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TENSILE CAPACITY OF PARTIAL PENETRATION WELDS

by
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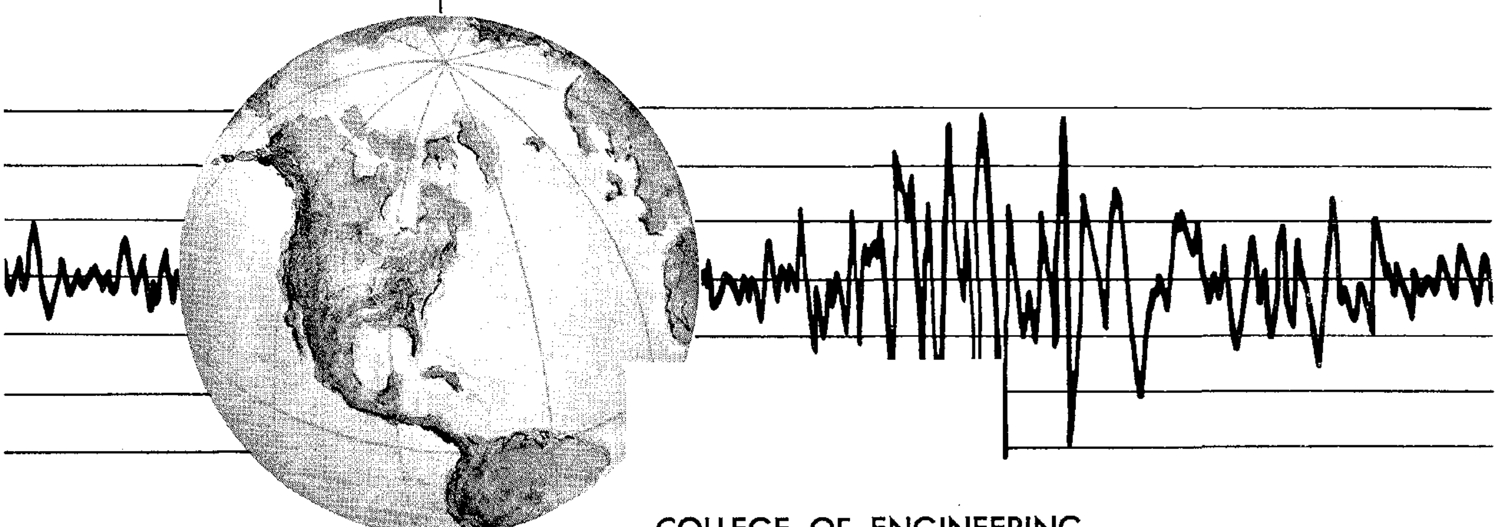
Report to Sponsors:
American Iron and Steel Institute and
National Science Foundation



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The flanges of six W 14 x 320 sections with milled ends were joined together at mid-height with different size partial penetration welds to form short columns; one additional specimen had full penetration welds through both flanges. Four of the specimens were failed directly in tension; three were subjected to a few compression-tension cycles until failure. The experiments were designed to simulate conditions which can develop in some columns of steel frames during severe earthquakes. The spliced joints developed their specified strengths with the smallest weld attaining highest stress. Failure for all specimens with partial penetration welds occurred through the splice exhibiting very limited ductility. The data generaged by these experiments should be useful for earthquake design as well as provide basic information on the behavior of partial penetration welds of unprecedented size.

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ABSTRACT

The flanges of six W 14 x 320 sections with milled ends were joined together at mid-height with different size partial penetration welds to form short columns; one additional specimen had full penetration welds through both flanges. Four of the specimens were failed directly in tension; three were subjected to a few compression-tension cycles until failure. The experiments were designed to simulate conditions which can develop in some columns of steel frames during severe earthquakes. The spliced joints developed their specified strengths with the smallest weld attaining highest stress. Failure for all specimens with partial penetration welds occurred through the splice exhibiting very limited ductility. The data generated by these experiments should be useful for earthquake design as well as provide basic information on the behavior of partial penetration welds of unprecedented size.

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I. INTRODUCTION

In multi-story steel structures that undergo lateral displacements due to seismic or wind loads, the large column-type members may experience tensile forces. These column members are generally spliced at discrete locations throughout their length. Also, in steel frames employing bracing a number of columns may go into tension during a seismic disturbance. Partial penetration welds are sometimes employed in joining such members and the capacity of these welds for large members when they resist tensile forces has not been clearly established. In addition, the data on the behavior of partial penetration welds under a few severe reverse loading cycles such as may occur during an earthquake is not available. Therefore, a limited investigation has been undertaken to provide some data on partial penetration welds in heavy column section.

II. GENERAL DESCRIPTION

The tests were conducted using W 14 x 320 shapes of ASTM A 572-70A Grade 50 material due to their availability and common use in buildings. Seven specimens were fabricated from this material in the general configuration as shown in Fig. 1. The specimens had an overall length of 6 ft. 6 in. and were spliced at mid-height with four different weld sizes.

The weld sizes used in the splices varied from the minimum allowed by the current AISC Specification [1] for the thickness of plates being joined to the full penetration weld. For the partial penetration welds the prepared bevel groove was 1/8 inch larger than the nominal weld size. The details of these welds are shown in Fig. 2. No welding was done on the inside face of the flanges or on the web of the splice. In making the welds the AWS specifications [2] were adhered to. The welds were made using NR311 inner-shield welding. In all cases the welds were identical on both flanges. The welds were visually inspected at the first root pass. All of the splice welds were ultrasonic tested. The average measured weld sizes, determined after the tension tests were completed are recorded in Table 1.

In order to accommodate the fixtures to be used in the tension tests, the flanges near each end of the specimen were reduced to a seven inch width. Each specimen was fitted with plates to distribute the tension loads through an eight inch pin.

One of the wide flange sections had an existing cutout in the web. This section was to have been used with the 3/8 in. or minimum weld, however, in fabrication this section was used for the 3/4 in. weld specimen. The web cutout was located under the end plates above the pin hole.

It was anticipated that the heavy and full penetration welds would fail at tension loads which exceeded the capacity of the 3000 kip testing machine. Therefore, the flanges of these specimens were reduced to the sizes as noted in Table 1.

In the case of the specimens that were tested in both compression and tension, the ends of the column sections were milled 90° to the center line.

A typical fully fabricated specimen is shown in Fig. 3.

III. TESTING PROCEDURE

Testing Equipment

The column tension and compression tests were performed in a 4,000,000 lb. Southwark-Emery Universal Testing machine.

Coupon Tests

Four standard tension test coupons were fabricated in accordance with ASTM A370 [3] from the excess flange material removed in connection with the full penetration weld. Two of the specimens were taken from the flange so that the weld was in the center of the specimen and the other two were taken some distance from the weld location.

Tension Tests

The first four column specimens were tested in tension to failure. This included each of the weld sizes tested. The pin ended fittings allowed the rotation of the longitudinal axis as the axial loads were applied.

