A STUDY OF HOOKED BAR ANCHORAGES IN BEAM-COLUMN JOINTS

by

JAMES O. JIRSA

and

JOSE L. G. MARQUES

REINFORCED CONCRETE RESEARCH COUNCIL

Project 33

FINAL REPORT

DEPARTMENT OF CIVIL ENGINEERING / Structures Research Laboratory
THE UNIVERSITY OF TEXAS AT AUSTIN, JULY 1972

A STUDY OF HOOKED BAR ANCHORAGES IN BEAM-COLUMN JOINTS

by

James O. Jirsa and Jose L. G. Marques

A Final Report on a Research Project Sponsored by the

Reinforced Concrete Research Council (Project 33)
Office of the Chief of Engineers, U. S. Army
(Contract Nos. DACW-73-70-C-0033, and
DACW-73-71-C-0048)
Committee of Reinforcing Bar Producers,
American Iron and Steel Institute

Tests Conducted at
Ryon Laboratory
Rice University
Houston, Texas

Report Prepared at
The University of Texas at Austin
Department of Civil Engineering
Austin, Texas

July 1972

SUMMARY OF REPORT

Nineteen specimens simulating typical beam-column joints in a structure were tested to evaluate the capacity of anchored beam reinforcement subjected to varying degrees of confinement at the joint. The types of confinement included column axial load, vertical column reinforcement, side concrete cover, and lateral reinforcement through the joint. The tests were conducted using either #7 or #11 beam bars anchored in the columns. Standard 90° or 180° hooks conforming to ACI 318-71 specifications were used throughout. The column cross section was either 12 x 15 or 12 x 12 in. The axial load varied from 135 to 540 kips. The influence of three types of lateral confinement was studied; longitudinal column bars; column ties through the joint; and concrete cover.

Stress-slip measurements were made at all stages of loading to failure. Failure was sudden and complete in all tests and resulted in the entire side cover of the column spalling away to the level of the hooked anchorage. Based on slip and strain measurements and on observations of failure, the following conclusions were reached.

(1) The level of column axial load did not significantly influence the behavior of the hooked bar anchorages.

(2) The embedment length between the beginning of a standard hook and the critical section at the face of the column was extremely important in determining the capacity of the hooked bar anchorages.

(3) Placement of column bars inside or outside the anchored beam bars did not influence stress or slip characteristics of the anchored bars.

(4) Ties through the joint reduced slip and increased capacity but only if the tie spacing was small relative to the diameter of the bend of the anchored beam bar.

(5) Concrete cover did not appear to influence the stress or slip characteristics provided that the cover was sufficient to prevent

a local failure in the vicinity of the bent portion of the hooked anchorage.

(6) There was very little difference between the capacity of 180° and 90° hooks; however, the slip at a given stress was greater for 180° hooks.

Based on observations of the test specimens, current ACI 318-71 Code specifications were studied and design recommendations for hooked bar anchorages were proposed. Comparison of the proposed design recommendations with the measured results indicated that higher stresses could be allowed for hooked anchorages provided that confinement in the form of cover or ties and straight length embedment before the hook were sufficient.

TABLE OF CONTENTS

Chapter | Page

A STUDY OF HOOKED BAR ANCHORAGES IN BEAM-COLUMN JOINTS

1. INTRODUCTION

1.1 Object and Scope

The object of this study was to examine some of the factors influencing the anchorage capacities of reinforcing bars in beam-column joints of reinforced concrete structures. The study was divided into two phases. The first phase consisted of tests of short bent bars anchored in small concrete blocks. A total of 80 specimens was tested to determine the influence of geometric configuration on bent bar behavior. This phase of the study has been reported in detail previously.[1] The second phase of the study is reported herein. In the second phase, specimens simulating typical beam-column joints in a structure were tested to evaluate the capacity of anchored beam reinforcement subjected to varying degrees of confinement at the joint. The types of confinement considered included column axial load, vertical column reinforcement, side concrete cover, and lateral reinforcement through the joint. A total of 19 specimens were tested and the response of the anchored bars as well as the ramifications of their results on design provisions for hooked bars will be discussed in detail.

1.2 Definition of the Problem

In reinforced concrete frames the beam moment at the face of a column must be developed by bond on some length of embedded reinforcing bar. In interior beam-column joints the reinforcement can be continued through to the beam on the opposite face and anchorage is generally not required through hooks or other devices. However, at exterior beam-column joints the column is generally not wide enough to permit the use of a straight bar embedded in the joint. To develop sufficient anchorage capacity, the bar must be bent. The anchored bar then includes either a 180° or 90° bend with the straight portion extending either up or down into the column. The anchorage capacity of the bar may be a function of a number of parameters, including the geometry of the bend, the axial load on the column, the amount of cover over the anchored bar, the restraint or confinement offered by column reinforcement, that is, longitudinal bars and ties, and the type of loading to which the bar will be subjected.

The lack of information regarding anchorage capacities of reinforcing bars in structural joints has become increasingly clear with the need for reinforced concrete structures designed to absorb energy and sustain large rotations in the members and joints. Typical of these requirements are structures subjected to blast or seismic loadings. A great deal of work has been done on the behavior of structural members, but the joints between members have not received much attention.

The problem is complicated by certain constructional limitations. For instance, where many bars converge increasing the anchorage length of the bars to increase the forces which the bars may resist may produce congestion of reinforcement at the joint, which makes fabrication difficult, if not impossible. In some cases the length of the hooked bar is controlled not only by the size of the column but also by the depth of the beam. The tail portion of the hooked bar will generally not extend below the soffit of the beam to permit placing beam reinforcement after the columns are cast. Ties in the joint may increase anchorage capacities but at the same time add to the congestion in the joint with the attendant problems of fabrication and concrete placement. To alleviate some of these difficulties and to permit more exact analyses and design of joints, the factors which affect anchorage capacity of bent reinforcing bars must be studied.

