NATIONAL BUREAU OF STANDARDS REPORT

5313

STRENGTH AND RIGIDITY OF LAP WELDED SPLICES
IN PRECAST REINFORCED CONCRETE FRAMING MEMBERS

by

L. F. Skoda, A. F. Kirstein, and D. Watstein

Report to

Bureau of Yards and Docks
Department of the Navy

U. S. DEPARTMENT OF COMMERCE
NATIONAL BUREAU OF STANDARDS

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Four box girders with lap welded splices were tested to determine the effect of rotation of eccentrically loaded lap welded reinforcing bars on the lateral rigidity of the splice and precast thin-shelled concrete framing members. Each specimen was made of six precast thin-shelled elements that were welded and spliced together to form a hollow box girder 1- by 2-ft in cross section and 8 ft 10 in. long. These girders were used to compare the relative rigidities of lap welded splices with and without transverse reinforcement. The comparative results of flexural tests of these girders indicate that there was little difference in the observed lateral displacements up to stresses in splices of about 35,000 psi. However, at higher stresses, the lateral movement of the end portion of the girders and the concrete surrounding the splices was significantly greater in the girders which contained no transverse reinforcement.

1. INTRODUCTION

In the previous tests of precast girders and Tee Heads with welded splices made at the request of the Bureau of Yards and Docks, it was observed that the eccentrically loaded lapped splices caused large displacements of the concrete which covered the welded splices in the joint. Four girders described in this report were tested to determine means of reducing the rotation of lap welded splices of the type used by the Department of the Navy in their precast thin-shelled structural framing members.

2. DESCRIPTION OF SPECIMENS AND MATERIALS

The type of specimen selected for this test was a combination of the modified center section of "Precast Continuous Girder," (BuY&D sketch B-3, dated 5-3-53), and the end blocks of the "Tee Head," (BuY&D sketch B-1, dated 1-12-53). This combination produced a six element specimen with four welded splices. The cross section of the girder was 1- by 2 ft and the length was 8 ft 10 in. Each element was essentially channel shaped in cross section as can be seen in the detail sketch of the resulting composite box girder shown in figure 1.

Transverse reinforcement was added to the welded splices in girder No. 1. Two No. 7 bars 10 in. long were selected to reinforce one of the splices and an equal volume of steel in the form of a plate 3- by 3/8- by 10 in. long of intermediate grade steel was used to reinforce the other. The welding was completed as indicated in detail views A and B of figure 1, by a qualified Navy welder. The transverse reinforcement added to the welded splices of girders Nos. 3 and 4 consisted of a single No. 2 bar that was tack welded in place simply to keep it in position during the grouting operation. Detail view C of figure 1 indicates the location of this bar. Girder No. 2 did not contain transverse reinforcement.

2.1 Forms

The forms used for casting the box girder elements were those used in the previous investigations of the Tee-Heads and continuous girders as reported in NBS Reports Nos. 2597 and 4286. The center section of the continuous girder form was shortened to 5 ft 2 in. while no changes were necessary in the Tee-Head end forms.

Although the forms had been used several times before, the plywood and 4- by 4-in. timber construction was sturdy enough to withstand additional repeated use.

2.2 Concrete

The concrete was made of Type III cement and had proportions of 1:2.45:2.02, by weight. A water-cement ratio of 0.49 was used to produce a fairly workable mix with a slump of 2 to 3 in. Both aggregates were siliceous materials that were obtained from White Marsh, Maryland. The coarse aggregate was a well rounded pea gravel with a maximum size of 3/8 in.

Two batches of concrete were required to cast each girder, and two 6- by 12-in. control cylinders were cast from each batch. All elements and their corresponding control cylinders were kept under damp burlap for 24 hr and then transferred to a curing chamber where they were moist-cured for six days. The control cylinders were tested on the seventh day, and the average compressive strengths for girders Nos. 1, 2, 3, and 4 were 5460, 4750, 6590, and 6525 psi, respectively. The elements were then air-dried until assembled and tested.

2.3 Reinforcement

The arrangement of longitudinal and web reinforcement is shown in figure 1. The reinforcement in the lower flanges of the center sections of each girder consisted of four No. 8 deformed bars. The two inner bars in the flanges of the channel sections were completely embedded while the outer bars extended 5 in. beyond the edge of the concrete. Similarly, the upper flanges of the center sections of each girder were reinforced with No. 4 bars.

The lower flanges of the end sections were reinforced with single No. 8 bars that extended beyond the form and lapped the extended No. 8 bar of the center section. Similarly, the upper flanges of the end sections were reinforced with No.4 bars.

Tensile tests were made of No. 8 reinforcing bars of the type used throughout this investigation. The average yield strength of the bars was 42,300 psi and the modulus of elasticity was 27×10^6 psi.

Three inclined stirrups of No. 4 bars were placed in the web of each element to resist diagonal tension. A bent No. 4 bar was also placed in the web of each element near the joint. The webs and flanges of each element were also reinforced throughout with 2- by 2-in. welded wire fabric of No. 6 gage. All the deformed reinforcing bars were of intermediate grade steel.

2.4 Girder assembly

After sufficient air drying, the girder elements were welded together to form the composite end sections and center section. These units were carefully positioned to form the girder, and the 4-in. lap welded splices of the reinforcement in the upper and lower flanges were made as indicated in figure 1.

The cavities that contained the lap welded splices were filled with concrete of the same mix proportions as was used in casting the girder elements. The 7-day compressive strength of this concrete for girders Nos. 1, 2, 3, and 4 was 6000, 4575, 6840, and 6840 psi, respectively.

3. TESTING PROCEDURE

3.1 Test setup

The girders were tested over an 8 ft span in a 600,000 lb capacity hydraulic testing machine. The load was applied applied through a loading beam with a knife edge at one end and a roller-plate assembly at the other. These load points were 40 in. apart and were placed symmetrically about the centerlines of the girders. Figure 2 is a sketch of a girder in the testing machine.

The girders were first flexed to a load of 5,000 lb, after which all load was removed and zero readings were taken on all gages. A complete set of readings was taken after each 10,000 lb increment of load until failure occurred.

3.2 Instrumentation

One thousandth inch dial gages with a range of one inch were used for all displacement measurements. Six of these gages were used to measure the lateral displacement of the concrete in the vicinity of each lap welded splice. These gages were supported by ring stands that rested on the platen of the testing machine. The vertical deflection at the center of the girder was measured by a dial gage on each side. These gages were attached to aluminum bars that were supported at their ends directly over the rocker supports of the girder. The location of all dial gages is shown in figure 2.

In addition to the dial gages, girders Nos. 3 and 4 had electrical resistance type strain gages attached to the No. 8 reinforcing bars of the center section which projected into the splice section. The location of the gages near the splice and at mid-span can be seen in detail C and Section 1-1 of figure 1. Readings on these gages were taken at the same time as the dial gages were being read.

4. RESULTS AND DISCUSSION

The primary cause of failure in the flexural tests of the girders appeared to be yielding of the tensile reinforcement in the splices. This is indicated by the views of the girders after test in figures 3 through 10 and by the load-strain relationship for girders 3 and 4 shown in figure 11. The views of all girders tested show wide cracks at the straight boundary of the grout pocket. These cracks were sufficiently wide in most of the specimens to reduce the compressive zone of the concrete to a depth which permitted shear failure to occur as a secondary mode of failure.

Girder No. 1, containing the most transverse reinforcement failed at a load of 174,000 lb while girder No. 2 having no transverse reinforcement failed at a load of 150,000 lb. Girders Nos. 3 and 4 containing a small amount of transverse reinforcement failed at identical loads of 155,000 lb.

The observed stresses in the longitudinal reinforcing steel at the centerline of girders Nos. 3 and 4 are shown in figure 12. Since the stress at the lap welded splice is of greater magnitude than the stress at the midspan, it is the limiting factor when considering the yielding of the reinforcement. The stresses observed at the lap welded splices of girders 3 and 4 are compared with the theoretical stresses in figure 11.

The lateral displacements of the concrete surrounding the welded splices and the adjoining elements are shown in the diagrams of figures 13 through 18. The load-displacement relationships for all girders are shown in the same manner for each gage position. The values of displacement shown on these curves are the total changes in the transverse dimensions of the specimens calculated as the algebraic sums of the movements measured by gages on opposite sides of the girders.

The load-displacement relationships for the girders were essentially the same up to a load of about 80 kips which corresponds to a theoretical stress in the steel at the splice that is approximately 15 percent below the yield strength of the material. However, as the load was increased, the load displacement curves for the girders displayed an increasing disparity. This difference in lateral rigidity is directly attributed to the effect of the various amounts of transverse reinforcement in girders Nos. 1, 3, and 4.

The load deflection diagrams for the girders are shown in figure 19. Attention is directed to the fact that the greater rotation of the splices in girder No. 4 had no appreciable effect on the center deflection.

Comparison of the displacement graphs in figures 13 through 16 shows that for loads exceeding 80,000 lb the lateral movement of the end sections of the girders and of the concrete surrounding the splices was significantly greater for the girder with no transverse reinforcement than for the three girders which contained varying amounts of transverse reinforcement. The theoretical steel stress in the splices at the load of 80,000 lb was approximately 35,000 psi. At higher stresses, the effect of the transverse reinforcement became particularly pronounced in reducing the lateral movement of the concrete surrounding the splices. As can be seen in figures 15 and 16, the maximum observed increases in the transverse dimensions of the splice pockets were about 0.01 in. for girder No. 1, 0.03 to 0.06 in. for girders Nos. 3 and 4 and about 0.25 in. for girder No. 2 which had no transverse reinforcement.

Figure 20 through 23 are crack patterns of the girders. The encircled numbers at the bottom of the crack pattern diagrams indicate the order in which the cracks appeared. Due to the fact that the lap welded splices in girder No. 2 rotated, large longitudinal and transverse cracks were evidenced in the concrete pocket while no cracking occurred in the end sections of the girder. Since the transverse reinforcement in girders Nos. 1, 3, and 4, reduced the rotation of the lap welded splices, a wider dispersion of small cracks appeared along their entire length.

It is concluded that a small amount of transverse reinforcement in the splices is well worth the slight additional cost in order to guard against excessive rotation of eccentrically tensioned lap welded splices and the accompanying lateral displacement of the concrete in the splice pockets which are observed at steel stresses in excess of 35,000 psi.