The only specimen instrumented in this test sequence was the full penetration weld. Clip gages were placed on the face of each flange across the weld. The gage length of the clip gages was 4.25 inches.

Compression-Tension Tests

The remaining three specimens, which included the minimum, intermediate and heavy weld sizes, were subjected to a limited number of cyclic loadings. The magnitude of the loads applied in compression and tension varied from the full 3,000,000 lb. compression on each specimen to the approximate yield load in tension. This yield load was determined from the average yield stress obtained from the coupon tests and the nominal weld area. Table 2 shows the cyclic load levels for each specimen. The typical cyclic load sequence is illustrated in Fig. 4.

Clip gages were located across the welds on both flanges for each of the load applications. Again the gage length was 4.25 inches.

For the compression loadings the columns were placed such that the longitudinal axis of the specimen was aligned with the axis of the testing machine. The ends of the columns were allowed to bear directly against the top head and the bottom platten of the machine. The top swivel head of the testing machine was locked in a horizontal alignment to prevent its rotation. Again for the tension tests the pin ended fittings allowed the rotation of the longitudinal axis as the axial loads were applied. On the final tension loading the specimen was loaded to failure.

A typical tension specimen mounted in the testing machine is shown in Fig. 5.

IV. TEST RESULTS

The average yield strength of the coupons without the weld was 57.4 ksi and the average with the weld was 56.2 ksi. The tensile strength was found to be 81.7 ksi and 82.3 ksi, respectively. The average modulus of elasticity for all four specimens was 30.4×10^3 ksi. The average elongation for all four specimens in a 2 inch gage length was 44 percent. The mill test report indicated a yield point of 64.0 ksi and a tensile strength of 91.5 ksi with the elongation of 22 percent in an 8 inch gage length.

A summary of the principal test results is given in Table 1. In testing specimen 2, the pin eye failed due to the presence of the cutout in the web as was mentioned previously. The ultimate tensile load shown in Table 1 for specimen 2 is the maximum load reached prior to the pin eye failure. In an attempt to retest this specimen approximately one inch was cut from each flange and the end with the failed pin eye was directly gripped in the testing machine. This, however, restrained the rotation of the longitudinal axis and contributed to premature failure of the connection. The area of the weld after the flanges had been reduced was 23.41 square inches and the ultimate load obtained was 1370 kip.

The load displacement curve across the weld for specimen 4, the full penetration weld, is given in Fig. 6. The load-displacement hysteretic curves across the weld for specimens 5, 6 and 7 are given in Figs. 7, 8 and 9, respectively.

In each case, except as noted above for specimen 2, the failure in tension was through the weld. In the case of the full penetration weld, the failure was through the weld on one flange and through the flange material about 2 inches above the weld on the other flange as can be noted in Fig. 10. Figure 11 shows the failure of the nominal 3/8 inch weld specimen which was typical of the failures for the other specimens.

Discussion of Results

The ultimate tensile strength of the columns varied considerably for the specimens tested when in general all of the failures were in the weld. However, they were still within the range of the requirements of AWS for filler metal which has a range of from 70 to 95 ksi [2]. As noted in

Table 1 the larger the weld the lower was the tensile strength. The ultimate tensile strength is based on the actual weld area and not the nominal weld size. Figure 12 exhibits these ultimate weld strengths for the different weld penetrations.

In reviewing the hysteretic curves for specimens 5, 6 and 7, the compression loadings were all very linear. In tension the specimens exhibited some non-linearity during the first two tensile cycles. The minimum or 3/8 inch weld specimen shows the largest elongation prior to failure which is some five times larger than the other two specimens. This ultimate elongation is a measure of the tensile ductility of the welds. The full penetration weld also exhibited as noted in Fig. 6 this large ductility in the same order of magnitude as the 3/8 inch weld. The failure of each of the seven specimens was very abrupt and sudden and not what is generally expected in mild steel structure members. This is also pointed out by the fact that the elongation at failure only varied from 0.2 to 1 percent.

V. CONCLUSIONS

The results of these tests indicate that from the strength point of view welded splices in large columns sections perform very satisfactorily even with the minimum weld. The fact that the splice has undergone severe cyclic loading seems to have little or no effect on the ultimate tensile strength. However, the designer must recognize that partial penetration welded connections exhibit very little ductility. To attain the specified weld size it is recommended that the welds be inspected visually at first root pass regardless of subsequent inspections.

VI. ACKNOWLEDGEMENTS

This work was conducted as part of the AISC Project 193 on Earthquake Bracing of Multistory Steel Frames with some assistance from a sub-project on Braced Frames, NSF Grant ENV76-04263. The authors are grateful for the support given.

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2. "Structural Welding Code", American Welding Society, 1976.
3. "American Society for Testing and Materials", Part 4, ASTM Philadelphia, April 1970.

TABLE 1. SUMMARY OF TEST RESULTS

SPECIMEN No.	NOMINAL WELD SIZE	BEVEL GROOVE SIZE S (IN)	AVERAGE FLANGE WIDTH	MEASURED AVERAGE WELD SIZE	AVERAGE WELD AREA (IN^2)	ULTIMATE TENSILE LOAD (KIP)	ULTIMATE TENSILE STRENGTH (KSI)
1	3/8	1/2	16.81	0.48	16.14	1593	98.70
2	3/4	7/8	16.75	0.799	26.77	2405	89.94
3	1	1 1/8	6.78	1.024	13.885	1045	75.05
4	Full Penetration	Full Penetration	7.06	2.02	28.52	2010	70.48
5	3/8	1/2	16.875	0.48	16.2	1575	97.22
6	3/4	7/8	16.875	0.830	28.01	2080	74.26
7	1	1 1/8	12.07	1.039	25.08	1810	72.17

TABLE 2. CYCLIC LOADING

SPECIMEN NO.	NOMINAL WELD SIZE (IN)	AVERAGE WELD AREA (IN^2)	MAXIMUM COMPRESSIVE LOAD (KIP)	CYCLIC TENSILE LOAD (KIP)
5	3/8	16.2	3000	930
6	3/4	28.01	3000	1530
7	1	25.08	3000	1400