1.3 Previous Studies

There is very little information available concerning anchorage capacities of hooked or bent bars which can be used as a background for the present study. In most cases bond stresses on straight bars were evaluated and bent bar anchorages were of secondary importance. The studies included in the first phase of this investigation[1] provide some definitive information regarding the parameters which influence the anchorage capacity of bent bars. In the first phase, the slip between reinforcing bar and concrete was determined at a number of load stages prior to yielding of the bar. The slips were measured at three points--at the lead end, the tail end, and at an intermediate point. The influence of geometric parameters on the anchorage behavior, including initial stiffness, slip at a given stress, and ultimate strength were discussed. The variables included the bar diameters, #5, #7, and #9; the bond length to bar diameter ratios, ranging from 2.4 to 9.6; and bend angles ranging from 0 to 180°, with inside radius of bend to bar diameter ratio varying from 1.6 to 4.6.

Based on these tests the following conclusions were reached:

(1) For equal bond length to bar diameter ratios, bent bar anchorages slip more at a given bar stress than straight bars.

(2) For hooks of equal bond length to bar diameter ratio, hooks with larger angles of bend tend to slip more at a given bar stress.

(3) For hooks of equal bond length to bar diameter ratio, hooks with smaller inside radius to bar diameter tend to slip more at a given bar stress.

(4) Where a bent bar anchorage consists of straight and curved sections, most of the slip is developed in the curved section.

(5) Hooks have little effect on ultimate strength except for very short bond lengths.

A brief summary of previous studies on anchorage of hooked bars is included in Ref. 1 and will not be repeated herein.

1.4 Acknowledgments

This report has been prepared as part of an investigation conducted in Ryon Laboratory at Rice University and at The University of Texas at Austin. The laboratory testing was carried out at Rice University and this report prepared at The University of Texas at Austin. The project was supported by the Reinforced Concrete Research Council (Project 33); the Office of the Chief of Engineers, U. S. Army (Contract Nos. DACW-73-70-C-0033 and

DACW-73-71-C-0048; and the Committee of Reinforcing Bar Producers, American Iron and Steel Institute. Cadweld splices used in the tests were donated by Erico Products, Inc.

The program of investigation has been guided by Task Committee 33 of the Reinforced Concrete Research Council, composed of the following persons:

D. J. Caldera, Chairman
Tippetts-Abbott-McCarthy-Stratton, Engineers and Consultants
New York, N. Y.
N. W. Hanson
Portland Cement Association, Skokie, Illinois
W. C. Buchholz
Headquarters, U. S. Air Force, Washington, D.C.
F. R. Kahn
Skidmore-Owings-Merrill, Chicago, Illinois
C. F. Corns
Office of the Chief of Engineers, U. S. Army, Washington, D.C.
H. E. Nelson
City of Chicago, Chicago, Illinois
P. F. Rice
Concrete Reinforcing Steel Institute, Chicago, Illinois

The project has been under the direction of Dr. J. O. Jirsa, Associate Professor of Civil Engineering. A portion of the project formed a Master's thesis submitted by Mr. J. L. Marques to Rice University.[2] The report has not been reviewed by the Advisory Task Committee.

Special acknowledgment is due Mr. William Black of Bethlehem Steel Corporation, who was instrumental in selecting representative hooked bar anchorages in beam-column joints for the test program.

2. EXPERIMENTAL PROGRAM

2.1 Introduction

In the first phase of this investigation,[1] the specimens consisted of relatively short bars embedded in small concrete blocks. The anchored lengths were short to ensure that bond failure would occur before the steel yielded. In addition, the side cover on the bars was sufficient to prevent fracturing the concrete block so that the bars could be considered as anchored in mass concrete. To evaluate the anchorage capacity of the hooked bars under more realistic conditions, the following criteria had to be satisfied.

(1) Specimens had to be full-scale models of beam-column joints in order to eliminate scale effects and to permit the use of fairly large diameter bars.

(2) The specimens had to duplicate the constraints and loading which would be expected on the region of the joint in which the bar was anchored.

(3) The hooked bars had to conform to current ACI standards for hook geometry.

2.2 Test Specimens

2.2.1 General Considerations. A diagram of the specimen selected to simulate a typical exterior beam-column joint is shown in Fig. 1. The column cross section was either 12 x 15 in. or 12 x 12 in. The assumed beam was 12 in. wide and 20 in. deep. The dimensions of the beam and the column were chosen so that the specimen would be a realistic full-scale simulation of a beam-column joint. In addition, it was desirable to select a specimen that was small enough to be easily handled and inexpensive to fabricate. The length of the column in all the tests was 50 in. This length was selected after taking into account the need for a small specimen and, in addition, requirement that the column be long enough to permit the embedment of the hooked bars which in some cases extended into the column past the assumed beam. In addition, it was necessary to extend the column above and below the beam to eliminate lateral constraints at the joint region produced by the axial loading heads.

To facilitate fabrication the beams were not cast with the columns. The anchored bars extended past the face of the column and were connnected to threaded rods which were loaded with hydraulic rams. The compression zone of the beam was duplicated with a steel plate bearing against the face of the column over an area which approximated that of the compression zone of the assumed beam.

In all of the 12 x 15 in. columns the column reinforcement consisted of six #8 longitudinal bars and #3 closed ties at 5 in. outside the joint. The 12 x 12 in. columns were reinforced with four #8 longitudinal column bars and #3 ties at 5 in. outside the joint. The clear cover over the ties was 1-1/2 in. In some of the tests the column bars were located inside the beam bars in order to provide an assessment of the effect of confinement that the column bars provided and in other tests ties were continued through the joint to provide information regarding the confining influence of lateral ties.

2.2.2 Variables. Table 1 summarizes the properties of the 19 specimens tested in this study. Figures 2 and 3 show plan and side views of the columns at the point where the beam bars are anchored in the joint. The variables considered and the range of those variables are discussed below.

(a) Size of Anchored Bars. The tests were conducted with either #7 or #11 beam bars anchored in the columns. For a 12 x 12 in. or a 12 x 15 in. column, the #7 bar represents a fairly realistic anchorage situation. However, to determine the limits of anchorage strength, it was felt that increasing the beam reinforcement to #11 bars would give an indication of the anchorage capacities of a fairly small joint. From a practical standpoint, bars larger than #11 would rarely be used in frame members. The details of the specimens in the J7 test series (reinforced with #7 beam bars) are shown in Fig. 2 and the details of the J11 series are shown in Fig. 3.