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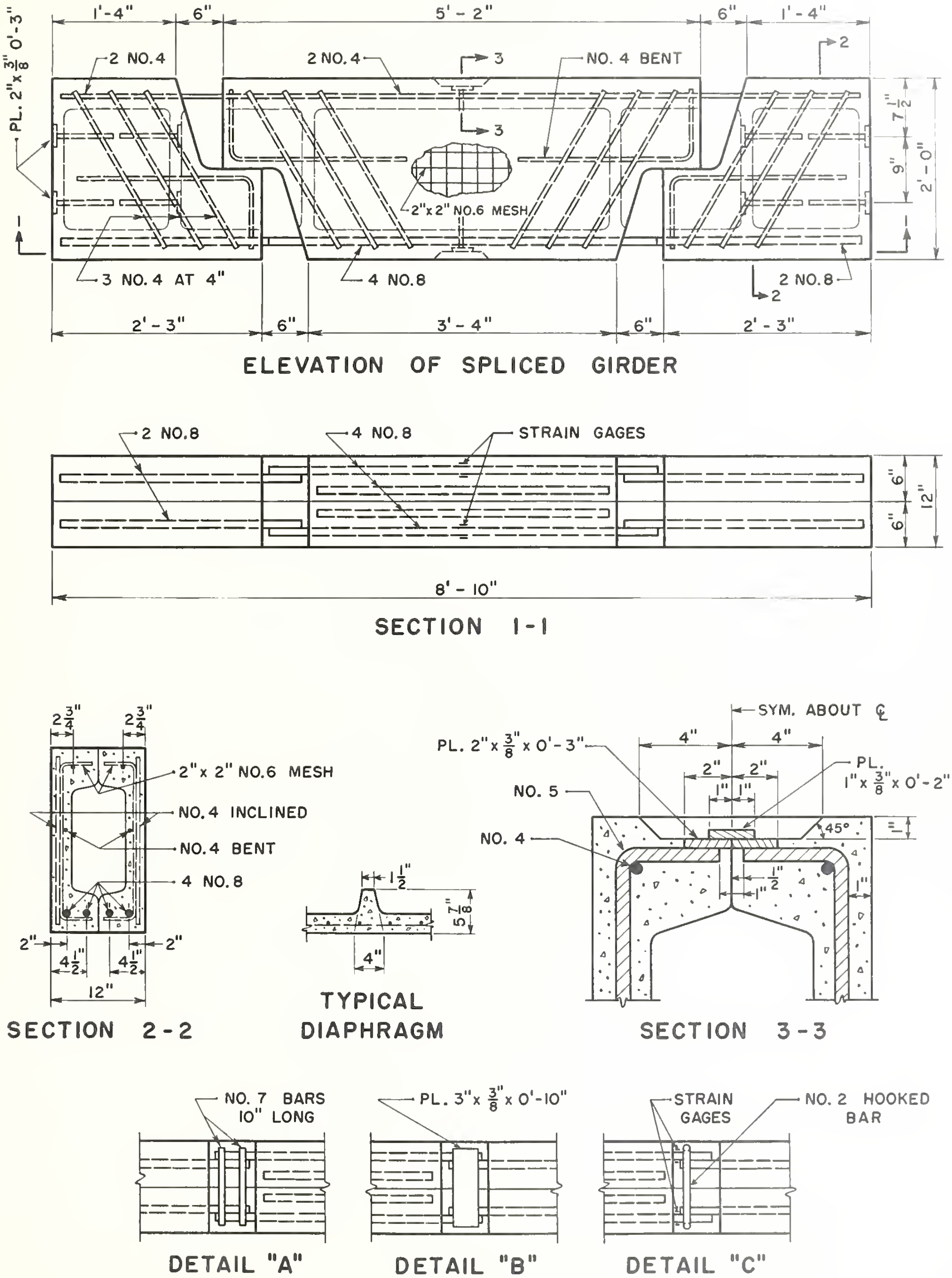

1'-4"
6"
5'-2"
6"
1'-4"
PL. 2" x 3/8" 0'-3"
2 NO. 4
2 NO. 4
3
3
NO. 4 BENT
2
7 1/2"
9"
2'-0"
2" x 2" NO. 6 MESH
3 NO. 4 AT 4"
4 NO. 8
2 NO. 8
2
1
1
2'-3"
6"
3'-4"
6"
2'-3"
ELEVATION OF SPLICED GIRDER
2 NO. 8
4 NO. 8
STRAIN GAGES
6"
6"
12"
8'-10"
SECTION 1-1
2 3/4"
2 3/4"
2" x 2" NO. 6 MESH
NO. 4 INCLINED
NO. 4 BENT
4 NO. 8
2"
2"
4 1/2"
4 1/2"
12"
SECTION 2-2
1 1/2"
5 7/8"
4"
TYPICAL DIAPHRAGM
SYM. ABOUT ℄
PL. 2" x 3/8" x 0'-3"
4"
4"
2"
2"
1"
1"
PL. 1" x 3/8" x 0'-2"
NO. 5
NO. 4
45°
1"
1/2"
1"
1"
SECTION 3-3
NO. 7 BARS 10" LONG
PL. 3" x 3/8" x 0'-10"
STRAIN GAGES
NO. 2 HOOKED BAR
DETAIL "A"
DETAIL "B"
DETAIL "C"

FIG. 1

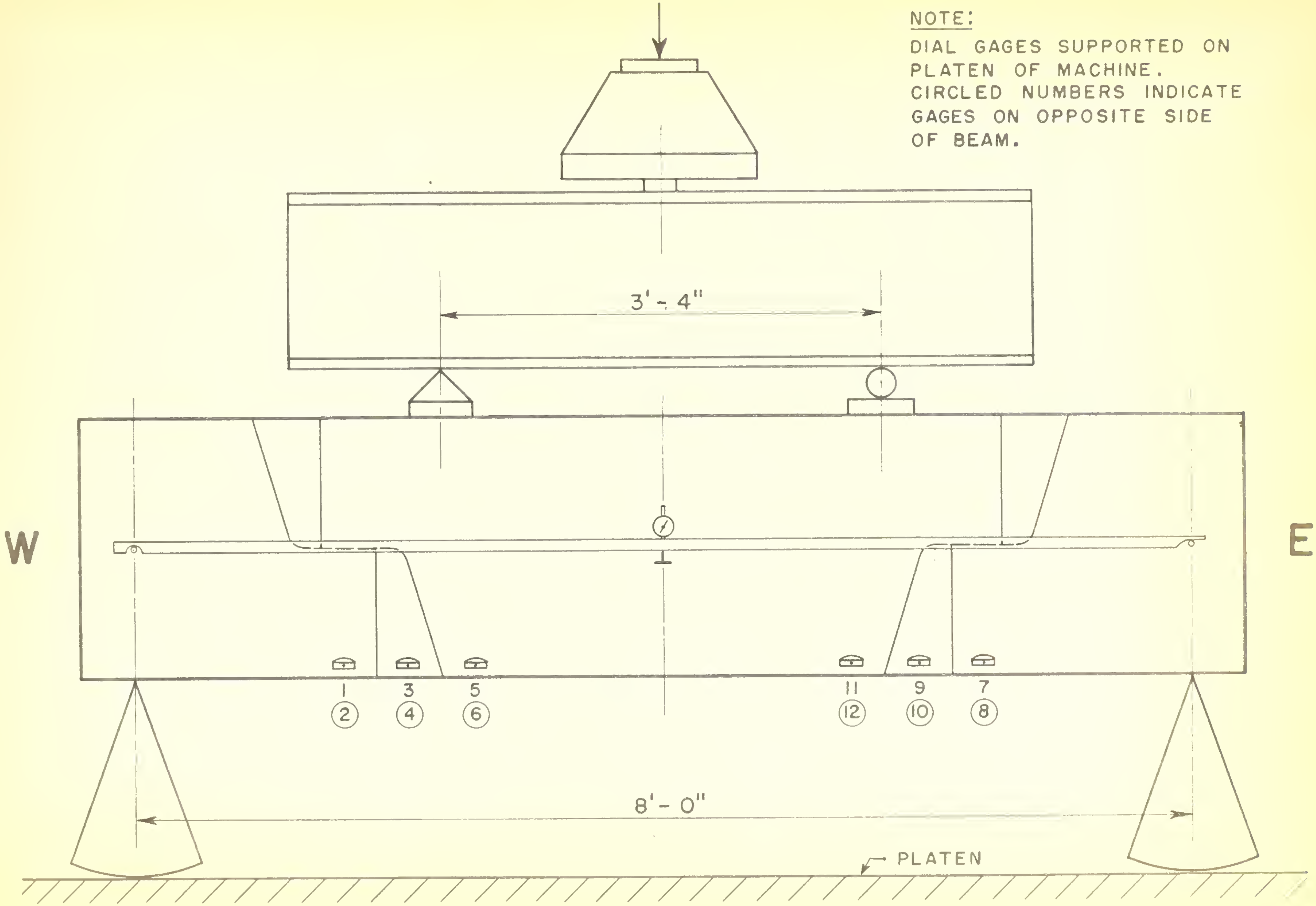

FIG. 2 TEST SETUP

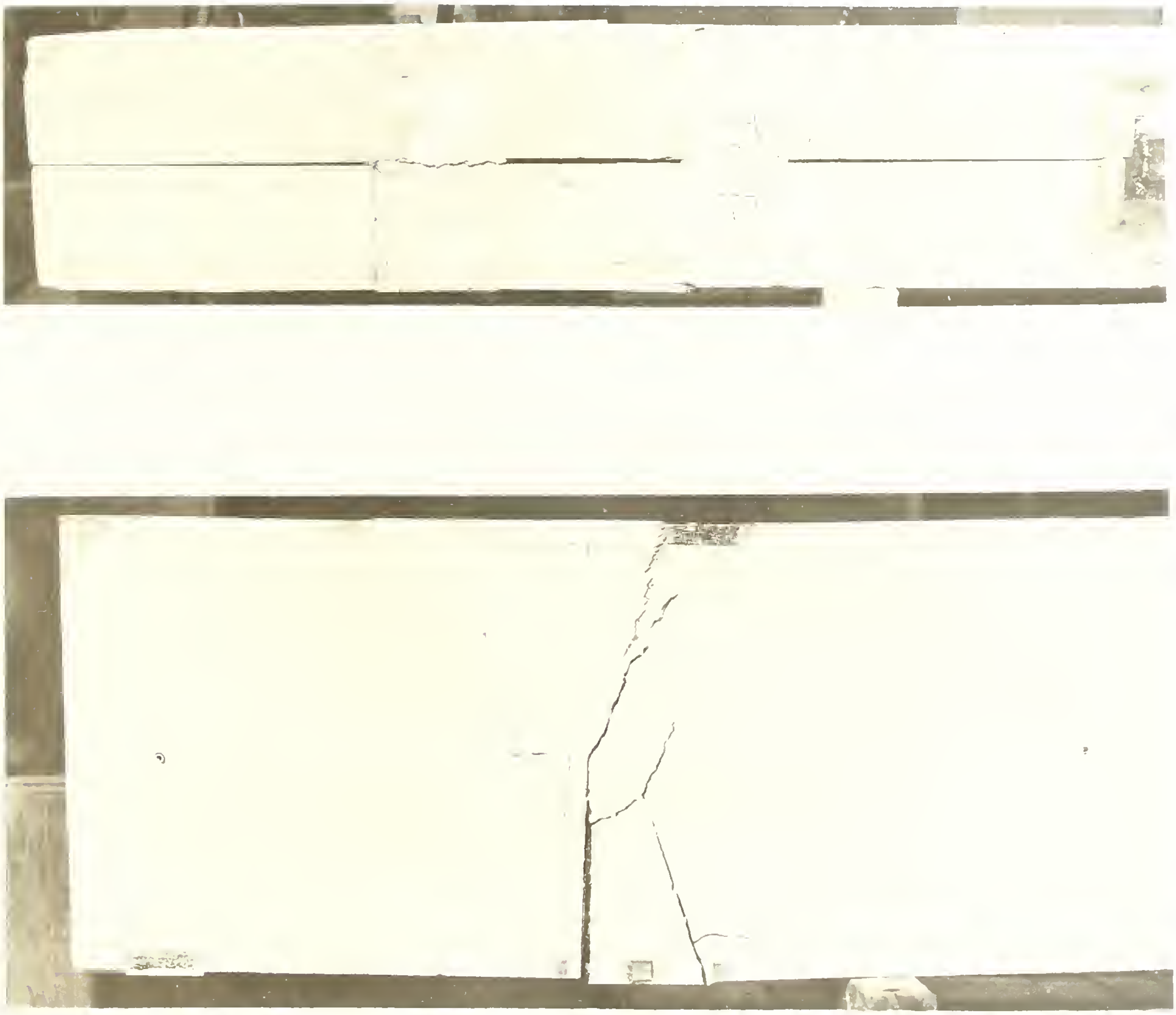

FIG. 3 VIEW OF FAILURE, GIRDER NO. I SOUTHWEST & TOP.