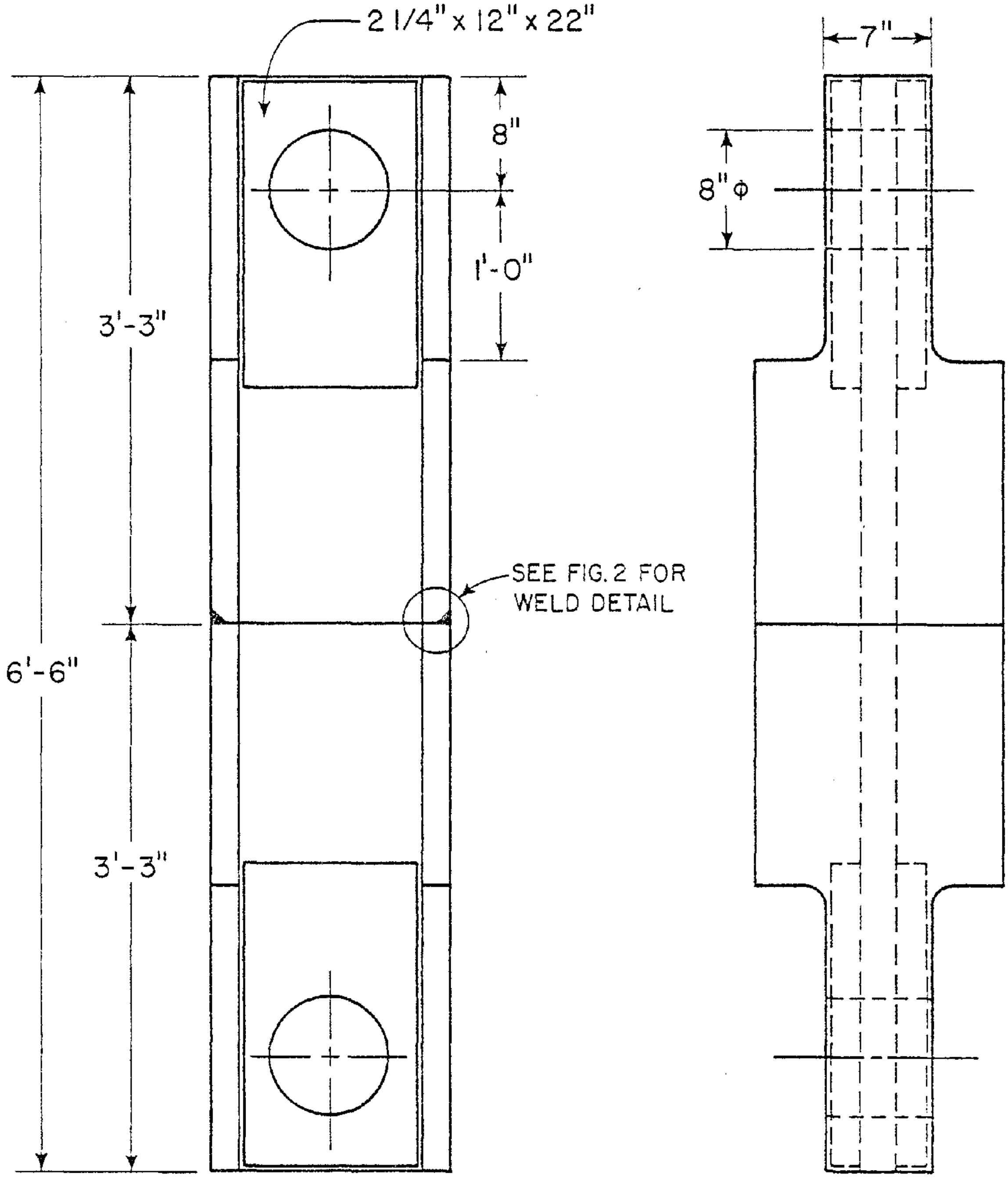

FIG. 1 TYPICAL COMPRESSION-TENSION SPECIMEN

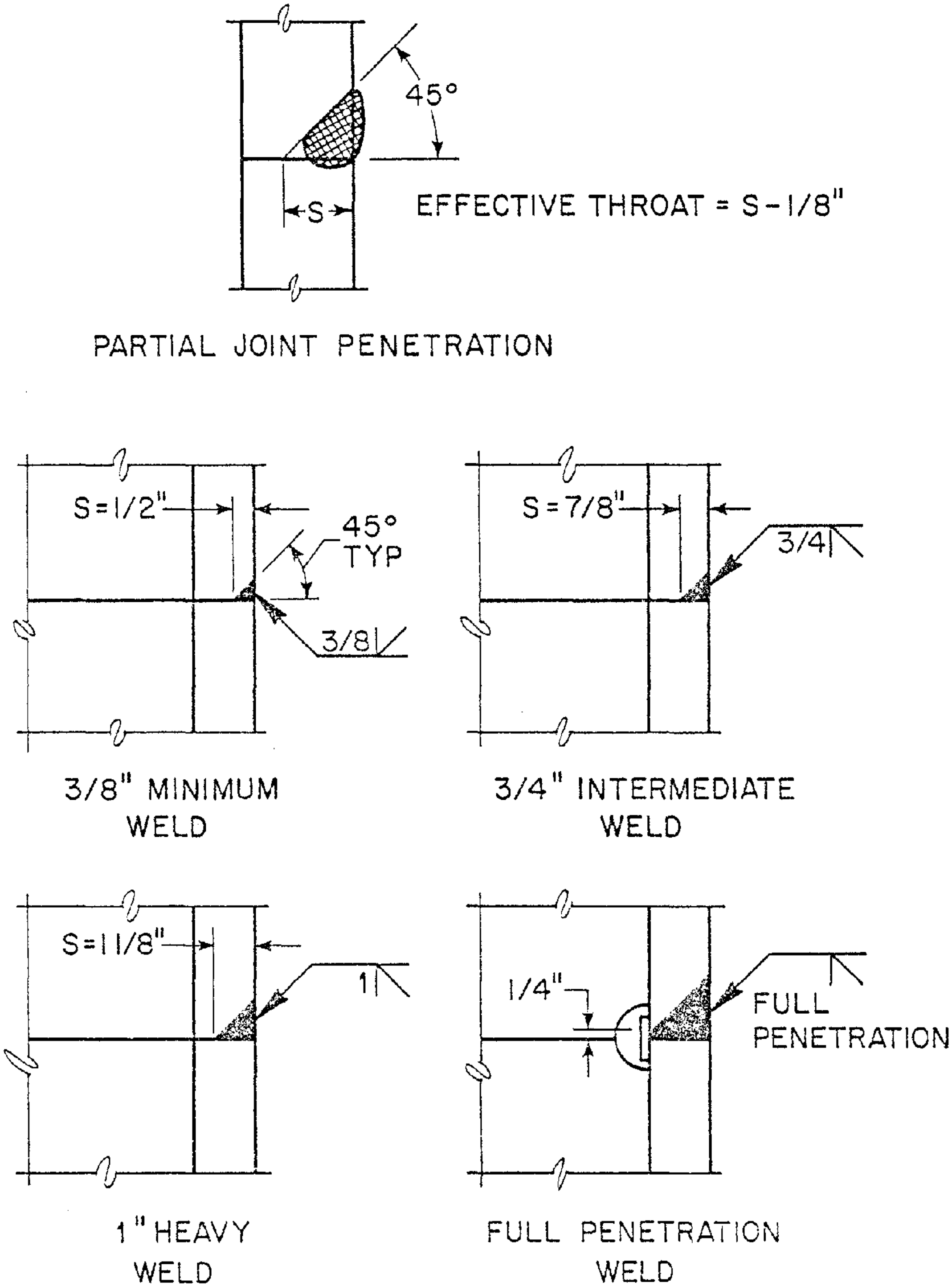