(b) Hook Geometry. All the tests were conducted using either 90° or 180° hooks conforming to ACI 318-71[3] specifications (Sec. 7.1). For these hooks the inside diameter of the bend was 6 bar diameters for the #7 bars and 8 bar diameters for the #11 bars. For the 90° hook the extension beyond the bend, which will be referred to as the tail extension, was 12 bar diameters and for a 180° bend the tail portion was 4 bar diameters.

(c) Lead Embedment Length. By varying the size of the column, the lead embedment before the hook portion of the anchored bar was varied. For the #7 bar anchored in a 12 x 15 in. column, the lead embedment was 9.5 in. For a 12 in. column this was reduced to 6.5 in. For the #11 bars anchored in a 15 in. column, the lead embedment length was 6 in., and for a 12 in. column, 3 in. It should be noted that it was impossible to test a #11, 180° hook in a 12 x 12 in. column, because the tail portion was too long to permit placement within the column cross section.

(d) Confinement. Three types of confinement were considered. First, the influence of the longitudinal column bars; second, the influence of column ties through the joint; and third, the influence of concrete cover. To determine the influence of the column bars, tests were run with column bars placed outside the anchored beam bars as would normally be done in practice. Comparison tests were run with the column bars placed inside the beam bars. In both cases the concrete cover over the anchored beam bars was 2-7/8 in. To determine the influence of column ties through the joint, the effect of the ties was isolated by retaining the same column steel, placing the column bars inside the beam bars, and carrying ties through the joint. In this case the confinement consisted of 2-7/8 in. concrete cover plus #3 ties at either 5 in. or 2-1/2 in. spacing through the joint. Finally, the effect of concrete cover was determined by conducting one test in the J7 series with concrete cover reduced from 2-7/8 to 1-1/2 in. and placing the column bars inside the beam bars so only clear concrete cover confined the anchored beam bars.

(e) Column Axial Load. It has been felt for some time that column axial loads would have a beneficial effect on anchorage strength of bars in joints. Some limited tests[5] indicate that normal pressure reduces the tendency for splitting the cover on an anchored bar. To determine the influence of column loads, tests were run holding all specimen dimensions and reinforcement constant and varying the level of axial load. Initially, axial loads of 270 and 550 kips were imposed on the 12 x 15 in. columns of the J7 series. The results indicated that this difference in axial load had very little influence on the hooked bar behavior and additional tests were run with column axial loads reduced to a nominal 135 kips. On a 12 x 15 in. column, a 550k column load produced a stress of about 2500 psi in the concrete and 20 ksi in the steel. To retain the same stress levels in the 12 x 12 in. column, a load of 420 kips was placed on the smaller column cross section.

For recognition of the variables in each specimen the following notation was used. For example, specimen J7-90-15-1-L: J signifies a joint test, #7 longitudinal beam reinforcement, 90^{o} the angle of bend, 15 in. column thickness, 1 indicates the type of confinement and L refers to the level of column axial load. The lateral confinement designations are indicated in Table 1 and are as follows: Numeral 1 refers to confinement consisting of column bars plus 2-7/8 in. concrete cover; 2 refers to only 2-7/8 in. cover, that is, column bars inside anchored beam bars; 3 refers to 2-7/8 in. cover, plus #3 ties at 5 in. through the joint with column bars inside anchored beam bars; 3a refers to 2-7/8 in. cover plus #3 ties at 2.5 in. through the joint with column bars inside the anchored beam bars, and 4 refers to a cover of only 1-1/2 in. with column bars inside anchored beam bars. The designation for axial load refers to the three levels, H for 50 kips, M for 270 kips, and L for 135 kips. It should be noted that these are nominal levels of axial load and the actual levels of load measured during testing are listed in Table 1.

The geometry of the test specimens was chosen so that a systematic evaluation of the variables affecting the anchorage capacity could be carried out. Specimen J7-90-15-1-H was selected as a standard specimen. The details of this specimen could be considered typical of practice in much of the United States. The beam bars are anchored inside the column bars, ties extend to the top and bottom of the beam, and the anchored bar consists of a 90^{o} hook with a tail extension conforming to ACI standards.[3] The remainder of the tests of the J7 series provides a means of comparing the effect of the other variables with this "standard" test. In the J11 series, specimen J11-90-15-1-H is the standard test.

Using the specifications of ACI 318-71[3] to determine the strength of the anchored bars at the face of the column indicates that for both the #7 and #11 bars the anchorage is inadequate to develop yield at the face of the column. Therefore, to use these hooks in a design of a structure would mean that the stress at the face of the column would have to be reduced below yield stress or the joint dimensions for anchored bars would have to be changed to allow yield stresses to be developed at the face of the column. Further discussion of the computed versus measured strengths is contained in Chapter 4.

2.2.3 Material Properties. The bars used throughout the study had the same deformation pattern and are shown in Fig. 4. The anchored bars, the longitudinal bars, and the ties were Grade 60. Typical stress-strain diagrams for the #7 and #11 beam bars are shown in Fig. 4.

One basic mix of concrete was used throughout the study. The mix proportions were selected to give an average compressive strength f'_c of 4500 psi at 14 to 21 days. All the specimens and control cylinders were cured at room temperatures. Each specimen required casting two batches of concrete. The first batch completed the column to a level just below the horizontal portion of the anchored beam bars. The concrete strengths for both batches are listed in Table 1. In general, the concrete strengths ranged from about 4.4 to 4.8 ksi. Several specimens varied from the nominal 4500 psi by values slightly greater than ±10 percent.

2.3 Loading Systems

A load frame shown in Fig. 5a was constructed to apply axial loads to the column, and tensile and compressive beam forces to the joint. The resulting forces on the specimen are shown in Fig. 5b.

To load the column four 100 ton center-hole rams (A) were placed below the bottom support platform and 2-1/2 in. alloy steel rods (B) were passed through the rams and bottom platform along the side faces of the column to the top head. The alloy steel rods were instrumented with strain gages and served as load cells to monitor the axial load. The top and bottom platforms were fabricated using a series of channels (C) to provide a rigid loading surface and distribute the load uniformly over the column section. In addition, a Hydrocal grout was used between the specimen and the loading surfaces.