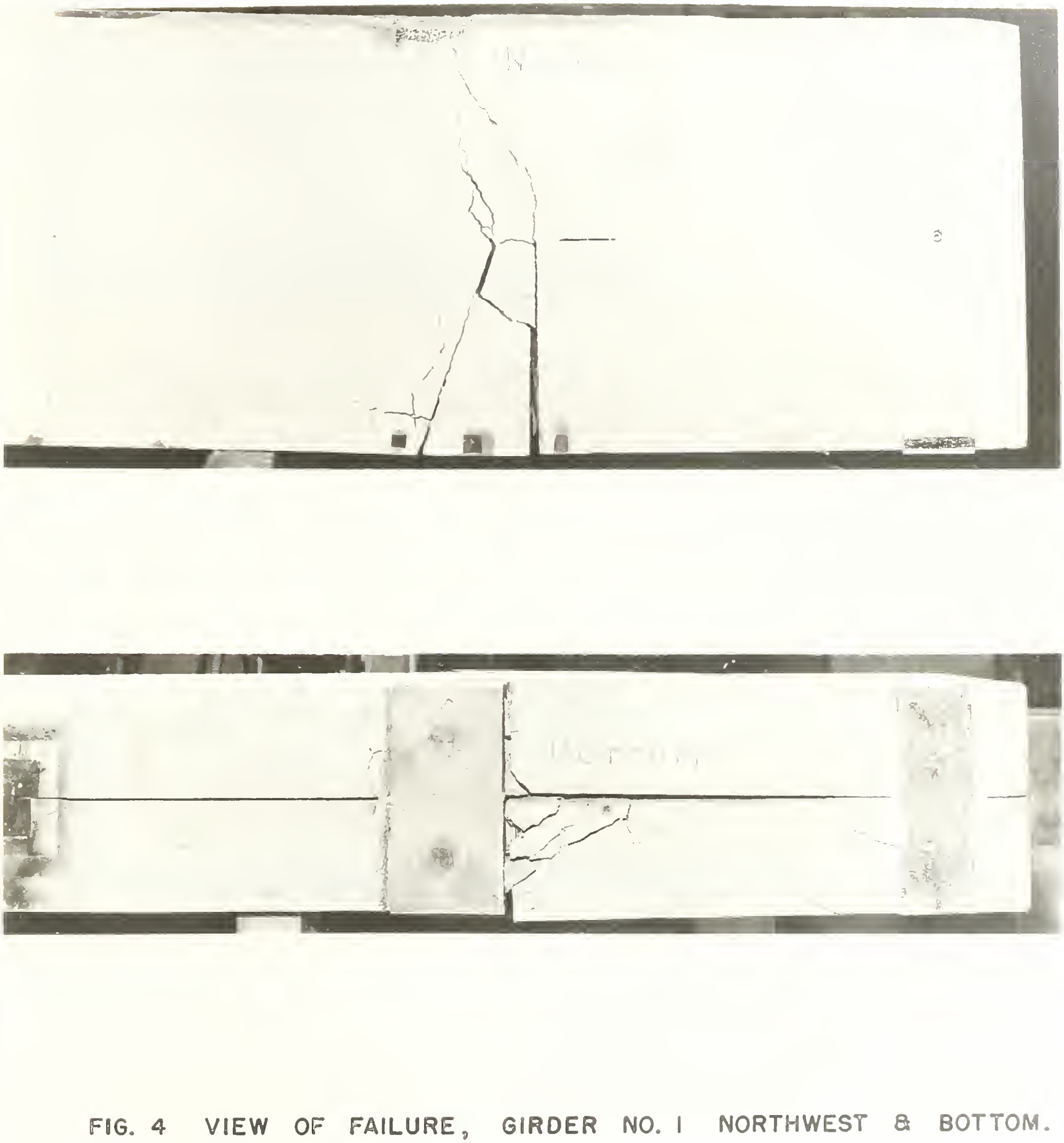

FIG. 4 VIEW OF FAILURE, GIRDER NO. 1 NORTHWEST & BOTTOM.

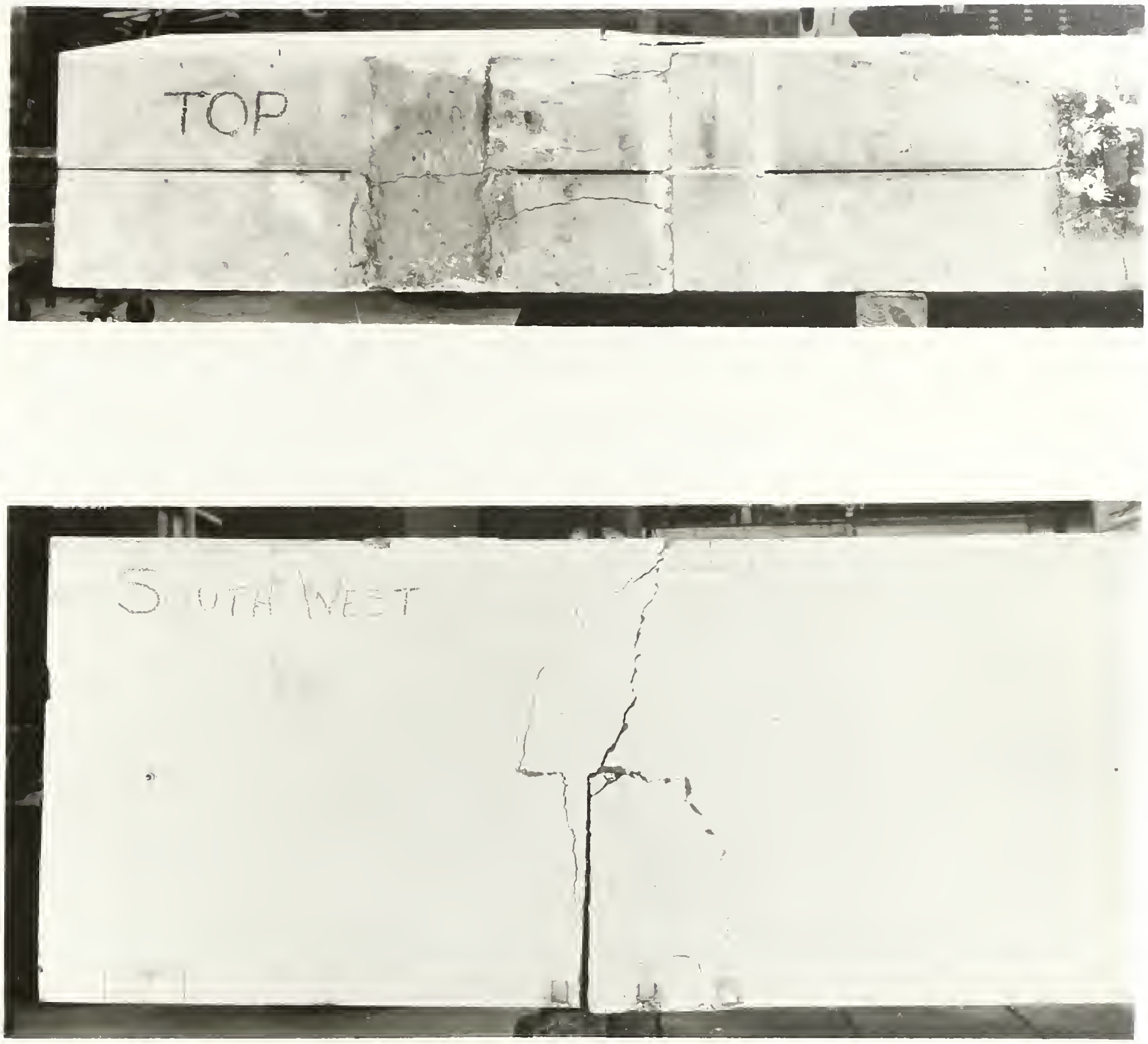

FIG. 5 VIEW OF FAILURE, GIRDER NO. 2 SOUTHWEST & TOP.

FIG. 6 VIEW OF FAILURE, GIRDER NO. 2 NORTHWEST & BOTTOM.

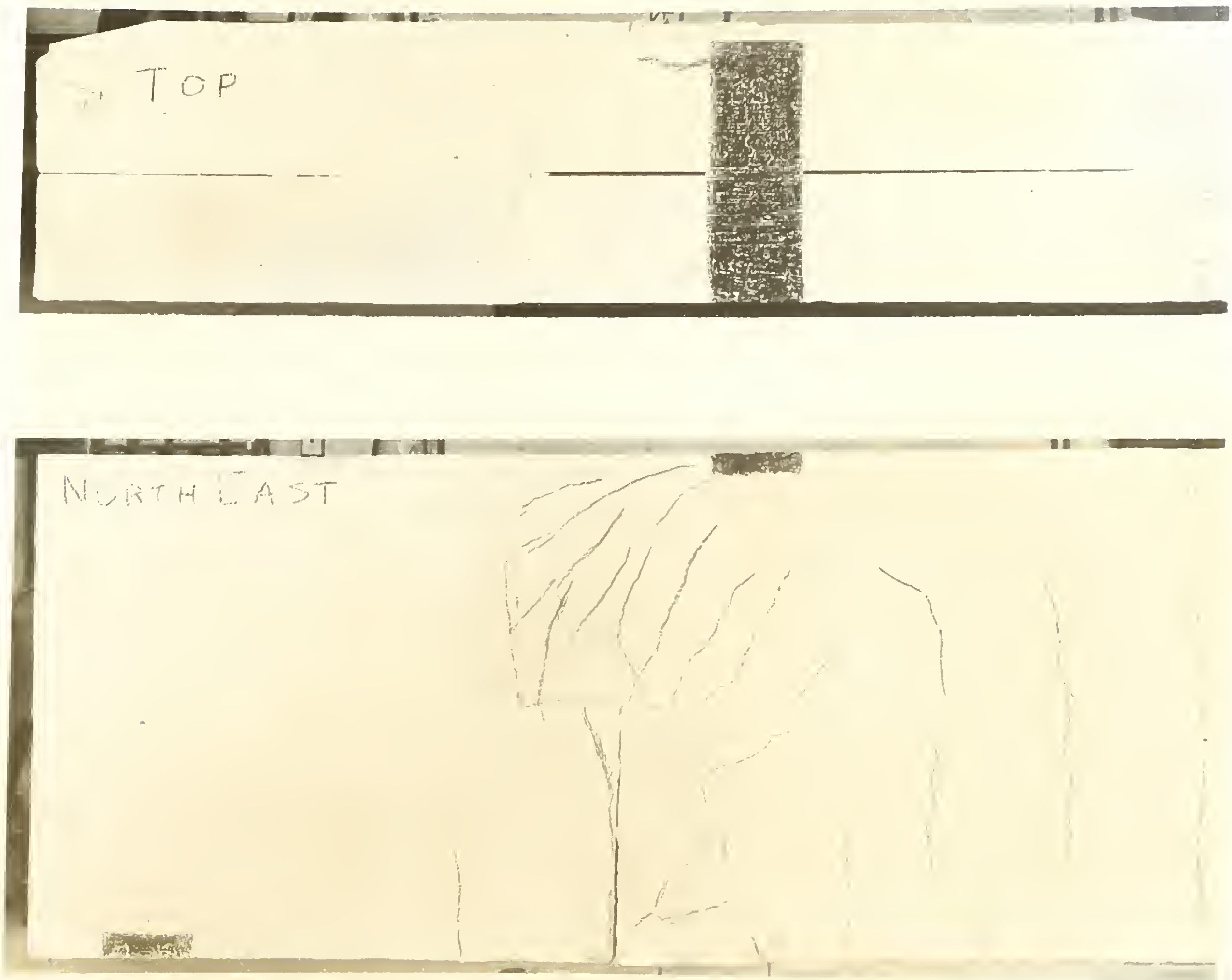

FIG. 7 VIEW OF FAILURE, GIRDER NO. 3 NORTHEAST & TOP.