FIG. 2 TYPICAL SPLICE WELD DETAILS

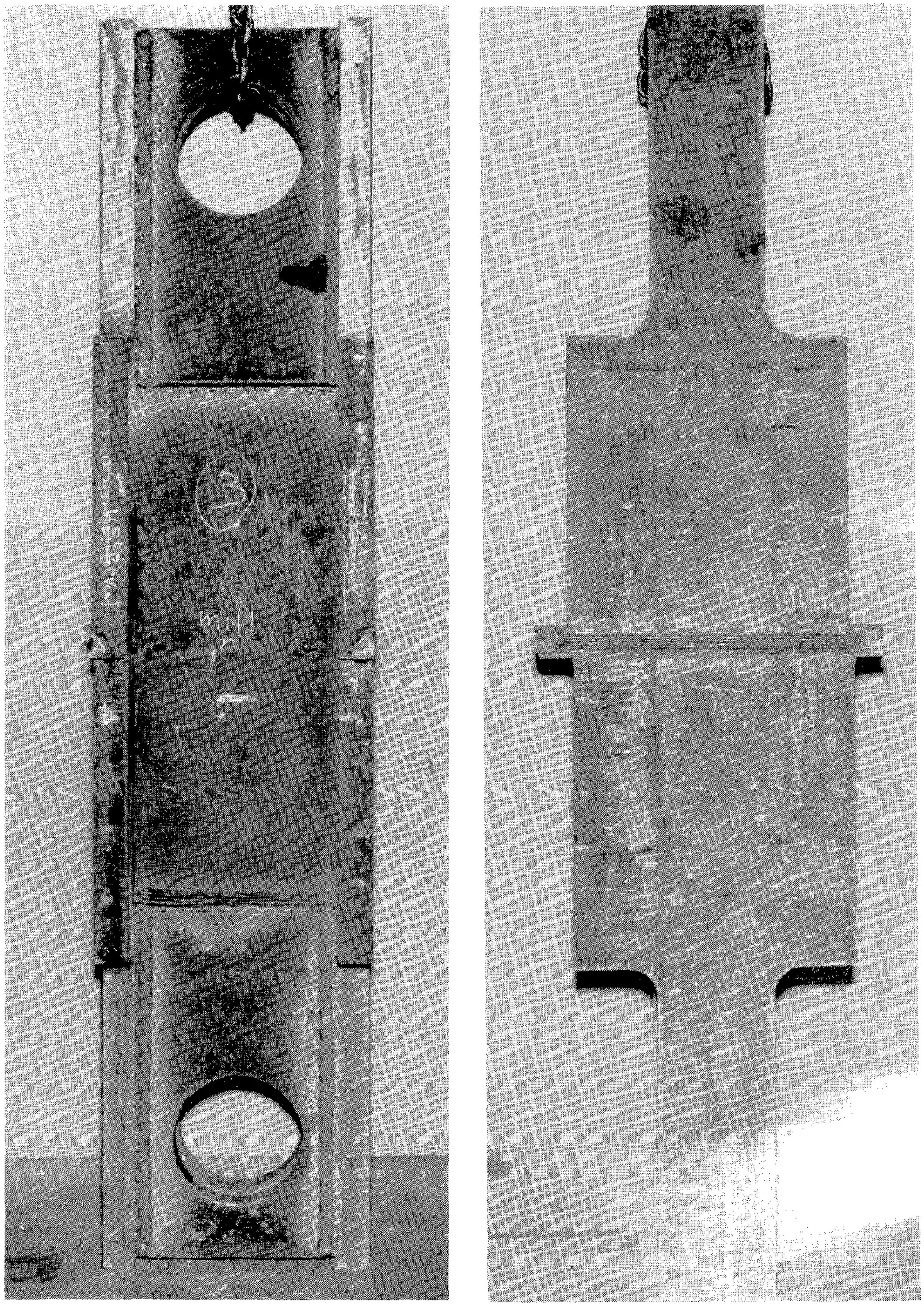

FIG. 3 TYPICAL FABRICATED SPECIMEN.