The anchored beam bars protruded from the column about 12 in. Threaded rods were attached to the reinforcing bars using a Cadweld splice. Two center-hole hydraulic rams (D) were placed over the threaded rods and reacted against a column made up of two 18 in. channels. The reaction column transferred a compressive load to the specimen through a large box section (E) to simulate the compressive zone of the beam. To equalize the overturning moment on the reaction column, threaded rods connected the bottom of the reaction column to the large channels supporting the bottom loading platform. In addition, it was necessary to provide a horizontal reaction (F) at the top of the test column to balance the moment imposed by the simulated beam. With this system the compressive beam force was slightly greater than the tensile beam force. However, this was not considered to be detrimental to the behavior of the test specimen and was felt to be warranted considering the resulting simplicity of both the test specimen and the loading system.

For each test the column axial load was applied and maintained constant throughout the loading sequence. Loads were applied to the anchored beam bars using a hand pump, and stresses were increased in increments to failure.

2.4 Instrumentation

2.4.1 Slip Measurement. In order to plot the lead stress-slip curves for the anchored bars (slip of the bar relative to the concrete) the procedure developed by Minor[1] in the basic hook tests was used in the joint tests.

A 0.059 in. piano wire was attached to the bar by making a short 90° bend at the end of the wire and inserting it into a drilled hole of equal diameter at the points where slip was to be measured. Figure 6 shows the slip measurement points. In all tests slip was measured at five points along the length of the anchored bar. The wire was long enough to reach outside the form for the back surface of the specimen. At the part where the wire was connected to the bar, the wire was oriented in the direction displacement was to be measured. The points where slip was measured and the expected directions of movement are shown in Fig. 6.

After the wire was placed on the bar a neoprene tube was placed over the entire length of the piano wire to prevent bond. A small amount of permagum sealer was placed at the connection between the wire and the anchored bar to seal the tube and to allow movement of the wire in the direction it was expected to travel. It was extremely important to use this sealer to prevent interlock between the protruding wire and the concrete. The amount of permagum was small, roughly 1/8 sq. in. of bar surface area was covered; therefore, the reduction of bond could be considered negligible. Figure 7a shows the wires and neoprene tubes in place on a specimen in the form prior to casting.

To reduce the wobble of the wire within the tube, the wire was placed in tension using a spring between the concrete surface and a small brass plug fastened to the wire with a set-screw. The installation is shown in Fig. 7b. A dial gage was used to measure movement of the wire and was attached to the back face of the column using a wood block. The wood block was glued to the surface with Eastman 910 Adhesive. A dial gage rested against the brass plug attached to the end of the wire. There was considerable concern in the early tests that the back surface of the specimen would not be a suitable reference point for measuring slips. However, after several tests had been completed, the back surface was uncracked until failure was imminent. Therefore, it was concluded that the surface would serve as a better reference point than an externally supported reference.

2.4.2 Bar Stresses. To measure the stresses in the anchored reinforcing bars, three strain gages were mounted on each bar. Two gages were mounted on the bars prior to casting and were waterproofed. These were located roughly at the point of horizontal and vertical tangency of the bent portion of the hook. The third gage was mounted on the bar protruding from the column just prior to testing. Figure 6 shows the points where strain gages were mounted. The application of strain gages provided a means of estimating the stress transferred from the bar to the concrete along various portions of the bar.

2.5 Test Procedure

Prior to placement of the specimens in the test frame, the threaded bars were attached to the anchored beam bars using Cadweld splices. The specimen was seated in the test frame with Hydrocal to reduce stress concentrations between the specimen and the support platform and top loading head. After the column was seated in the frame the box section which transferred compressive beam forces to the column was seated with Hydrocal against the column face.

The column axial load was applied to the level designated and maintained constant throughout the test. Such a column load could be considered to duplicate the dead loads in a structure which remain fairly constant with increasing moments on a beam.

The reinforcing bars of the assumed beam were loaded in increments of roughly 2000 psi, which provided about 30 load stages prior to yield. The stress increment was selected to provide sufficient points to evaluate slip and stress transfer phenomena. In the early tests several loading patterns were tried. In the first loading pattern, increments were applied at 1 minute intervals and every fourth load stage was held for 5 minutes to allow slip to stabilize. In the second loading pattern the load increments were applied continuously at 2 minute intervals. Both patterns were equivalent to a given stress level applied per 8 minute period. After evaluating the results of these tests, the 2 minute interval was used in all remaining tests because the differences between stress slip curves using both methods were negligible and the 2 minute interval allowed slightly more time for recording data at each load stage. The reason for considering different loading patterns was the indication in the basic hook tests that the duration of loading significantly influenced slip values. However, it should be noted that the basic hook tests considered only very short anchored bars. With long anchored bars the level of stress between the steel and the concrete was much lower and the stresses stabilized in a very short period of time.

Crack patterns were marked at all load stages and photographs taken at critical points. The test was terminated when one of the anchored bars pulled out of the column. In general, failure was fairly sudden and resulted in the entire side face of the column spalling.

3. THE EVALUATION OF TEST RESULTS

3.1 Introduction

The measured data for all tests are presented in the Appendix. The slip and strain measurements at the points shown in Fig. 6 are plotted against measured lead bar stress. Curves are plotted for both bars in each specimen. A general discussion of the results is included in the Appendix and the results are briefly reviewed here. The data shown in Figs. A-1 through A-12 indicate the following general trends:

(1) Most of the slip occurs over the lead straight embedment and the curved portion of the hooked bar. Very little slip was measured on the tail extensions of the hooks. This is consistent with trends noted in the basic hook tests.[1] The lead slip (point 1H) is greatest in all cases. If the lead straight embedment was small, the slip at point 2H was nearly as large as the lead slip. The slip at points 3H, 3V, and 4H or 4V was very small. In most tests no slip was measured at points 4H or 4V until failure was imminent. In many tests there was no measurable slip at the end of the tail extension throughout loading.

(2) Measured slip indicated that 180° hooks pulled toward the front face of the specimen rather than around the bend. As noted in Fig. 6, the designated positive direction of movement for point 4H on 180° hooks was toward the back surface of the specimen. The slip at points 4H for 180° hooks, Fig. A-4 was toward the front surface, which indicates the entire hook was being pulled forward rather than around the bend.

(3) The initial stiffness (stress over slip up to levels of about 30 ksi lead stress) of #11 bars was about one-third that of the #7 bars.