FIG. 8 VIEW OF FAILURE, GIRDER NO. 3 SOUTHEAST & BOTTOM.

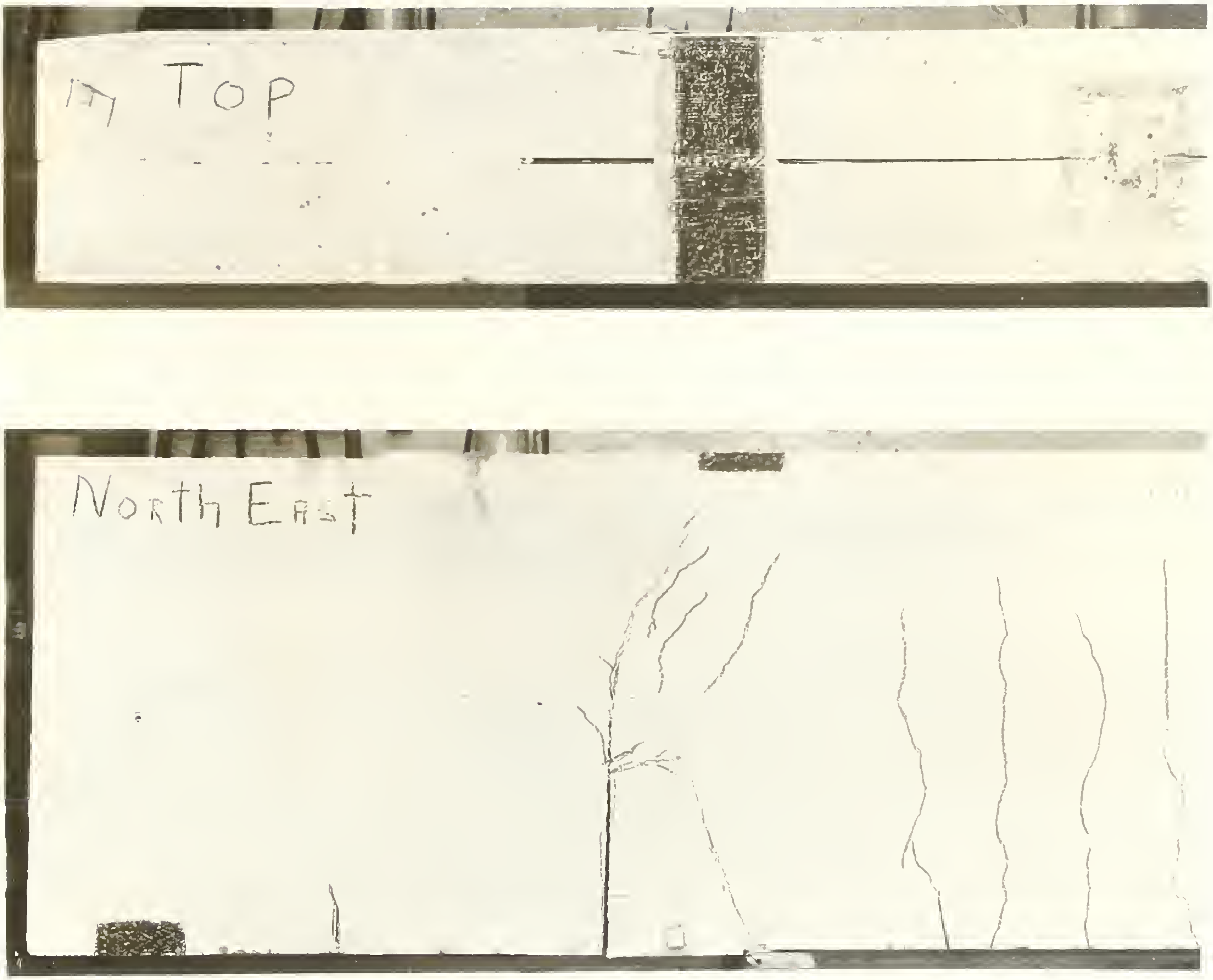

FIG. 9 VIEW OF FAILURE, GIRDER NO. 4 NORTHEAST & TOP.

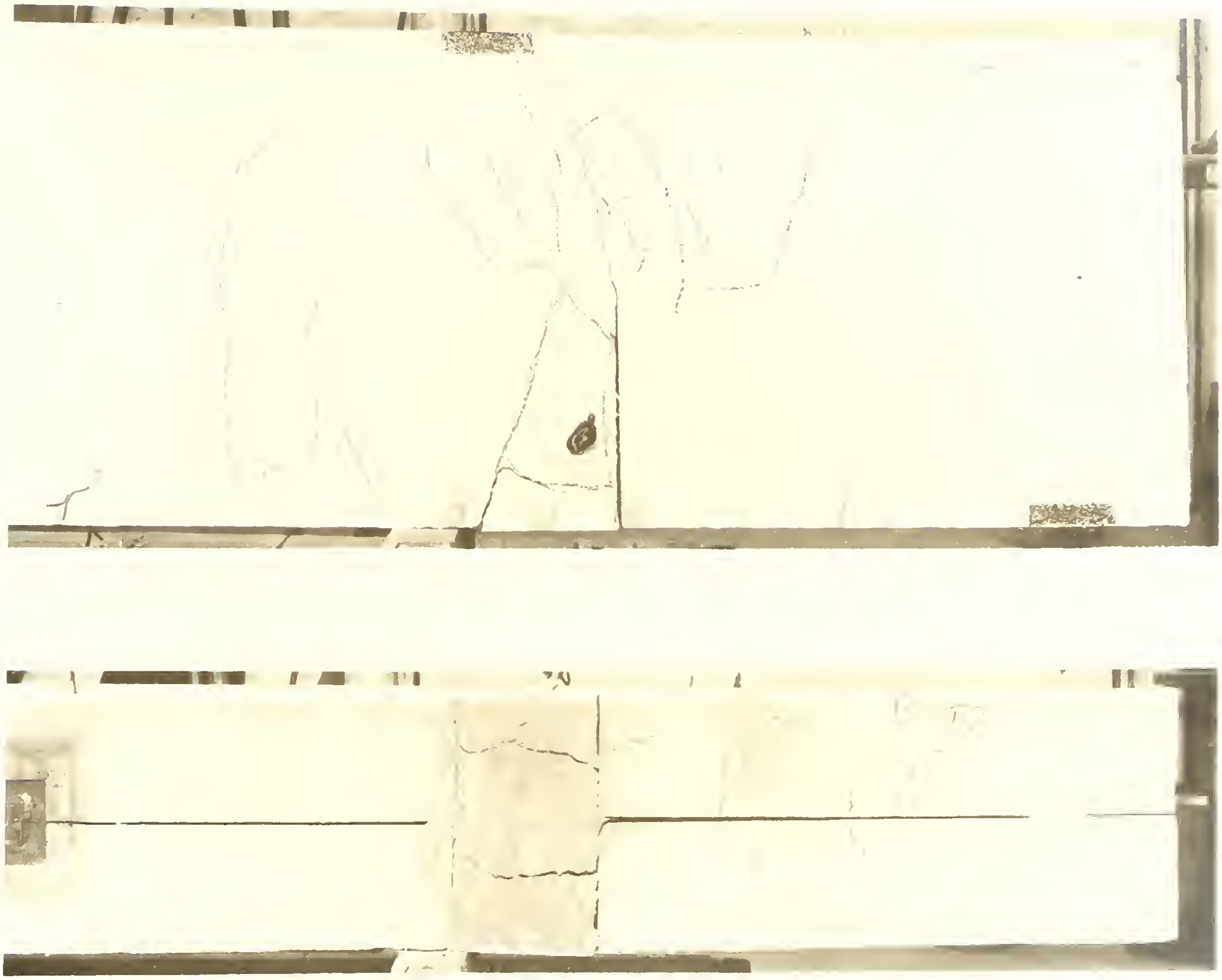

FIG. 10 VIEW OF FAILURE, GIRDER NO. 4 SOUTHEAST & BOTTOM.

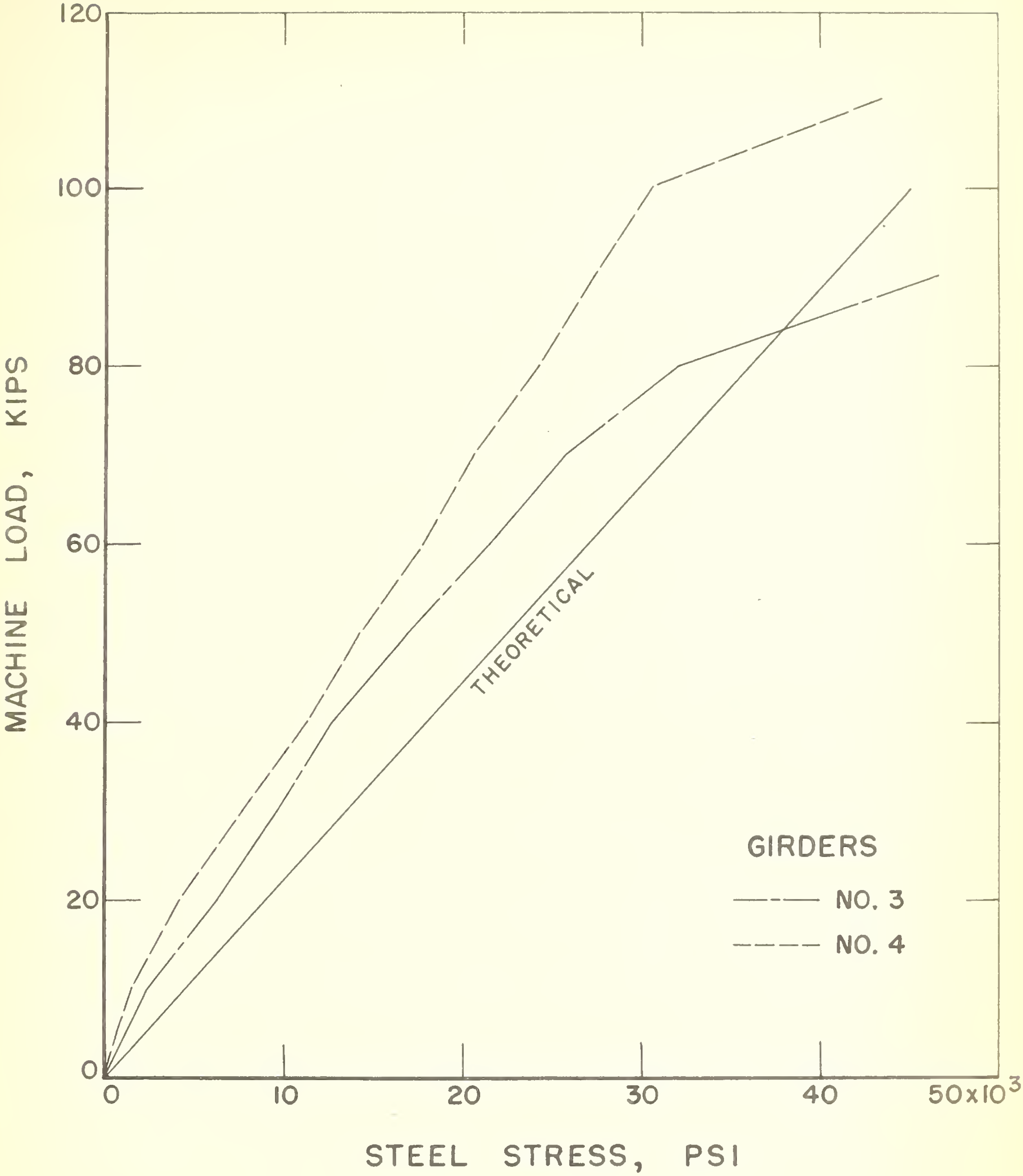

FIG. 11 OBSERVED AND THEORETICAL STRESSES AT SPLICES OF GIRDERS NOS. 3 AND 4.