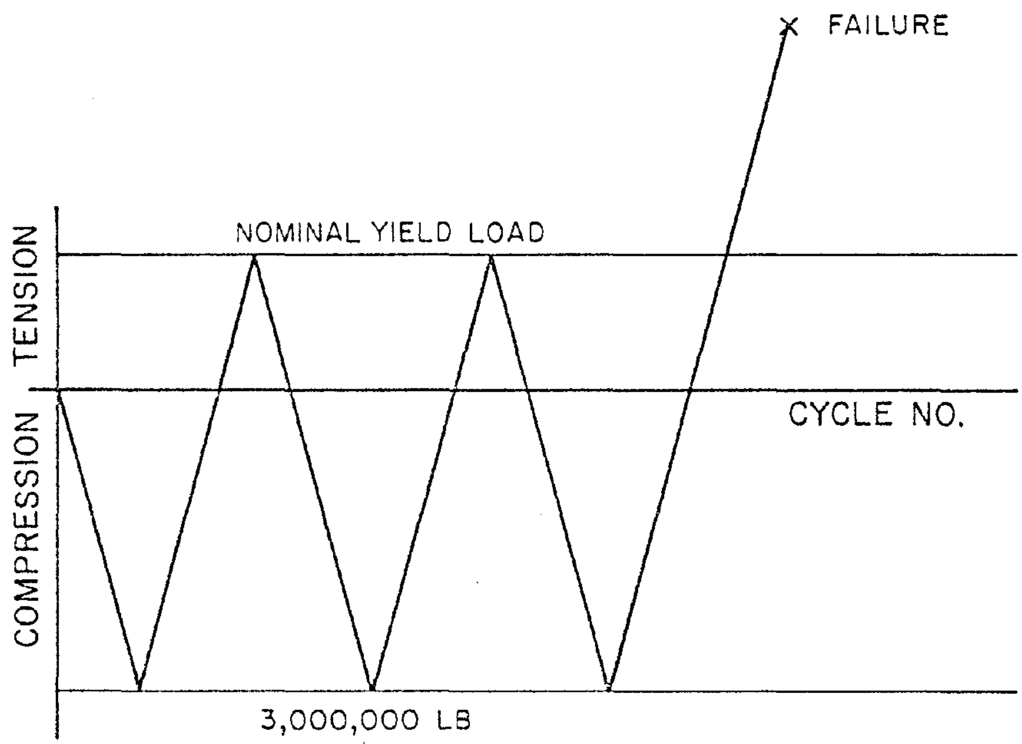

FIG. 4 TYPICAL CYCLIC LOADING SEQUENCE

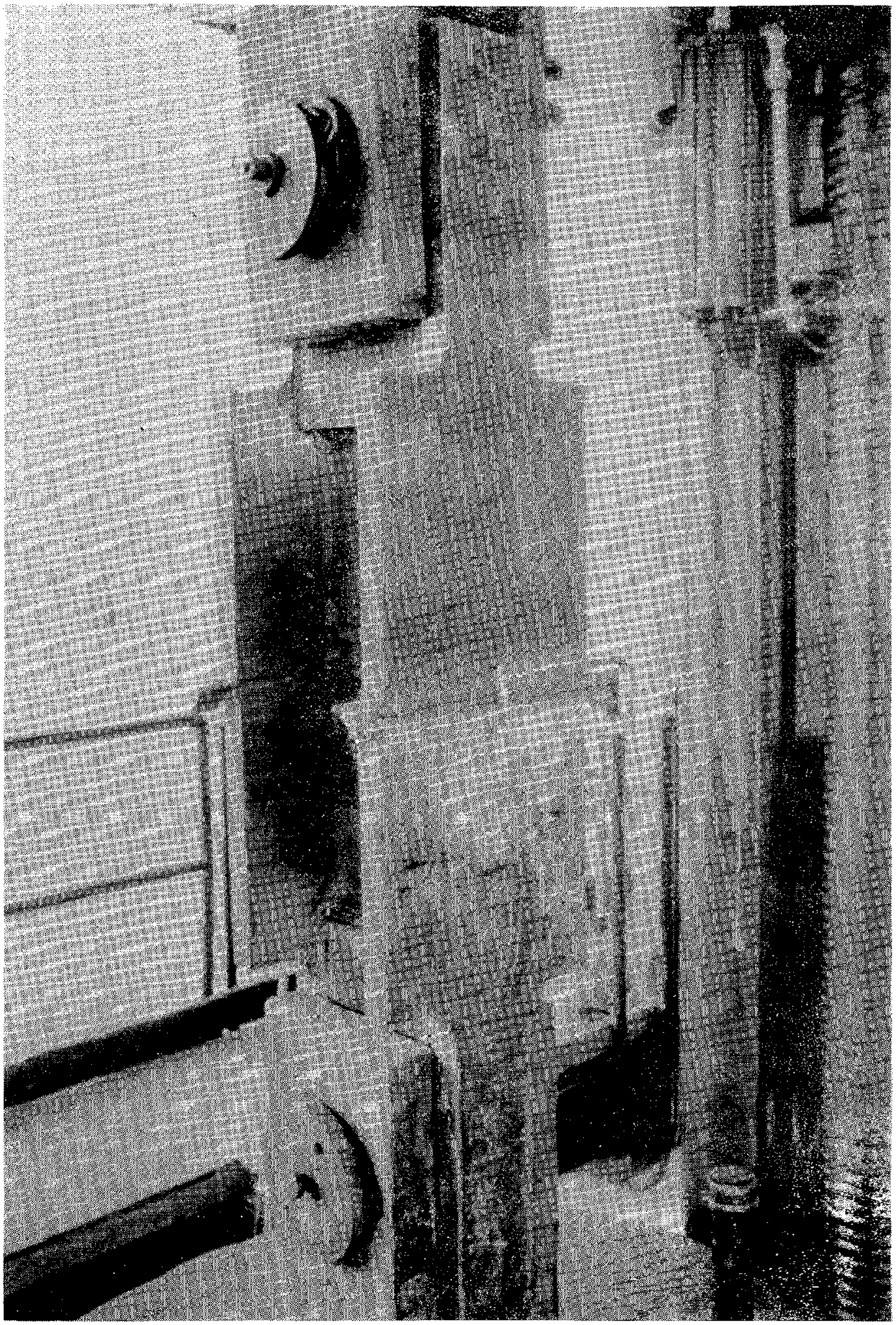

FIG. 5 TYPICAL TENSION TEST CONFIGURATION.

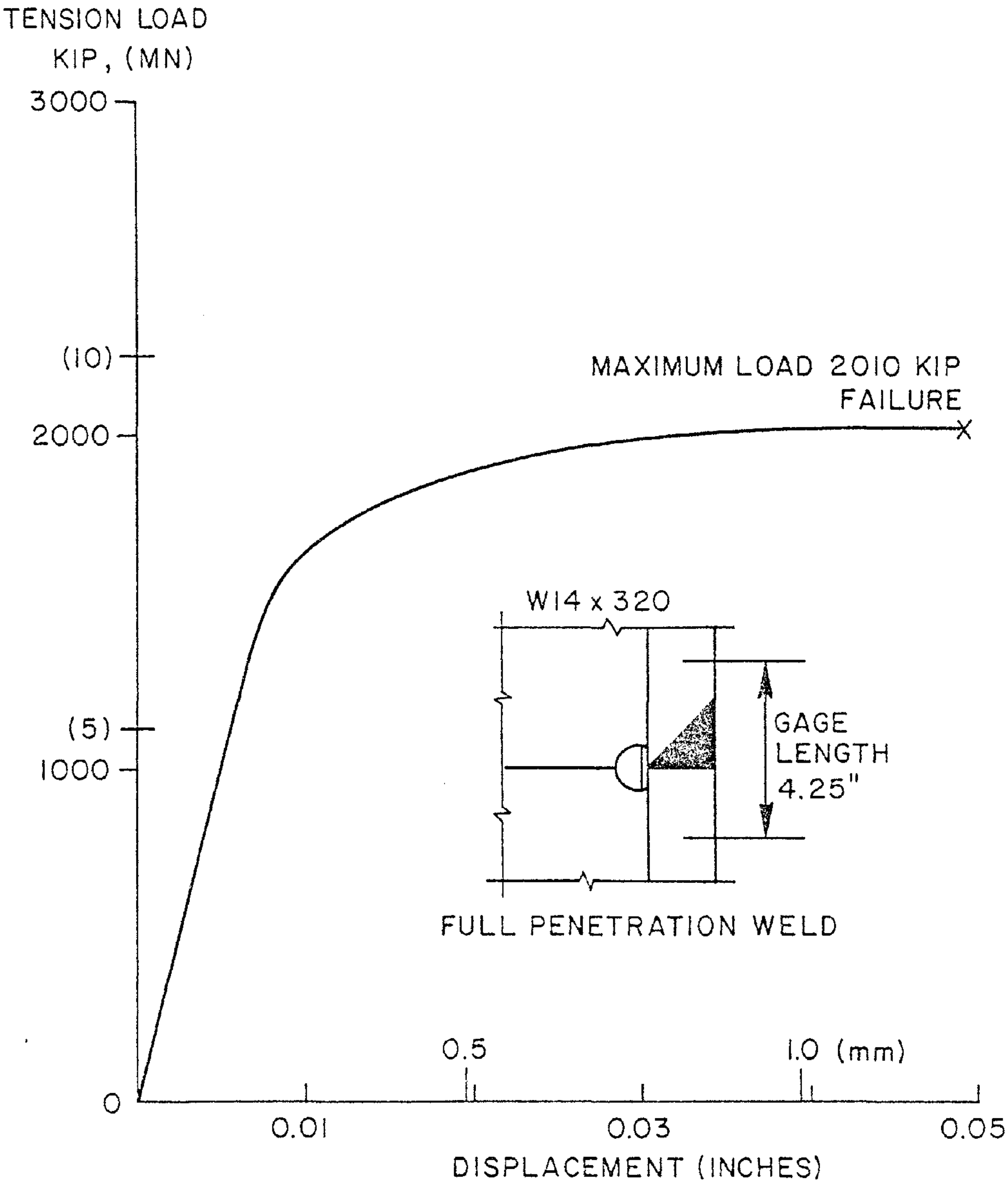

FIG. 6 LOAD-DISPLACEMENT CURVE ACROSS WELD SPECIMEN NO. 4.