(4) At failure the lead slip of the #7 bars was about two to three times that of the #11 bars.

(5) Stress transfer along the straight length embedment varied from about 20 to 40 ksi for the #7 bars and was negligible for the #11 bars.

(6) In nearly all the tests the stress transferred to the tail extension was generally small, less than 20 ksi until failure was imminent. Near failure stresses on the tail increased rapidly.

A summary of measured slip behavior is listed in Table 2. Applied lead stress at lead slips of 0.005, 0.016, and 0.05 in. is listed. In addition, slip at points 1H and 2H under applied bar stresses of 0.6 of the computed strength is given. The strength of the hook was computed using the provisions of ACI 318-71.[3] In addition, slip and stress at failure are tabulated. Stress at a slip of 0.016 in. was selected because it is in the range suggested as a permissible crack width in beams.[4] If it is assumed that the crack width at the beam column joint is about equal to the slip of the anchored bar, the observed stress at 0.016 in. slip provides a measure of the serviceability of the hooked bar. In a similar manner, slips at a level of 0.6 of computed strength correspond to the suggestion made in Sec. 10.6.3 of ACI 318-71[3] for computing crack width at service loads.

The stress levels at the specified lead slips indicate that in the J7 series stresses were nearly equal for all the specimens except the two specimens with a 12 in. column dimension and the specimen with a 15 in. column and a 180° hook. For these three specimens the stresses were about 60 to 70 percent of the measured stresses in the other specimens of the J7 series. In general, stresses at 0.005 in. slip were between 30 and 40 ksi, at 0.016 in. slip between 50 and 60 ksi, and at 0.05 in. slip between 70 and 80 ksi. In the J11 series the stresses were comparable for all tests except J11-180-15-1-H, which was lower at all three specified levels of slip. For these tests the stresses at 0.005 in. slip were around 20 ksi, at 0.016 in. slip stresses were between 25 and 35 ksi, and at 0.05 in. slip stresses were around 50 ksi. A comparison of the stress-slip curves shows that the stiffness (stress/slip) for the #7 bars is about three times that of the #11 bars. In addition, stresses at failure for the #7 bars were nearly double those measured for the #11 bars. The lead slip at failure for the #7 bars ranged from 0.05 to 0.22 in., with most specimens failing at slips greater than 0.14 in. However, the #11 bars failed at lower slip values ranging from 0.04 to 0.09 in.

To evaluate the influence of the individual parameters considered in this study, the stress and slip data for comparable tests will be discussed in detail in the following section. In each case average values for the two bars of a specimen are compared. Slip data plotted are for the lead slip only.

3.2 Influence of Column Axial Load

Figure 8 shows curves for five specimens, three with #7 and two with #11 bars with the same lateral confinement, column bars outside the beam bars and 2-7/8 in. concrete cover. Only the level of column load was varied from about 145 to 545 kips. In both series the stress-slip curves are virtually identical. Figure 9 shows comparison specimens with lateral confinement consisting of only 2-7/8 in. cover, with column bars inside the beam bars and with varying column loads. Again, the stress-slip curves are nearly the same.

Figures 10 and 11 show the stresses before and after the bend plotted against lead bar stress. In the J7 series (Fig. 10) the stresses are nearly equal for both lateral confinement situations, with only the stresses for the specimen with the column load of 269 kips varying from the other tests. In the J11 series (Fig. 11) the curves are identical for the specimens compared.

Based on the stress and slip measurements for the tests in which axial loads were varied, the influence of column axial loads appears to be negligible. It should be noted that in all cases the tail of the hook was oriented in the direction of the axial load. Other orientations of bent bars and different lateral confinement might produce different results.

3.3 Influence of Bend Angle

Stress-slip curves are shown in Fig. 12 for tests in which axial load and lateral confinement remain constant, but the bend geometry varied. Curves are shown for 90° and 180° hooks in 12 x 15 in. and 12 x 12 in. columns. The curves confirm the trend observed previously[1] that 90° hooks tend to be stiffer (slip less at a given stress) than 180° hooks. Only the 180° hook in a 12 x 12 in. column did not indicate this trend. The influence of bend angle on stresses is shown in Fig. 13 for the #7 bars and Fig. 14 for #11 bars. There does not appear to be any definitive trend with regard to stresses and may be an indication that there is little significant difference in the behavior of 90° and 180° hooks.

3.4 Influence of Lead Embedment

The influence of the lead embedment, that is, the straight bar embedment before the hook, is shown in Fig. 12. It is apparent that the strength and stiffness of the hook anchorages are significantly affected by the thickness of the column or the lead embedment. The #7 bars embedded in 12 x 15 in. columns reached stresses of over 90 ksi at failure. Those in a 12 x 12 in. column reached just over 60 ksi. In addition, the slip is greater at all stress levels with shorter lead embedment. The influence of lead embedment on stresses in #7 hooks is shown in Fig. 15. The stresses build up very rapidly with short lead embedment.

For the #11 bars with a 90° hook, the lead embedment to bar diameter ratio is very small and only a very small stress increment is transferred to the concrete in both the 12 x 12 and 12 x 15 in. columns. However, it is apparent that the slip is greater for the 90° hook in a 12 x 12 in. column with 3 in. lead embedment. The stresses are nearly the same for both column sizes (Fig. 14) and in the case of a 12 x 12 in. column, the stress after the hook increases more rapidly. The difference in lead embedment for large bars provides only limited additional length for stress transfer to the concrete before the hook, but more importantly, it provides increased concrete area and improves the lateral restraint against splitting as the bar is loaded.

3.5 Influence of Lateral Confinement

Because of the systematic variation of specimen geometry, the confinement provided by the column bars, the side concrete cover, and ties to the joint can be evaluated. Figure 16 shows stress-slip curves for the #7 bars with 90° hooks anchored in 12 x 15 in. columns and a nominal axial load of 540 kips. The curves lie in a very narrow band and indicate that there was little difference in slip behavior for the four specimens. There is an indication that the column bars outside beam bars and ties through the joints slightly increase stresses at a given slip (specimens with ties had column bars located inside the beam bar with lateral confinement consisting of 2-7/8 in. cover and #3 ties at 2-1/2 or 5 in. spacing). However, it does not appear that either the location of the column bars or ties in the joint substantially influenced stress-slip characteristics. It should be noted that the stresses at a given slip for the anchorages with ties at 2-1/2 in. through the joint would probably have been different if the concrete strength had not been low for this test. The value of f'_c for J7-90-15-3-H was about 4.65 ksi, while f'_c was about 3.7 ksi for specimen J7-90-15-3a-H.