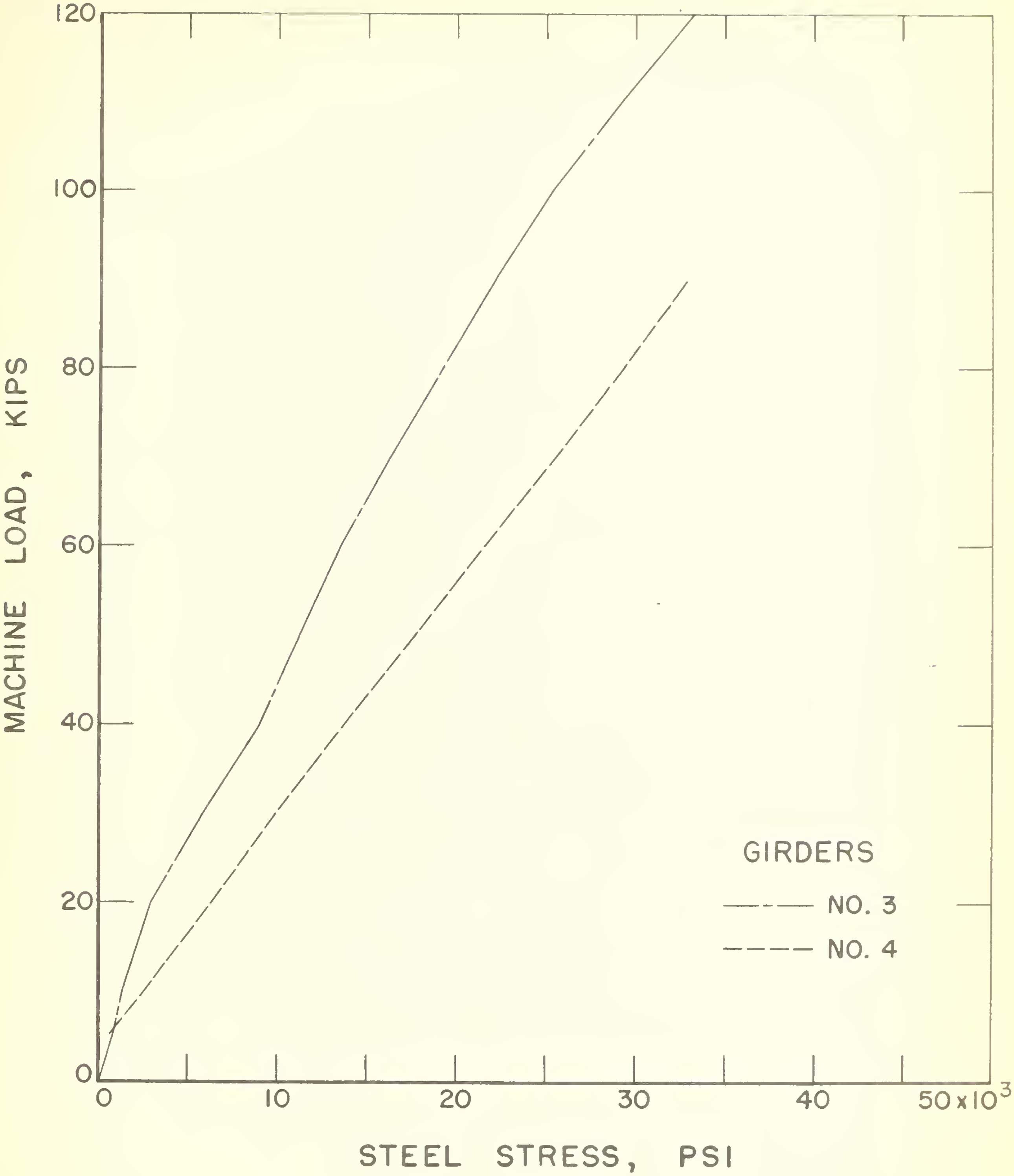

FIG. 12 OBSERVED STRESSES AT MID-SPAN, GIRDERS NOS. 3 AND 4.

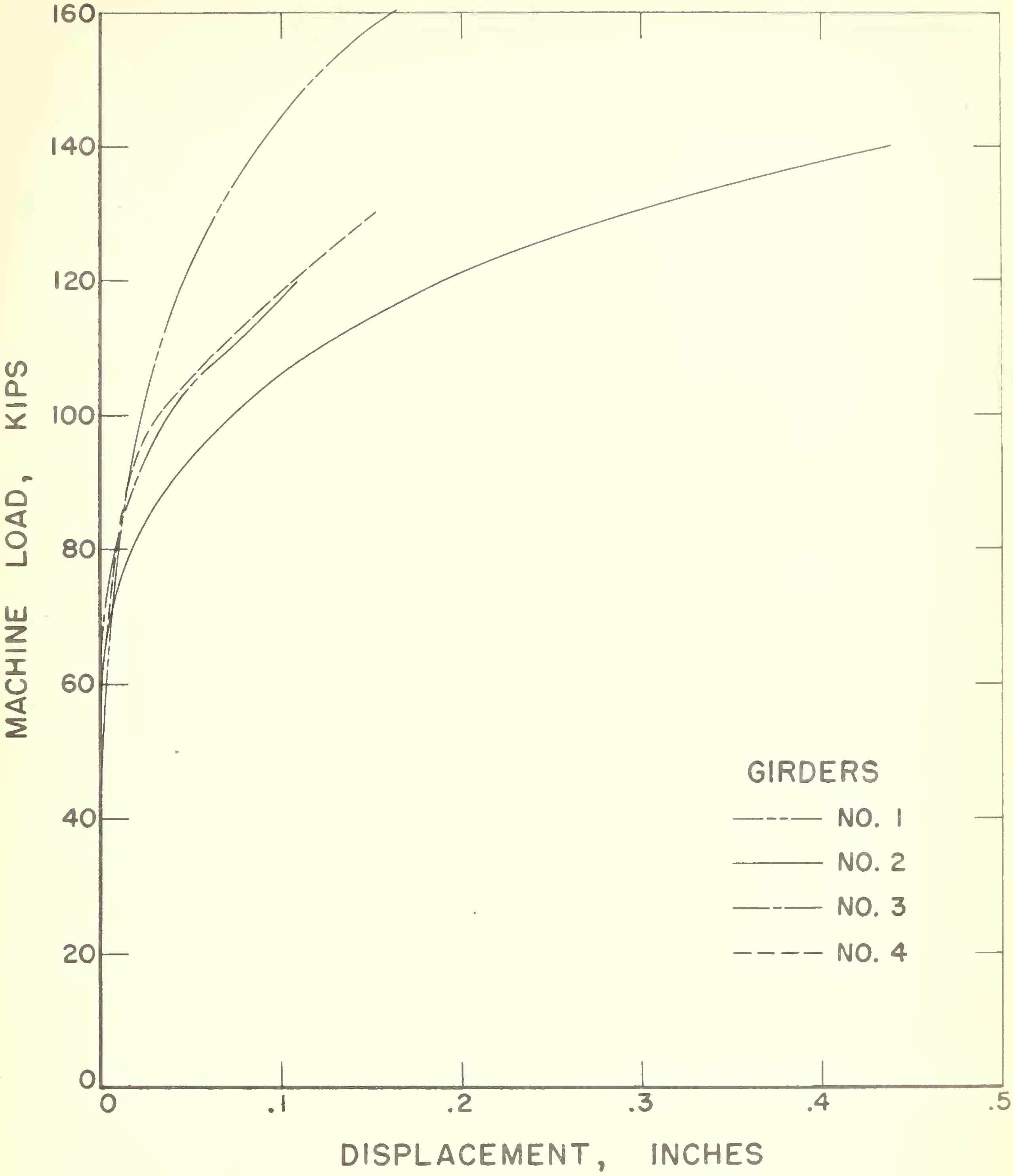

FIG. 13 LATERAL DISPLACEMENT, EAST END SECTION OF ALL GIRDERS.

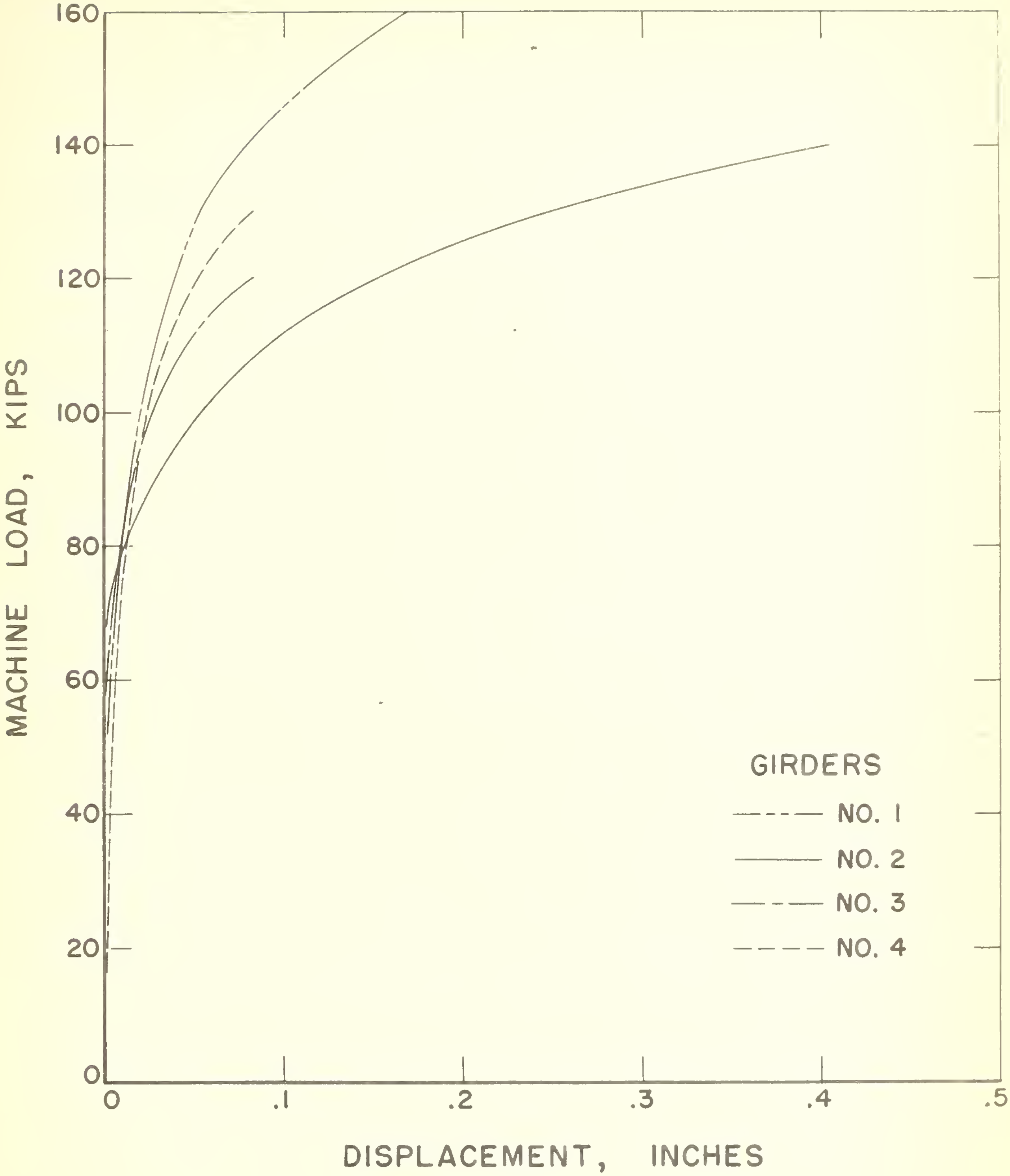

FIG. 14 LATERAL DISPLACEMENT, WEST END SECTION OF ALL GIRDERS.