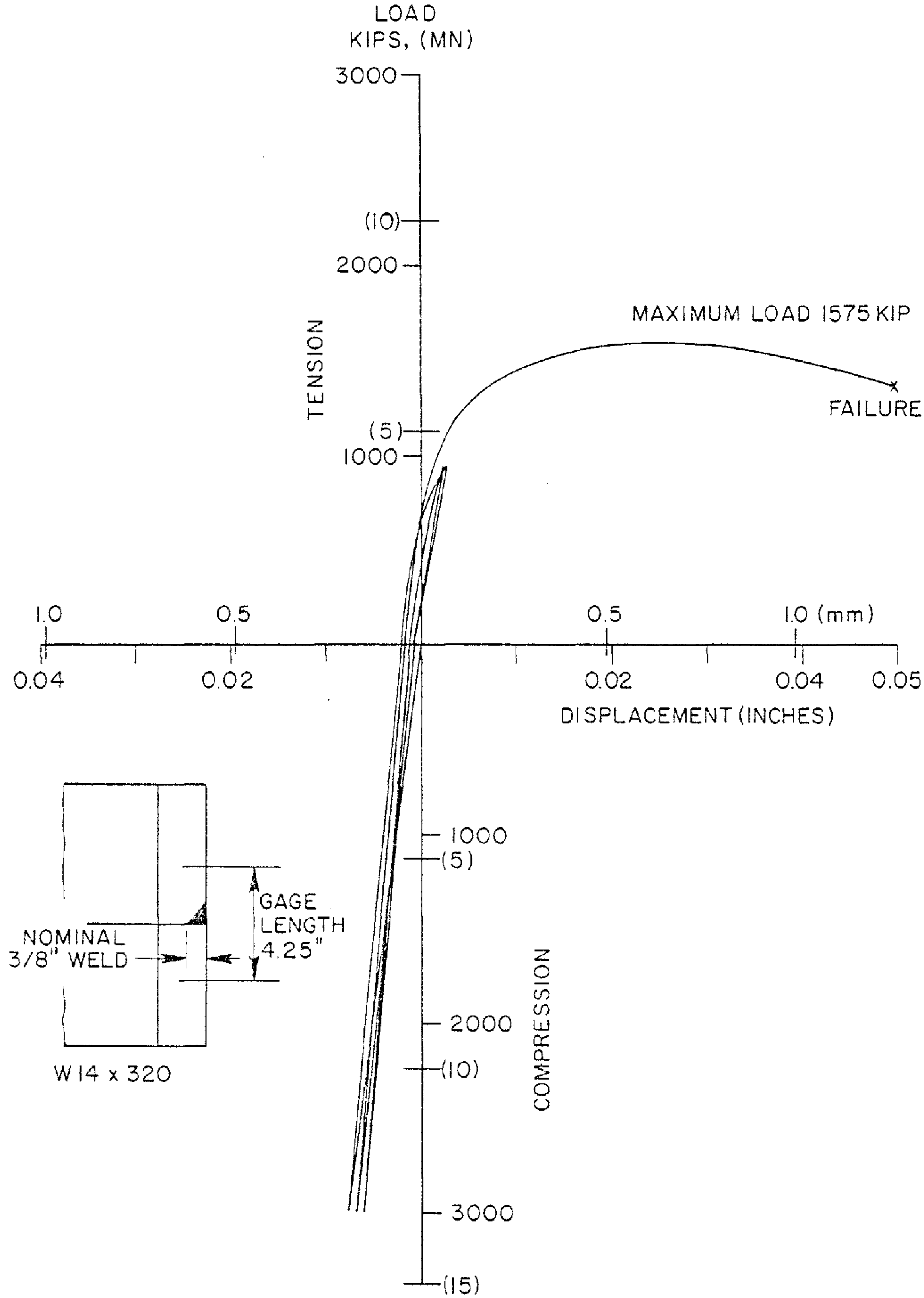

FIG. 7 LOAD-DISPLACEMENT HYSTERESIS ACROSS WELD SPECIMEN NO. 5.