Reduction of concrete cover from 2-7/8 in. to 1-1/2 in. did not change the shape of the curve but did reduce drastically the stress and slip at failure. Failure stress was about 75 percent and slip was about 1/3 of that in the other three tests. The stress measurements for the #7 bars shownin Fig. 18 confirm the trends indicated by the slip measurements.

Figure 17 shows the effect of lateral confinement on the response of the #11 bars. It is apparent that the location of the column bars had very little influence. However, ties through the joints significantly improved the strength. The inclusion of ties enabled the stress at failure to reach yield in both cases, whereas the other specimens failed at stresses of about 50 ksi. The stress measurements for the #11 bars (Fig. 18) indicate the same trends. Ties through the joint reduced the stresses both before and after the hook at a given lead stress. The test results indicate that both the strength and deformation capacity of the #11 anchored bars was improved with the inclusion of ties in the joint.

It appears that ties are especially beneficial in the case of large anchored bars. Cover does not appear to change the stress-slip characteristics of the anchorage materially, but a reduction in cover reduces both the strength and deformation capacity. It appears that the location of the column bars offers little, if any, lateral restraint, because the lateral stiffness of #8 bars unsupported by ties over a 2-ft. length is quite low. It is likely that a combination of ties through the joint and column bars outside the anchored bars would have improved the response further, because the ties would provide support for the column bars. However, no tests were run using this combination of variables.

3.6 Stress-Slip Curve for Hooks

By plotting the stress before the start of the hook (at the end of the straight bar lead embedment) against the slip at the start of the hook (point 2H), the stress-slip characteristics for the standard ACI hook anchorages considered in this study can be estimated. Figures 19 and 20 show stress-slip curves for the 19 specimens tested. In general, these curves duplicate the trends observed previously, but it should be noted that the stress measurements before and after the bent portion of the bar were influenced by bending in the bars and cannot be considered accurate over the whole range of loading.

In the J7 series (Figure 19) it is apparent that hook performance was poor in the 12 in. as compared with the 15 in. columns. Slip at similar stress levels was nearly doubled for bars in the 12 in. column. Ties through the joint appear to improve slip behavior. The large slip measured in J7-90-15-3a-H is obviously a reflection of the lower concrete strength of that specimen, particularly when compared with the slip behavior measured for specimen J7-90-15-3-H.

The performance of the #11 hooked bars was similar to that of the #7 bars. The most obvious result is the marked reduction in slip at given stress levels for the specimens with closely spaced ties through the joints.

3.7 Mode of Failure

In nearly all the tests the sequence of cracking and subsequent failure followed a similar pattern. First cracking occurred on the front face of the column with cracks radiating outward from the bars. Vertical cracks developed most rapidly and eventually terminated in the compression zone of the beam and near the top of the specimen. In most tests a vertical crack appeared on the side faces of the specimen at about the column longitudinal bars located nearest the front face. Figure 21 shows typical crack patterns prior to failure.

As stress and slip progressed, cracks appeared on the two side faces in the vicinity of the bent portion of the anchored bar. When a load increment was applied at high stress levels, slip continued to increase and stresses tended to reduce slightly. In some cases failure occurred while a load was being held or was dropping.

Failure was always sudden and complete, with the entire side cover spalling away to the level of the anchored bars and the load dropping immediately to a fraction of its previous level. A series of photographs is shown in Figs. 22 and 23 to provide an idea of the type of failure observed. There was very little difference between failure patterns in specimens with column bars located inside or outside the anchored bars. In those cases where the column bars were outside the beam bars, the column bars were bent outward after failure. Ties through the joint had the same appearance. This indicates that the compressive forces developed under the hook are very large and force the concrete outward. It is also possible that the hook pulling forward forms a wedge which forces the concrete cover off and deforms the steel bars confining the joint core. In most tests a very large area of the side cover was destroyed, as can be seen in the photographs. The smallest area of side cover removed at failure was in specimen J7-90-15-4-H, which had the smallest cover (1-1/2 in.) on the anchored bars.

Close examination of the failed specimens showed crushing at the inner surface of the bend and a gap at the back surface; see Fig. 24. This behavior is consistent with the failures observed in the basic hook tests. That is, bent portions tend to straighten pulling the bar toward the center of the bend and reducing the arc distance between the horizontal and vertical bar segments of the anchorage. This produces intense lateral compressive stresses at the bend, which in effect "punch out" the side cover at the bend and force the entire side to spall away. In some of the photographs in Fig. 24, concrete can be seen adhering to the inside surface of the bend. A similar condition was observed in the basic hook tests.[1] Specimen J11-90-15-3a-L did not fail by complete loss of side cover, but loading was stopped because flexural cracks across the column became very large and column distortions excessive. The beam forces in this specimen were the largest of any tested.

Prior to testing it was felt that shear would be a significant factor in influencing failure in the specimen. However, shear cracks were observed only in the specimens of the J11 series with low column axial loads and in none of the tests was shear considered a contributing factor in producing failure. The significance of the very small degree of shear distress will be discussed in Chapter 4.

3.8 A Failure Hypothesis for Hooked Bars in Beam-Column Joints

Considering the measured data and observed modes of failure, a reasonable estimate of the failure phenomena of a hooked bar can be made. It is apparent that the failure of a hooked bar is governed not by pulling out, but rather by loss of cover. In nearly all cases the side cover spalled away completely at the moment of failure with a sudden and nearly complete loss of load carrying capacity. Slip between the bar and the concrete produced splitting of the side cover at the lead end of the anchorage, which gradually progressed backward. As indicated by some of the stress measurements at the start of the hook, the effect of the lead slip was to destroy the stress transfer capacity along the straight lead embedment (especially for short straight lead embedments) and this portion of the hooked bar anchorage was not transferring any stress to the concrete as failure approached.