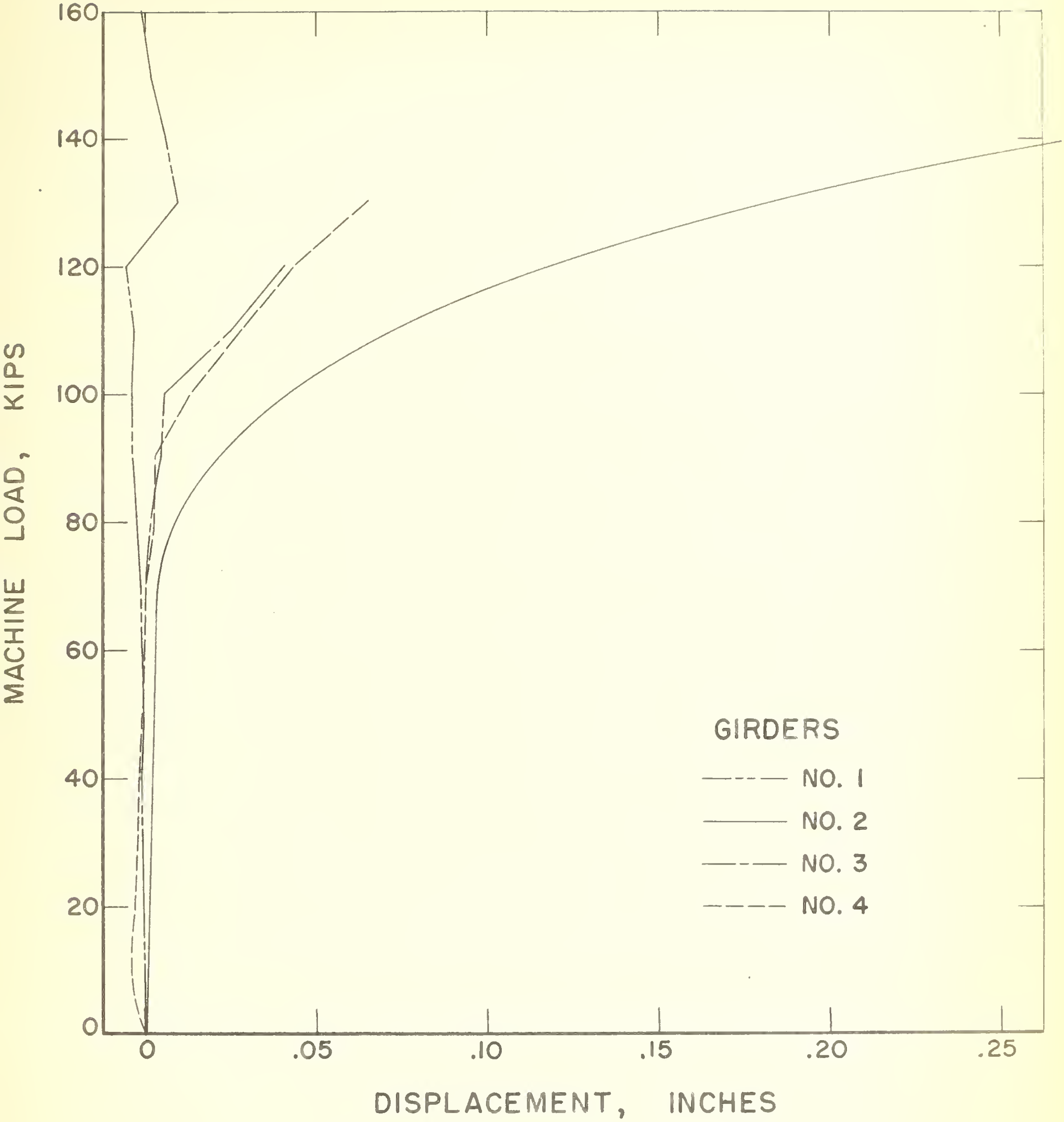

FIG. 15 LATERAL DISPLACEMENT, EAST SPLICE POCKET IN ALL GIRDERS.

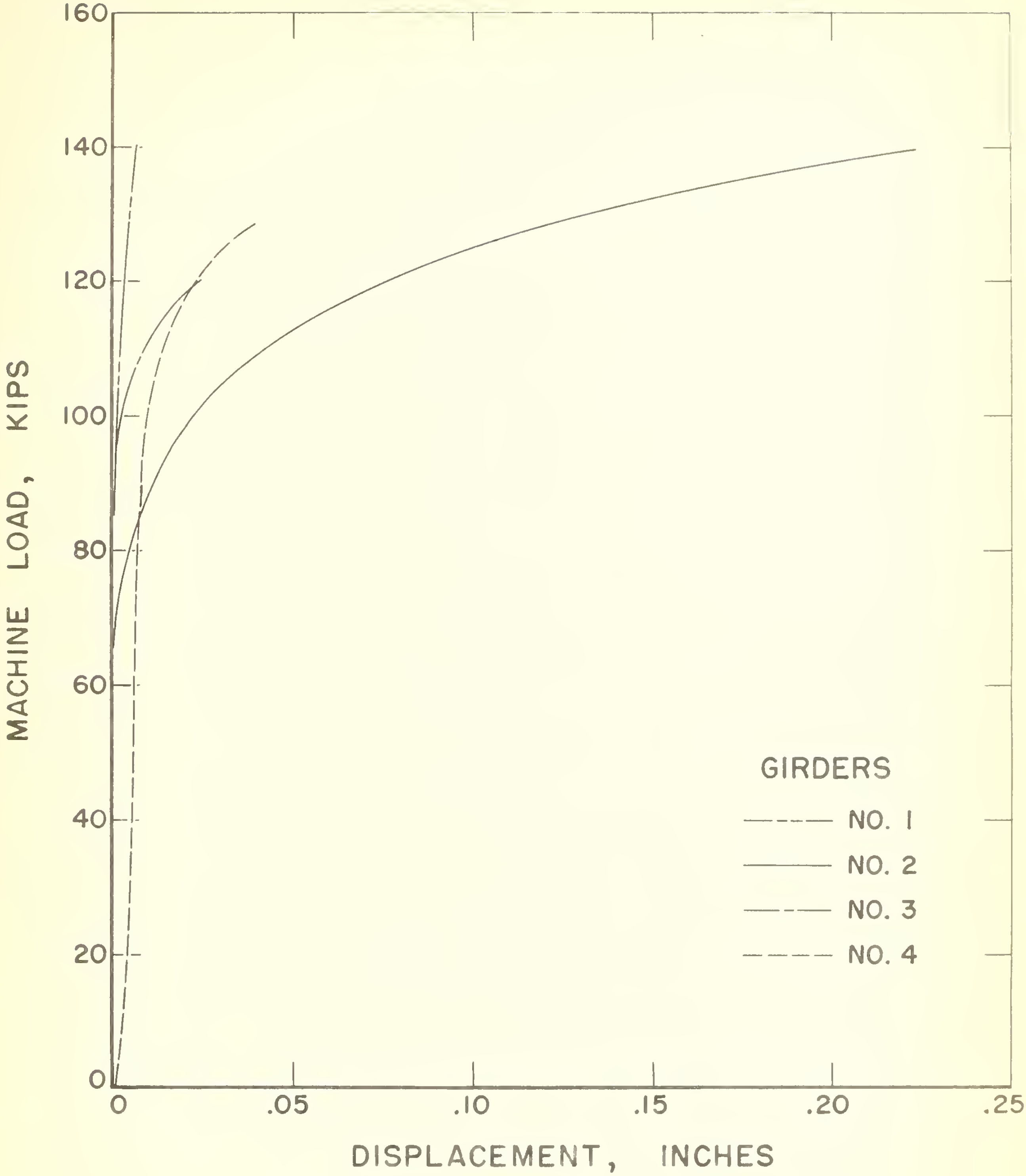

FIG. 16 LATERAL DISPLACEMENT, WEST SPLICE POCKET IN ALL GIRDERS.

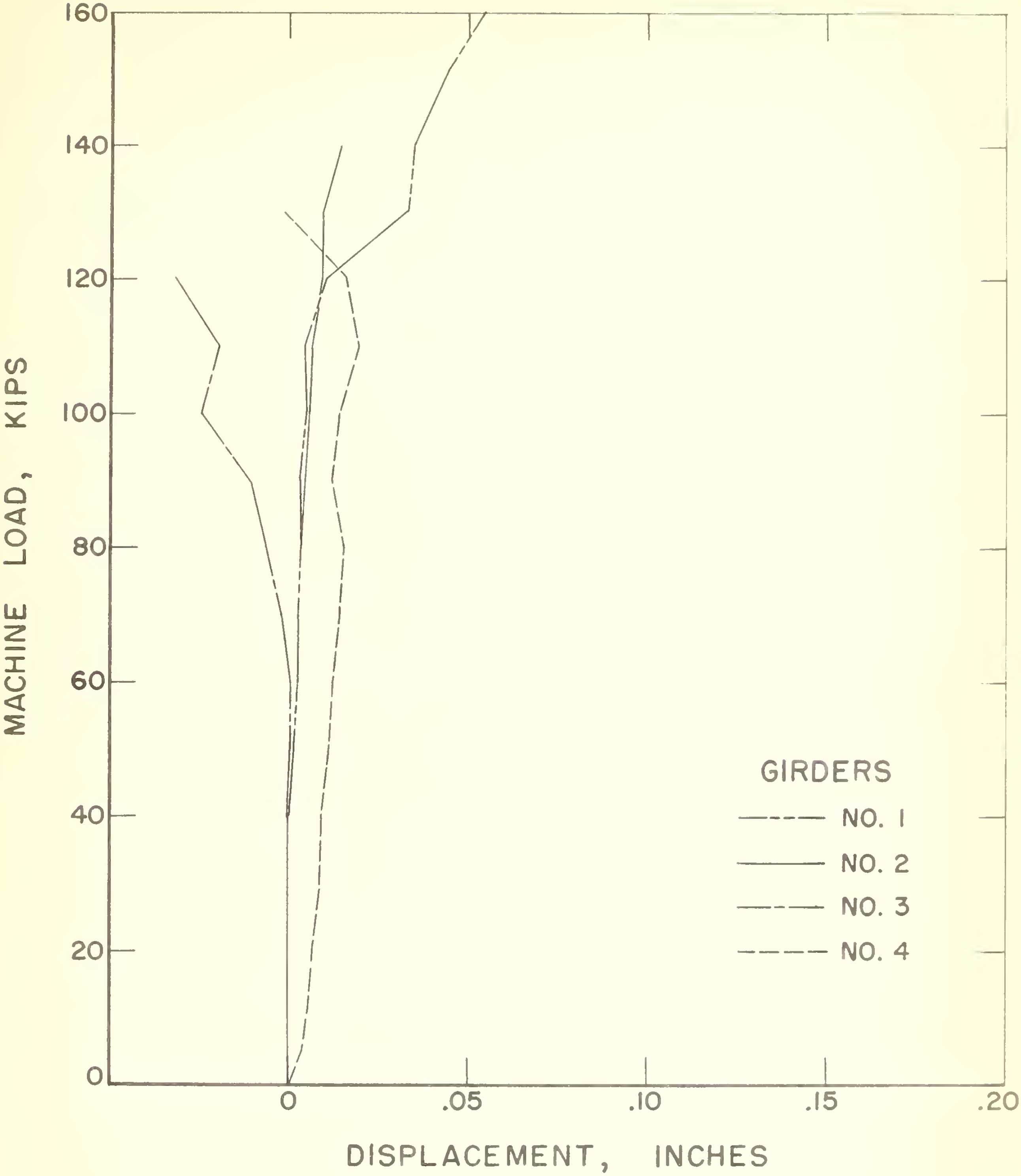

FIG. 17 LATERAL DISPLACEMENT, EAST END OF CENTER SECTION OF ALL GIRDERS.