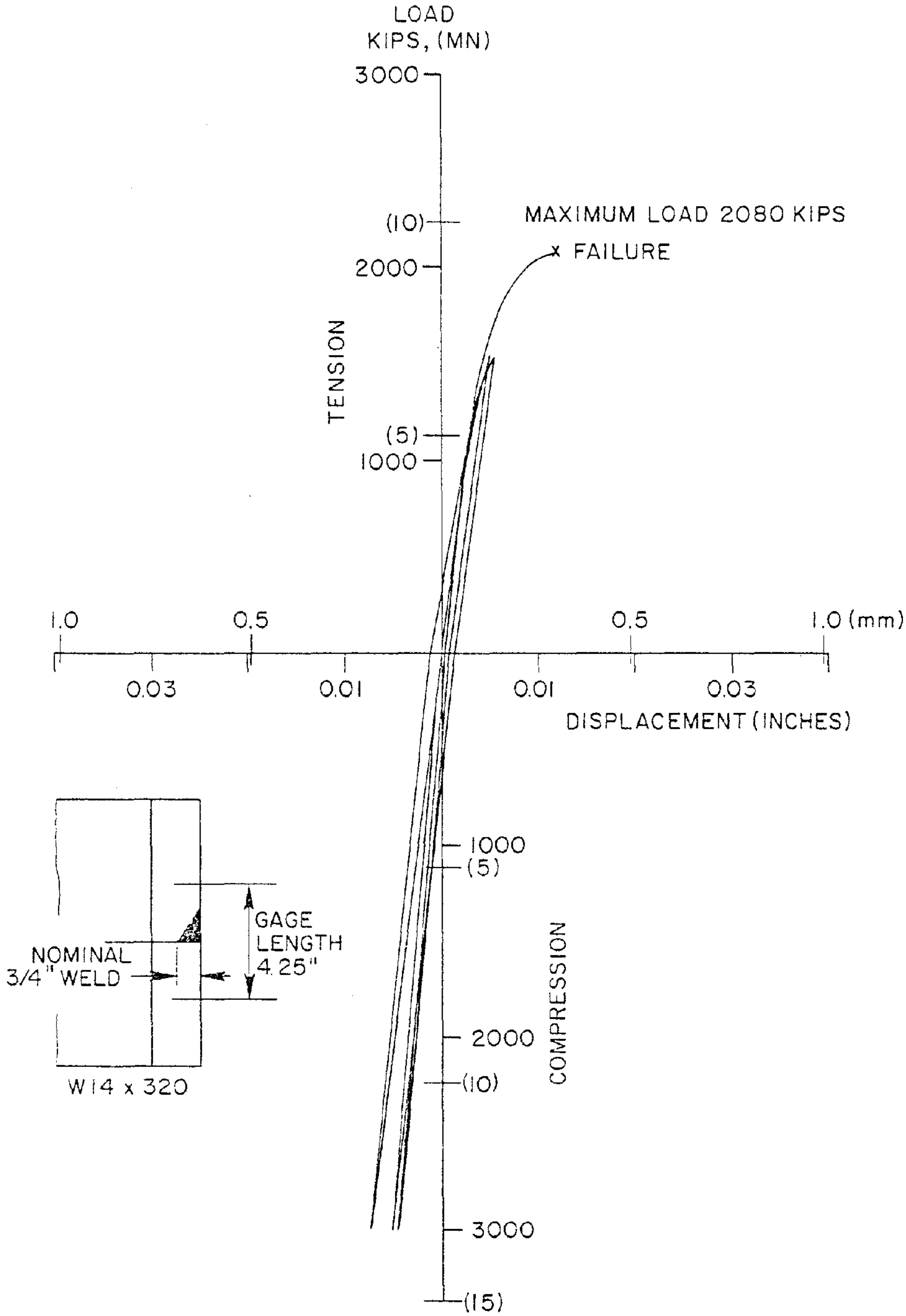

FIG. 8 LOAD-DISPLACEMENT HYSTERESIS ACROSS WELD SPECIMEN NO. 6.

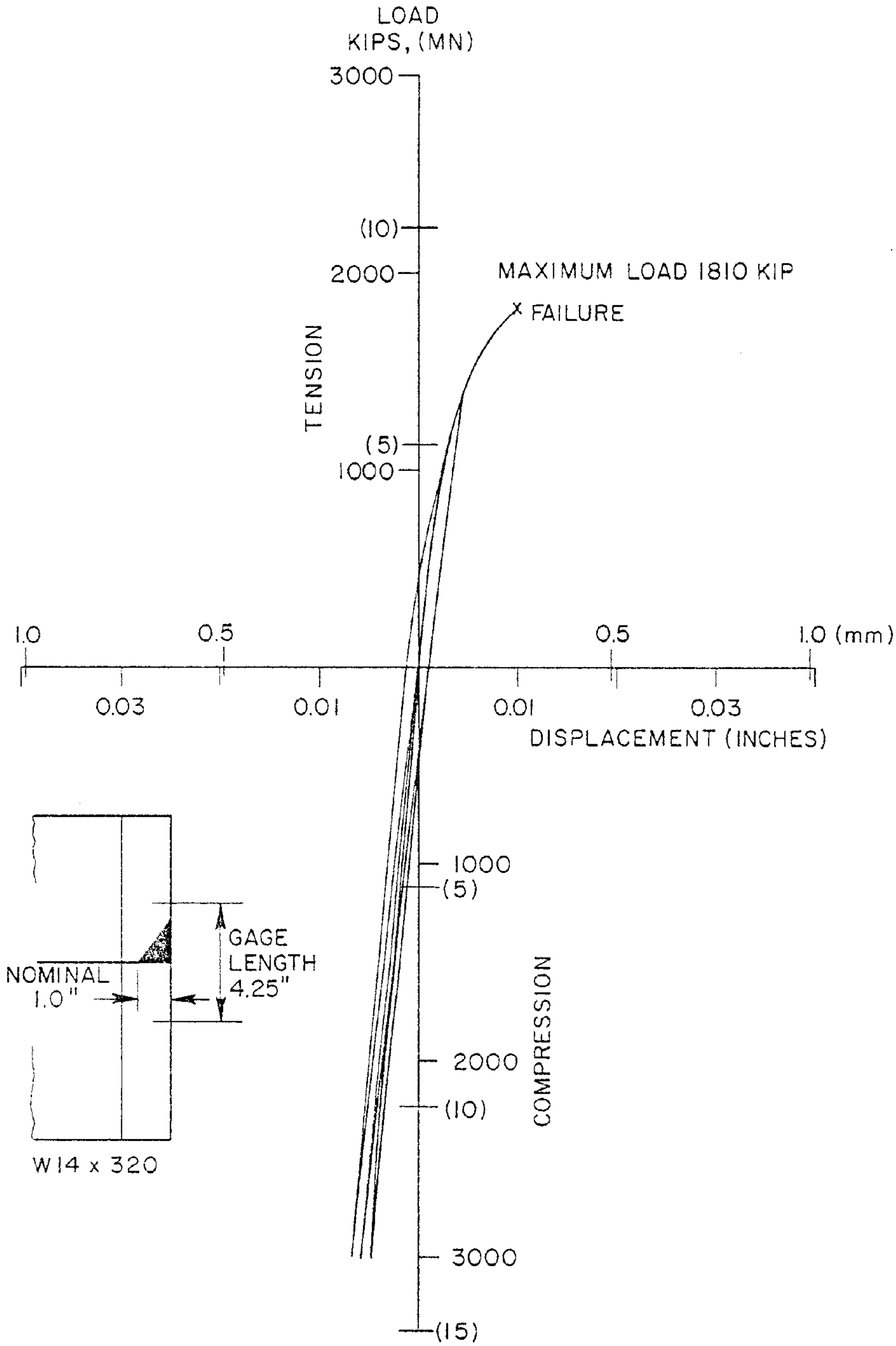

FIG. 9 LOAD-DISPLACEMENT HYSTERESIS ACROSS WELD SPECIMEN NO. 7.

