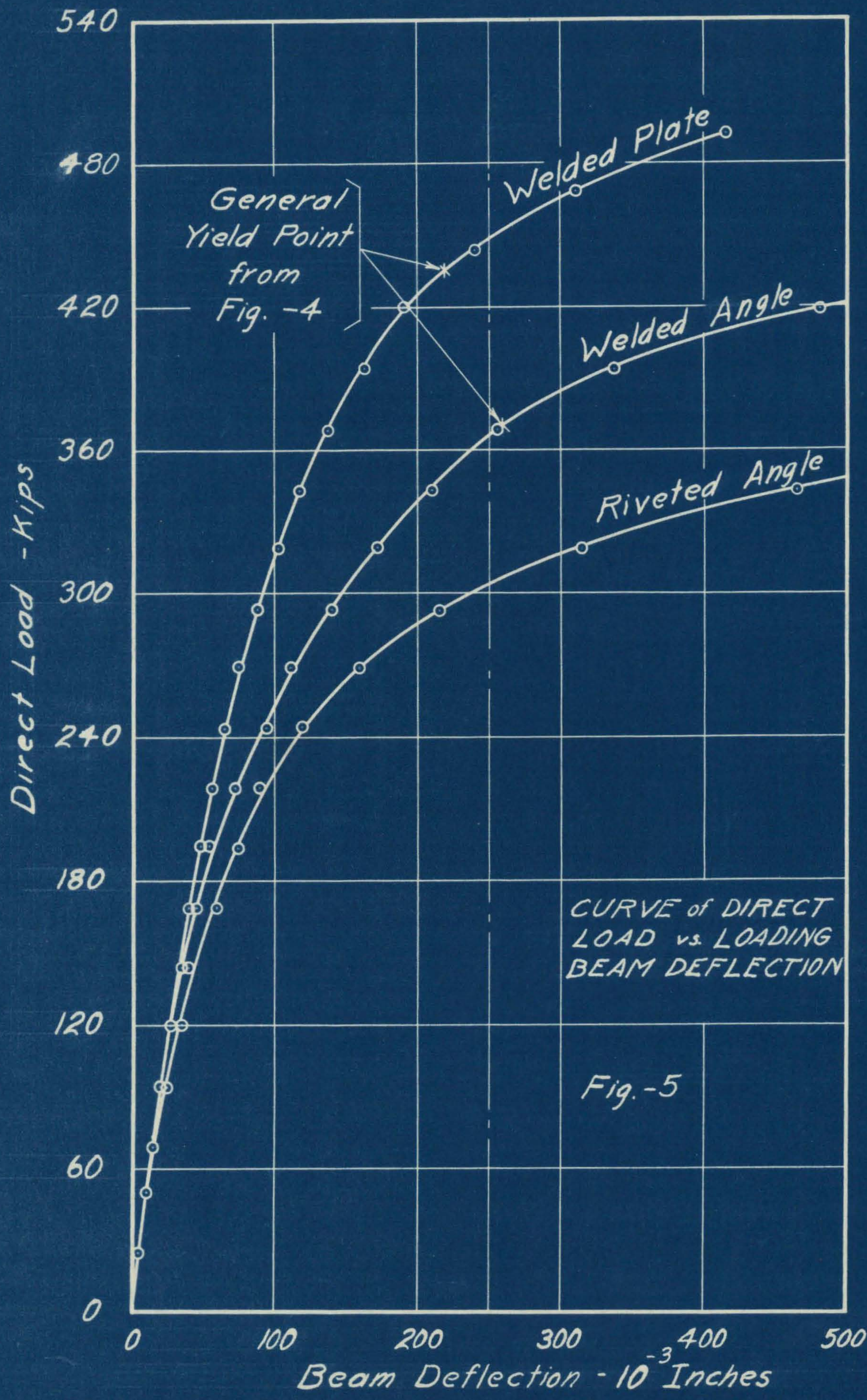

540
480
420
360
300
240
180
120
60
0
0
5
10
15
20
Direct Load - Kips
Flange Strain - 10^{-4} Inches per Inch
General Yield Point
Welded Plate
Welded Angle
CURVE of DIRECT LOAD vs. AVERAGE COLUMN FLANGE STRAIN

Fig. - 4



540
480
420
360
300
240
180
120
60
0
0
100
200
300
400
500
Direct Load - Kips
Beam Deflection - 10^{-3} Inches
Welded Plate
Welded Angle
Riveted Angle
General Yield Point from Fig. -4
CURVE of DIRECT LOAD vs. LOADING BEAM DEFLECTION
Fig. -5