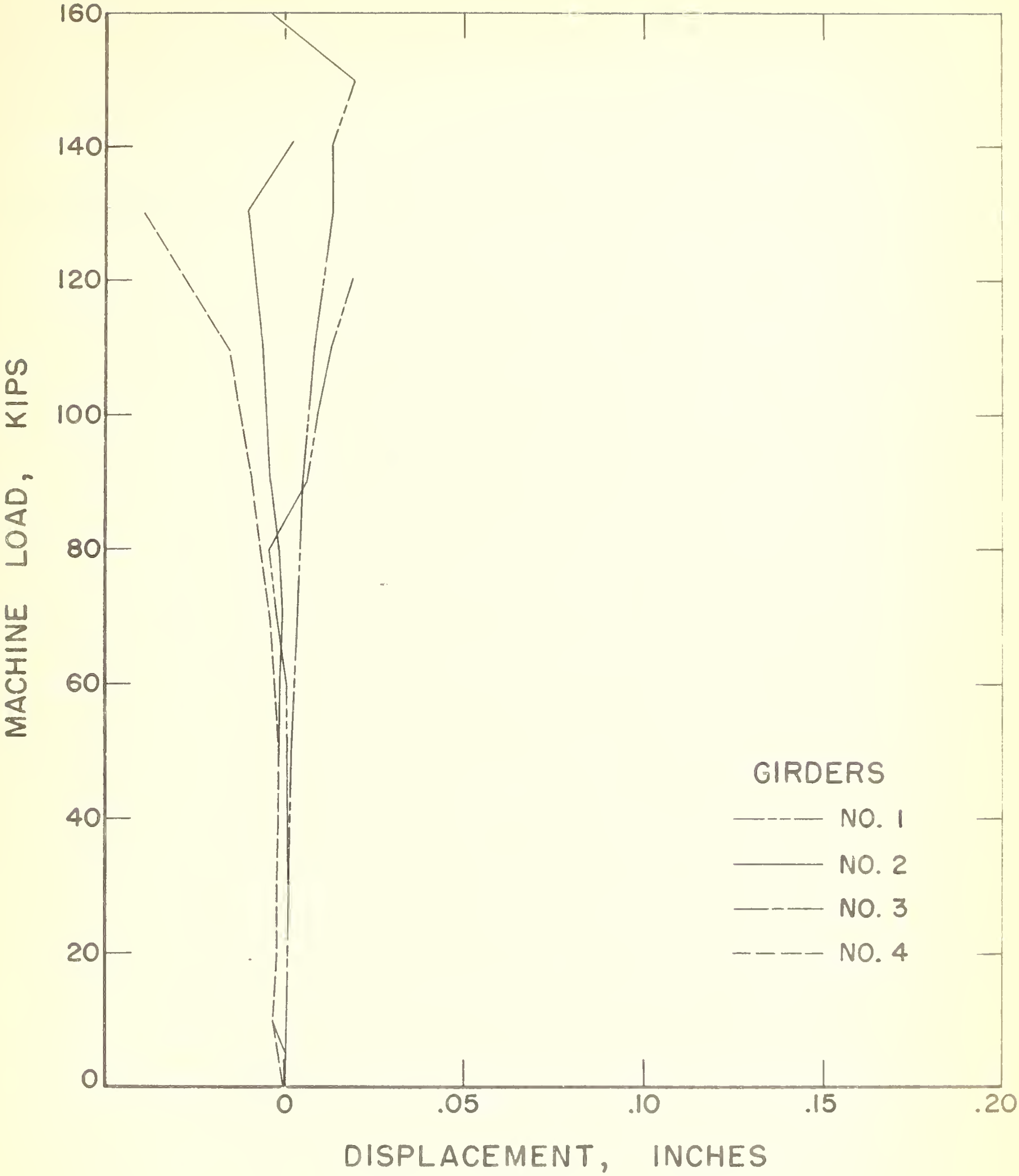

FIG. 18 LATERAL DISPLACEMENT, WEST END OF CENTER SECTION OF ALL GIRDERS.

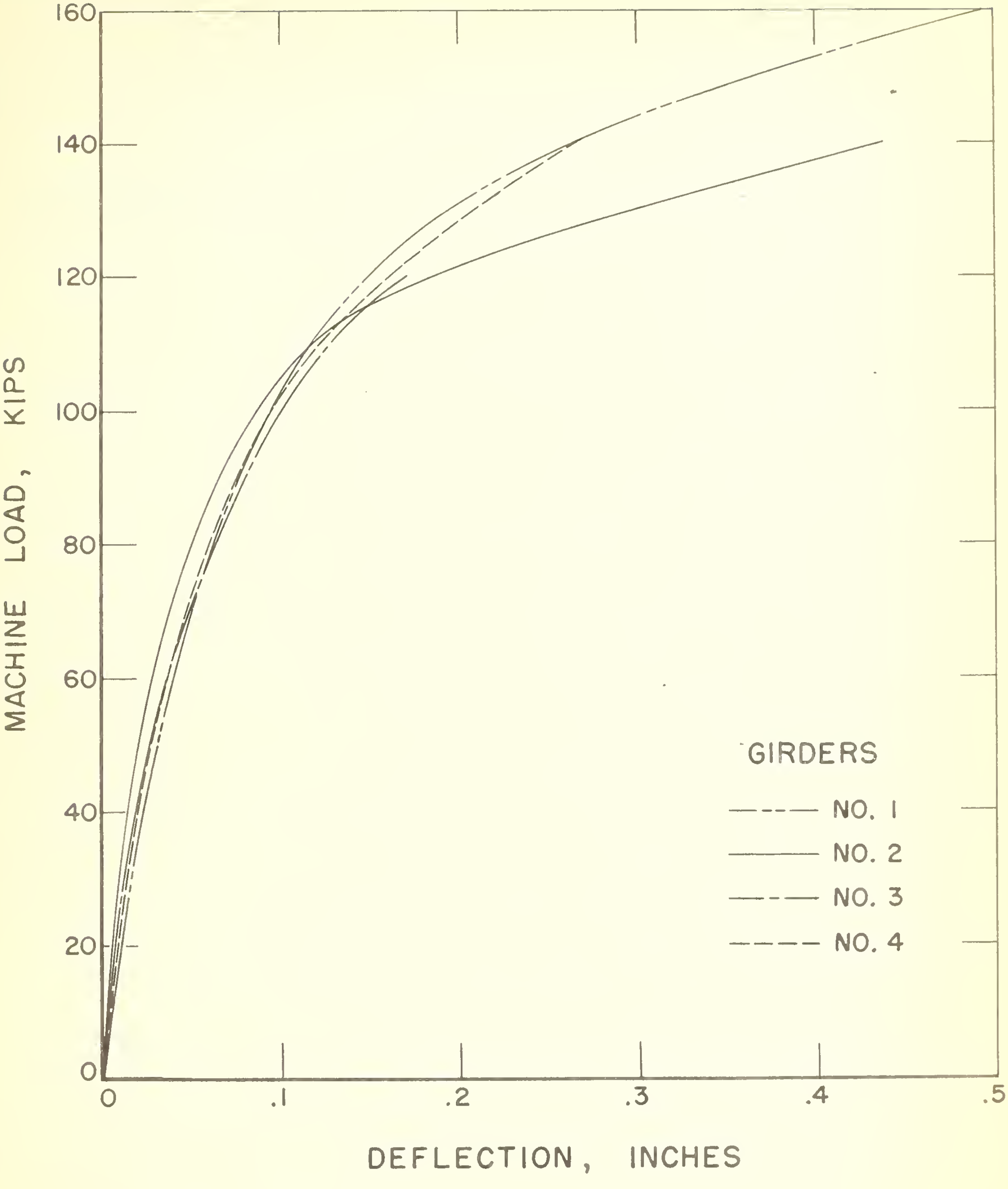

FIG. 19 LOAD VS. CENTER DEFLECTION

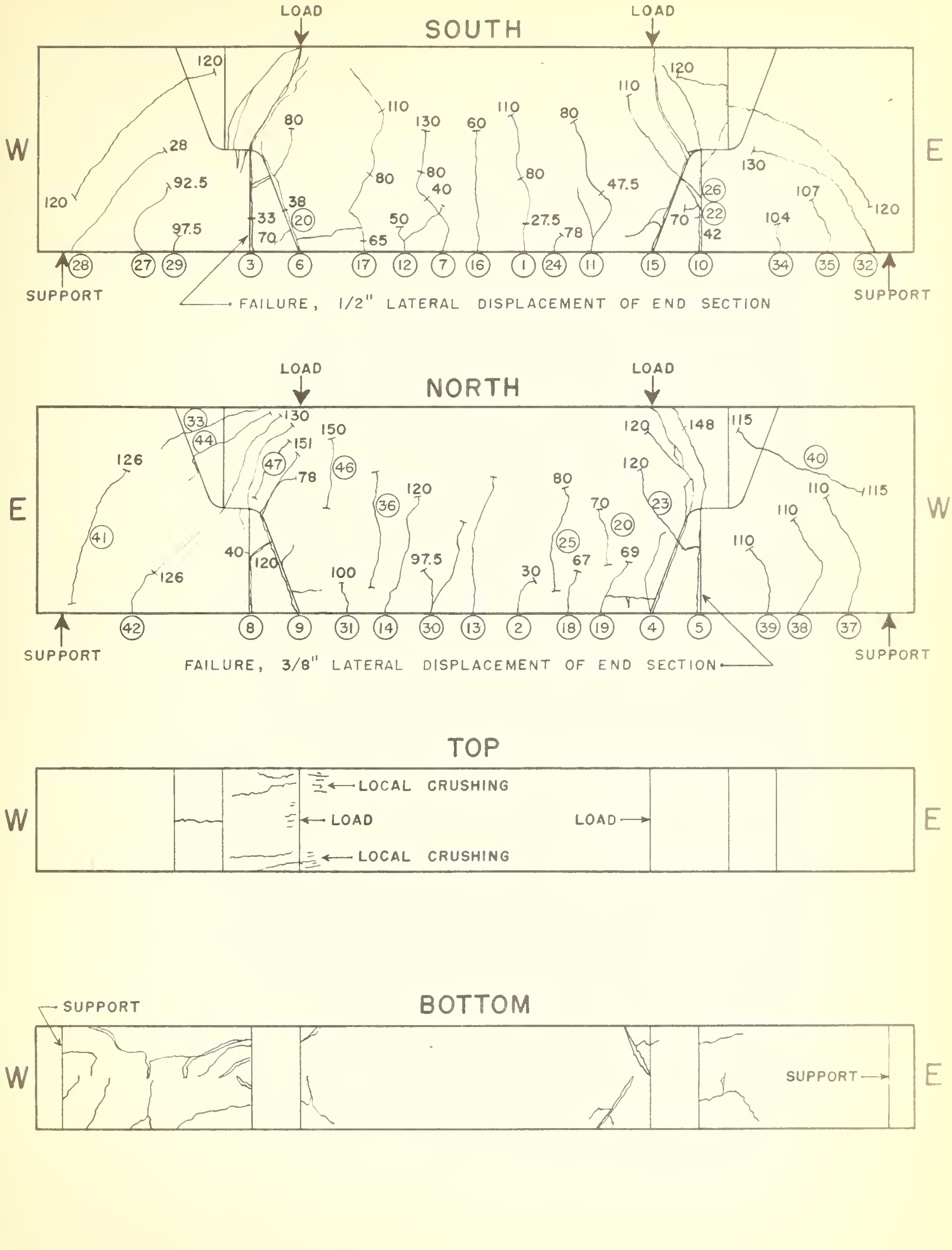

FIG. 20 CRACK PATTERNS, GIRDER NO. 1.