The very large compressive stresses at the inside surface of the bend produced a stress condition which also tended to split the cover. As slip progressed the hook pulled forward and, near failure, acted much as a wedge forcing the concrete cover to spall. This action produced an outward deformation of the unsupported longitudinal column bars and increased the instability of the bar under axial loads.

Once the sequence of events leading to failure is understood, the influence of various parameters on the strength and slip characteristics of the hooked bars also can be more clearly established. While column axial loads would appear beneficial, in reality they produce lateral strains which act in the same direction as the strains produced by the hooked bars. Therefore, the axial load does not significantly change the performance because it does not offer any restraint to the tendency of the side cover towards splitting, and may actually reduce strength by causing lateral strains in the same direction.

Column bars unrestrained by ties through the joint likewise offer little lateral support. Such bars are too flexible and are not located near enough to the region of most intense distress, that is, the inside of the hook. As the column thickness was increased or ties carried through the joint, some improvement in stress and slip characteristics was noted. The improved performance of hooks in 12 x 15 in. columns can be attributed primarily to the added straight lead embedment, but in the case of the #11 bars this is not significant. For large bar diameters, the increased column thickness provides more concrete area and the total tensile force developed before the side cover spalls is obviously larger. Ties through the joint are most beneficial if the spacing is about equal to or less than the radius of the hook, as indicated by the test results. No. 3 ties at 2-1/2 in. did not substantially improve the performance of the #7 hooked bar and #3 ties at 5 in. only slightly improved the performance of a #11 bar, but when the spacing was reduced to 2-1/2 in. the performance was substantially improved. In the case of the #7 hooked bars, the lead stresses at failure were very high in all cases and more definitive results might have been obtained with a shorter straight lead embedment. It would seem that smaller tie diameters with closer spacing are more beneficial than larger ties with greater spacing which provide the same percentage of confining reinforcement. In addition, it would appear that only those bars located very near the bent portion of the anchorage are effective in providing lateral restraint to side splitting.

The thickness of concrete cover does not seem to be too important as long as a local failure at the inside of the bend does not occur. Such a failure could be likened to punching out a circular area of concrete roughly defined by the diameter of the hook. In no case did such a failure occur, indicating that for #7 bars 1-1/2 in. of cover is barely sufficient, and for #11 bars a 2-7/8 in. cover is satisfactory. No lower bound can be determined on the basis of these very limited test results. In general it would appear that if anchored bars are placed inside column bars and a 1-1/2 in. clear cover is maintained on the ties, side cover on the anchored bars will be sufficient to prevent a very localized failure of the side cover.

4. IMPLICATIONS OF TEST RESULTS ON DESIGN PROCEDURES FOR HOOKED BARS

4.1 Introduction

The design provisions for hooked bar anchorages in ACI 318-71[3] are based primarily on provisions appearing in previous codes, the origin of which is not well-documented. Recent pullout tests of hooked bars embedded in massive concrete slabs[6] provide some additional information, but very little data are available regarding hooked bar strength in typical structural applications. Therefore, the tests conducted in this study offer an opportunity to evaluate present design recommendations and to suggest changes. In the subsequent discussions the ACI Building Code ACI 318-71[3] and the Commentary to the Code[4] will be referred to as ACI Code and Commentary.

4.2 Measured and Computed Strength

Using the provisions of ACI Code, Sec. 12.8, the strength of the bars in the test specimens was evaluated. The computed strength was determined in the following manner.

First, the stress developed by the standard hook was computed using values listed in Table 12.8.1 of the Code. The hooked bar stress is given as $f_h = \xi\sqrt{f'_c}$ where ξ is 360 for top cast #7 or #11 bars and 540 for other #7 bars, 420 for other #11 bars. Using the measured concrete strengths, f_h values are listed in Table 3. Since a hook generally extends vertically through the concrete, it is not clear whether "top cast" or "other" ξ coefficients should be used.

Second, the stress developed over the straight lead embedment f_ℓ was computed using the basic equation for development length (Sec. 12.5) and solving for f_ℓ in terms of a known anchorage length.

$$f_\ell = \frac{\ell_\ell\sqrt{f'_c}}{0.04A_b}$$

where ℓ_ℓ is the straight lead embedment and A_b is the area of the bar. No modification was made in f_ℓ for top cast or other bars. Values for f_ℓ are also listed in Table 3. Measured stresses at the start of the hook and at the lead end at failure are listed. Because of bending in the bars as slip progressed, stresses at the start of the hook can only be considered as estimates of stress.

Comparing measured and computed values of f_h, the measured stress is about 70 to 90 percent greater than computed for the #7 bars and at least 50 percent greater than computed for the #11 bars. These results indicate that the provisions of Sec. 12.8 are quite conservative when applied to hooked bars in beam-column joints such as those tested. There does not appear to be justification to limit f_h to values less than or equal to $f_y/2$ as noted in the Commentary for Sec. 12.8.

A ratio of measured to computed stress at failure at the lead end of the anchorage varies from about 1.2 to 1.9. In no case was the stress underestimated. The values of lead bar stress indicate that the stresses computed for the lead straight bar length f_ℓ tend to be unconservative because the ratio of measured to computed lead bar stress $(f_h + f_\ell)$ is less than the ratio of measured to computed hook stress f_h. Since the equation for development length was not intended for bar lengths shorter than those required to develop yield stress, it is unrealistic to compute the anchorage strength on that basis. Previous investigations have shown that where long straight embedments are tested, the worst stress transfer condition is near the loaded end of the bar. This same phenomenon has been observed in the hook tests of this study. The very short straight embedment is not efficient in transferring stress from the bar to the concrete.

Measured results show that for the #7 bars a straight length embedment of 6.5 in. and for the #11 bars a 6 in. lead embedment did not improve the strength. For the #7 bars a 9-1/2 in. lead embedment increased stresses at failure but the hook stress at failure was at or exceeding yield in all cases. Therefore, the importance of short straight lead embedment should probably be minimized in making design calculations. The comparison of computed and measured results indicates that on the basis of strength alone no distinction can be made between 90° and 180° standard hooks, although on the basis of stiffness, that is, stress over slip, a 90^{o} hook is preferable.