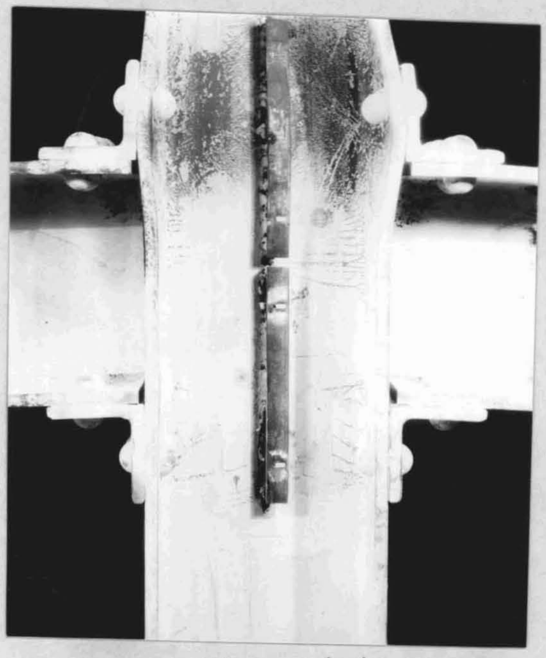


Fig. 6(a)

Fig. 6(b)

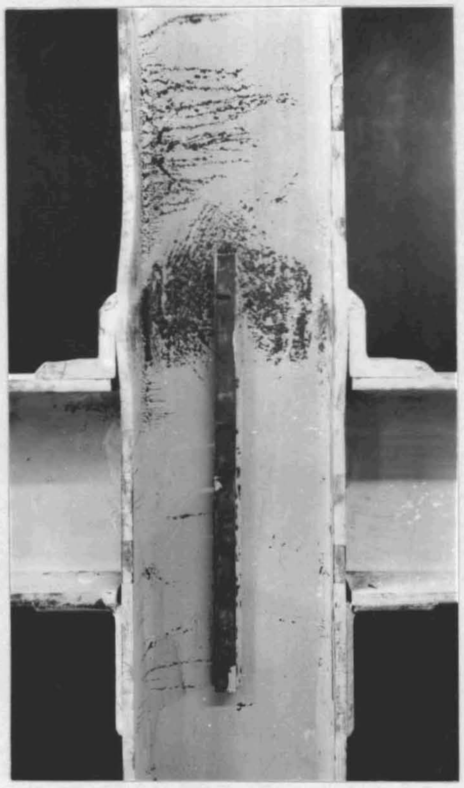


Fig. 7(a)

Fig. 7(b)

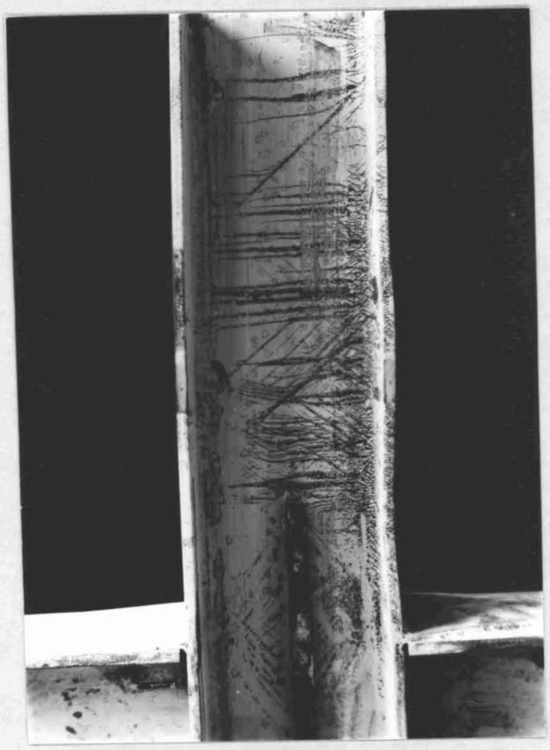


Fig. 8(a)

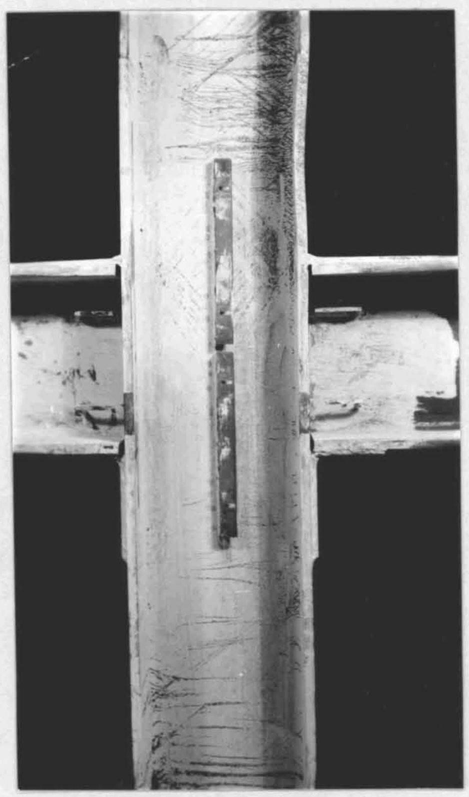


Fig. 8(b)