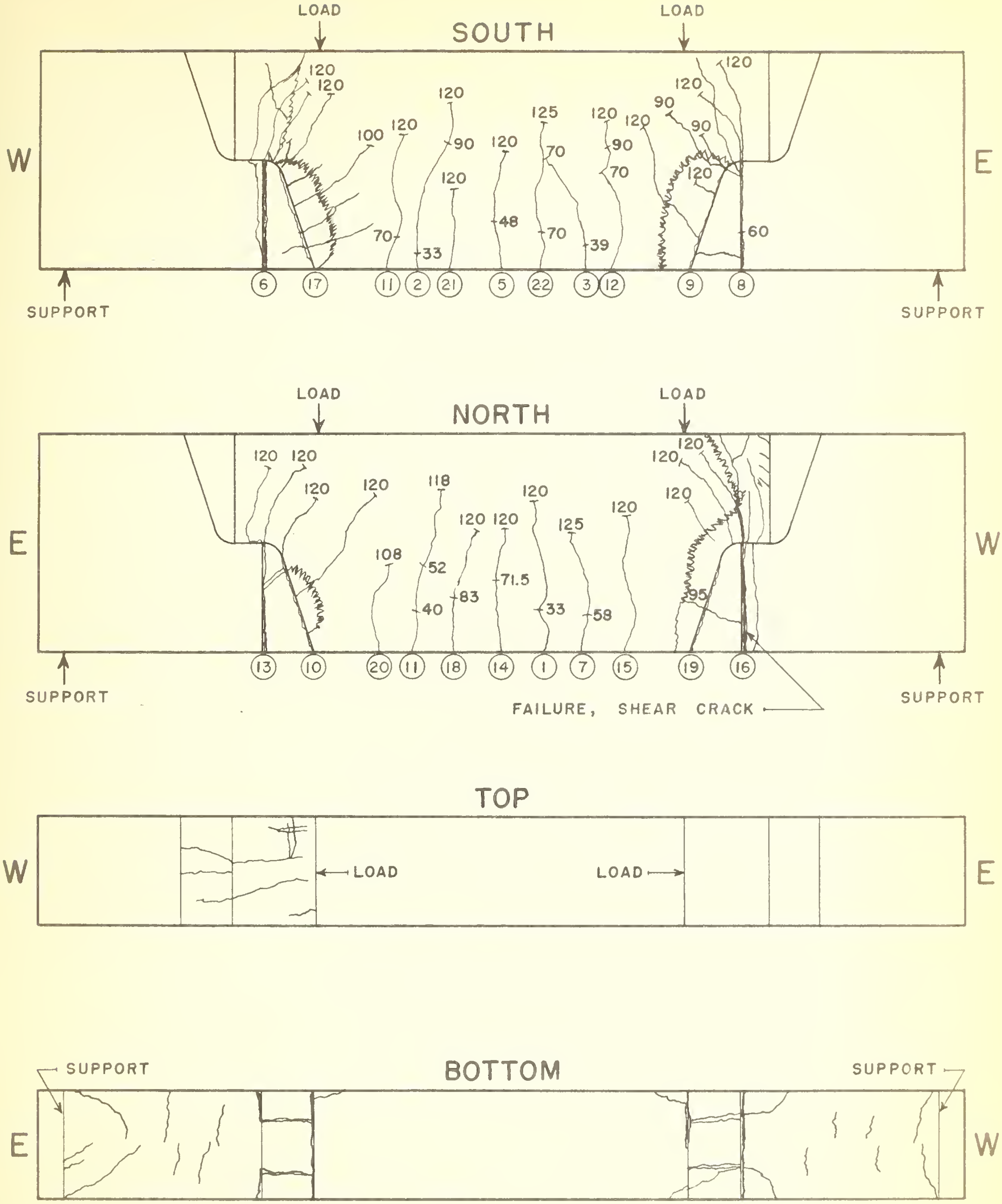

FIG. 21 CRACK PATTERNS, GIRDER NO. 2.

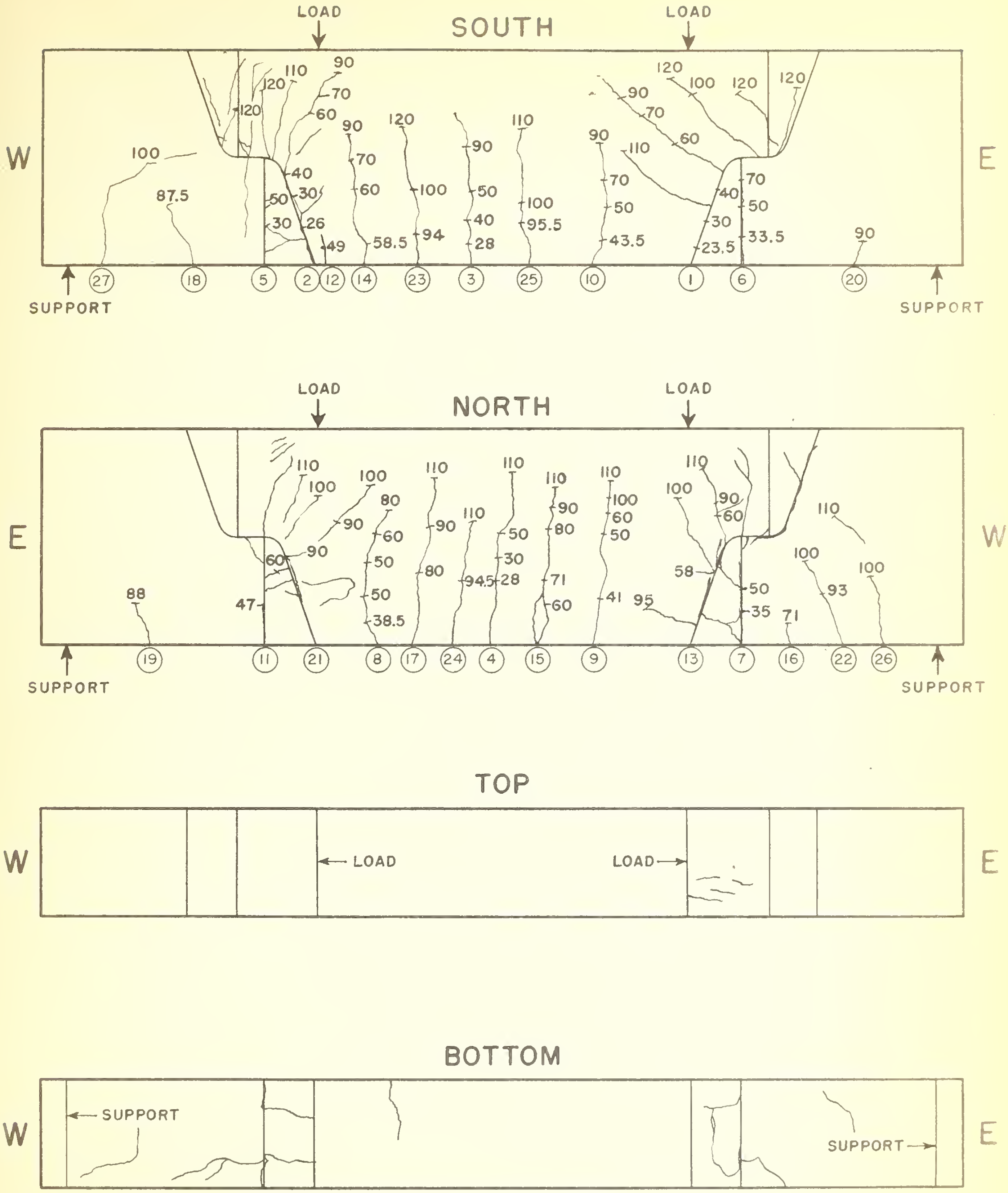

FIG. 22 CRACK PATTERNS, GIRDER NO. 3.

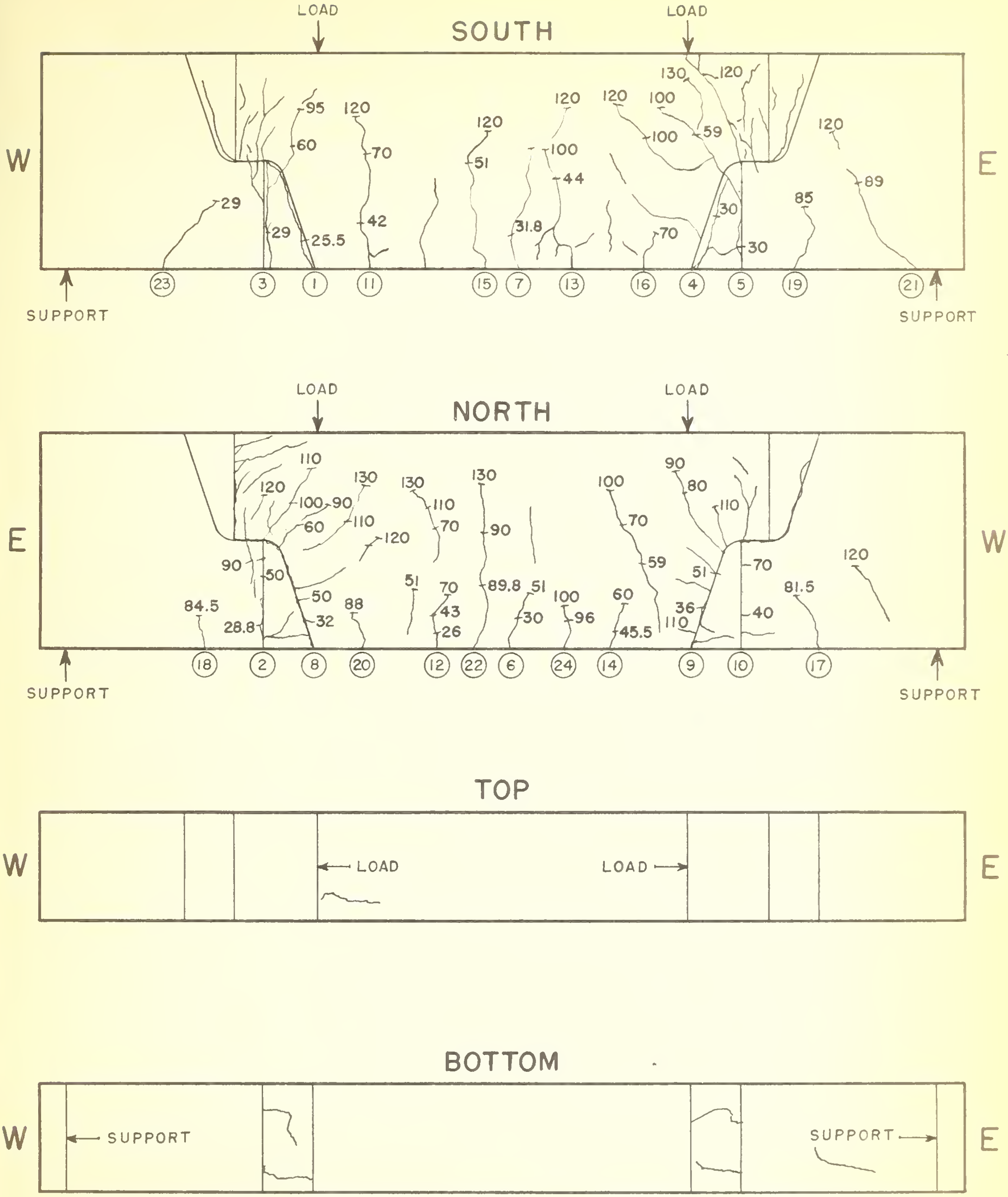

FIG. 23 CRACK PATTERNS, GIRDER NO. 4.

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