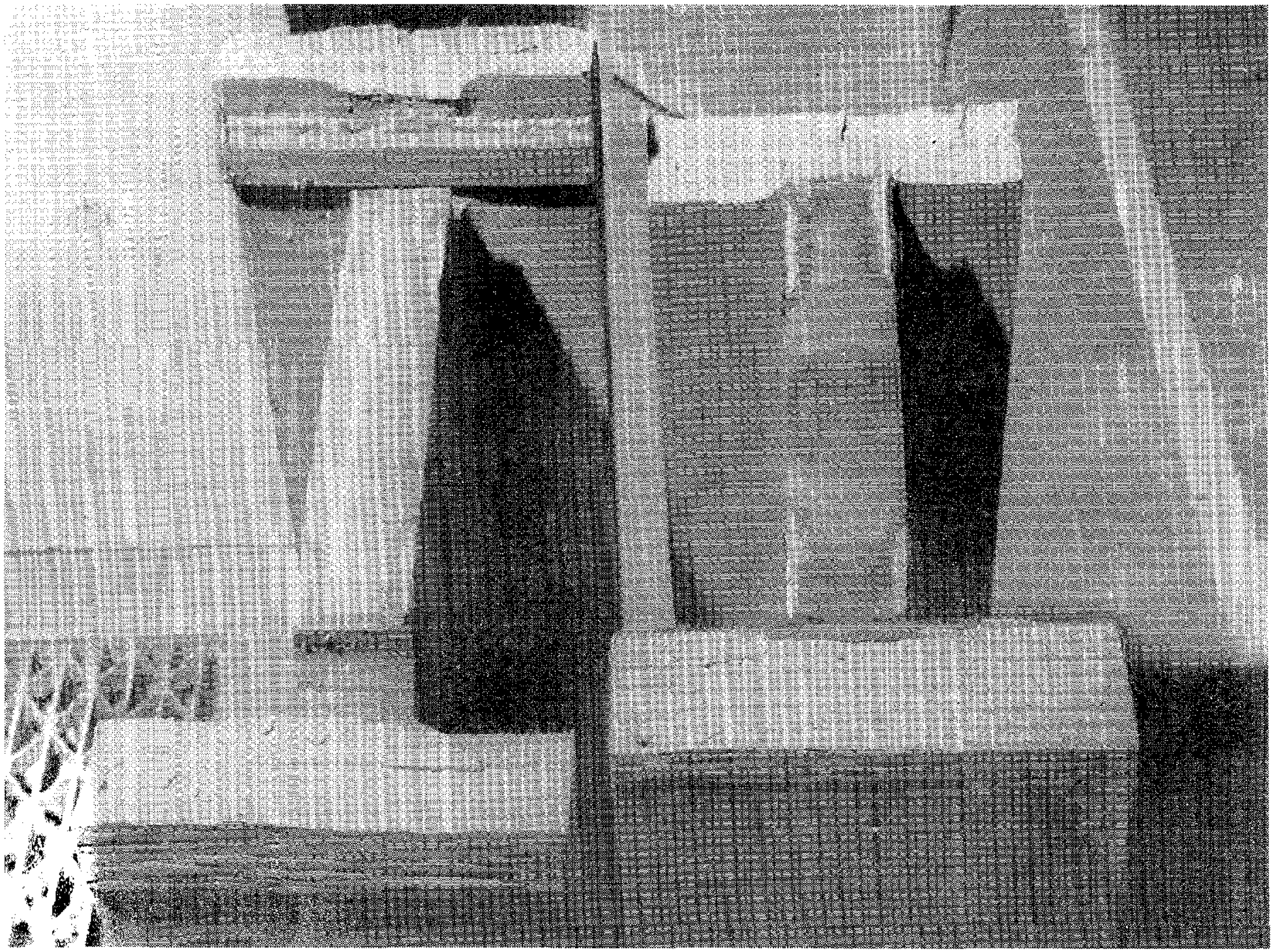

FIG. 10 FAILURE OF FULL PENETRATION WELD SPECIMEN NO. 4.

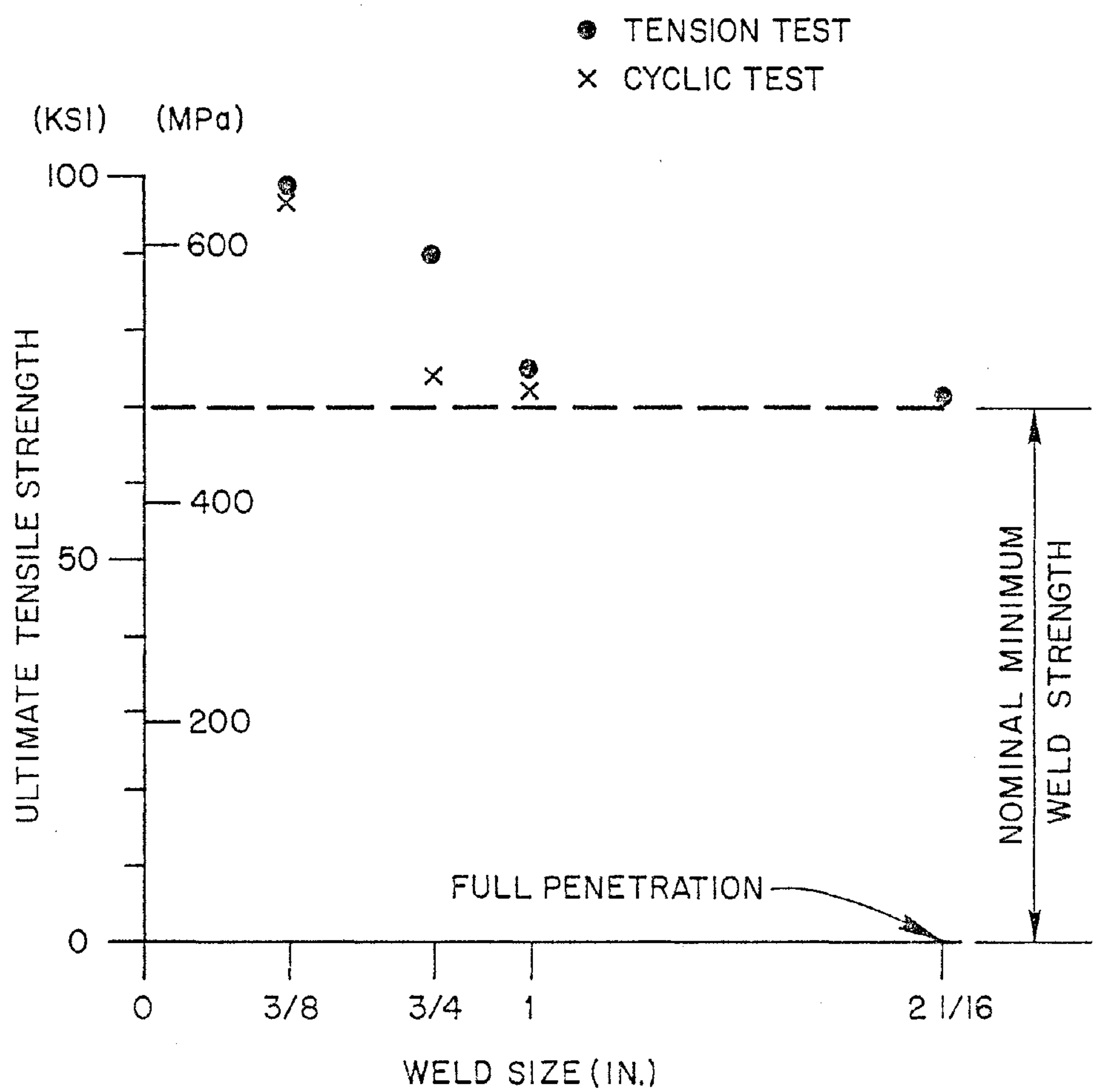

FIG. 12 ULTIMATE WELD STRENGTHS FOR DIFFERENT AMOUNTS OF WELD PENETRATION

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