4.3 Strength of Hooks in Mass Concrete

Research results reported by Hribar and Vasko[6] indicate that standard hooks in mass concrete develop stresses well in excess of yield. Figure 25 shows stress-slip characteristics of standard #7 and #11 hooks in mass concrete (extracted from Ref. 6) compared with the stress-slip characteristics of hooked anchorages tested in this study. As expected, the initial stiffness of the hooks embedded in mass concrete was larger than in a typical beam-column joint. The #11 bars show a reduced stiffness throughout the load range when anchored in a beam-column joint. However, the #7 bars anchored in joints did not exhibit a marked change in slope until a slip of about 0.02 in. was measured while the stiffness of #7 bars in mass concrete was reduced at a slip of less than 0.01 in. The improved behavior of the #7 bars in the joints cannot be easily explained. Although the number of tests run is insufficient to establish reasons for the stress-slip characteristics observed, one possible explanation is the proximity of the tail section of the hook to the compressed longitudinal column bars. The tail of the #7 bar was anchored immediately adjacent to a #8 column bar in compression and stress may have been transferred directly to the column bar with the tail section performing more efficiently as a result. In the case of the #11 bar, the tail section was also anchored next to the column bar; however, the #11 bar was so much larger than the #8 column bar that stress transferred to the column bar was not as significant.

4.4 Development Length for Reinforcement of Beam-Column Joints

Section 12.3 of the Code requires that the embedment length of tension reinforcement from a critical section satisfy the requirements for the development length (see Fig. 12.4 of the Commentary). Implications are that the total length of embedment from the critical section, including the hook, may be considered as development length. However, the standard hook and its tail extension, if any, cannot be counted at a stress greater than that given by Sec. 12.8 of the Code unless bend radii larger than the minimum specified in Sec. 7.12 are used. However, no guidance is given as to the amount of increase in stress to be allowed if the radius is increased. As it stands, the statement in the Commentary is too ambiguous to be of value in design computations.

Figure 12.4 of the Commentary may be particularly misleading in view of the recommendations contained in Sec. A.5.5 dealing with seismic design of ductile frames. In view of the test results in this study, tail extension beyond those of the standard hook cannot be considered as adequate for providing development length. In all cases, failure was produced by side splitting of the joint and not by pullout of the hooked bar. Therefore, tail extensions would not be effective in increasing anchorage strength. Tail extensions beyond those required in a standard hook may provide a margin against bond deterioration under cyclic loading, such as would be expected in earthquakes. But even for this purpose, extensions would seem to be of dubious value.

4.5 Shear Strength of Beam Column Joints

Although the study was not intended to provide information regarding the shear strength of the core region of beam-column joints, the test results provide information which is not currently available elsewhere regarding shear in joints. Based on measured bar forces the shear forces in the core were computed (see Fig. 26) and are listed in Table 4. The nominal shear stresses $v = V/bd$ (d is the depth to the center of the tail extension of the hooked bar) are listed in Table 4. The reason for using this dimension is the nature of the shear crack (shown in Fig. 26) observed in several specimens. The magnitude of shear stress in the specimens in relation to the concrete strength parameter $\sqrt{f'_c}$ was computed and the value of $v/\sqrt{f'_c}$ is shown in Table 4. It is immediately apparent that the measured values are greatly in excess of shear stresses producing failure in typical flexural members.

The shear stress at diagonal tensile cracking v_c is given in the Code (Eq. 11.7) by

$$v_c \leq 3.5\sqrt{f'_c}\sqrt{1 + 0.002\, N_u/A_g}$$

where N_u is the axial load on the section and A_g is the gross area. For the specimens tested, the shear stress at cracking would be predicted at about $5.5\sqrt{f'_c}$ for the columns with low axial load (L), about $7\sqrt{f'_c}$ for those with moderate axial load (M), and about $9.3\sqrt{f'_c}$ for those with high axial load (H). In the case of the J7 series specimens, shear stress values in excess of the predicted values were measured at failure of the anchored bars and no shear cracks were observed. In the case of the J11 series, only the specimens with low axial loads cracked in shear. The shear force in the core at cracking and nominal shear stresses at cracking are listed in Table 4 for only the specimens which developed shear cracks. In those four specimens the measured shear stress at cracking was about 60 percent greater than computed and at the time the bars failed in anchorage the nominal shear stress in the core was about double that computed at cracking. This is particularly significant in specimen J11-90-15-1-L and J11-90-15-2-L, which contained no lateral reinforcement through the joint. Careful observation of the specimens indicated that after the shear crack formed there was very little change in the length or width of the crack and at no time was shear distress evident.

It is apparent that shear stress equations for flexural members cannot be realistically used for shear capacities of beam-column joints. While the equations for shear given in ACI 318-71 provide a conservative estimate of shear strength, the margin is so great that the designer may be unduly penalized by being forced to use large columns to satisfy joint strength requirements. The test results indicate that the shear strength of the joint is much larger than given by Eq. 11.7 of the Code. This is probably a result of the compressive forces on the core and of the large area of steel (column bars) crossing the crack. It is also possible that limiting the increase in shear stress provided by the lateral reinforcement to $8\sqrt{f'_c}$ may be quite conservative. However, considerable research is needed in this area before the influence of the significant parameters can be established.

5. PROPOSED DESIGN RECOMMENDATIONS

5.1 Introduction

The design recommendations proposed for computing the anchorage capacities of hooked bars reflect the trends observed in the experimental program and are summarized below.

(1) Strength is increased as restraint against side splitting is increased. Standard hooks embedded in mass concrete exhibit strengths well in excess of yield.[6]

(2) Much higher strengths can be permitted than in current specifications. For example, there is no reason a standard hooked bar with 60 ksi yield strength should be permitted to develop 30 ksi, while an identical hook of 40 ksi yield strength should be permitted to develop only 20 ksi. For small bar sizes these values are unrealistically prohibitive and as a result are probably frequently violated by designers. However, to allow higher stresses, minimum straight embedments before the hook are required, especially for larger bar sizes.

(3) No distinction is made between the strength of 90^{o} and 180^{o} standard hooks.

(4) Additional tail extensions over those required for a standard hook are not effective in increasing strength.

(5) Strength of #14 and #18 bars is not changed from ACI 318-71, because no valid test results are available for large hooks and such hooks would likely not commonly be used in frame structures.

To take these factors into account in developing design recommendations, the strength of hooked bar anchorages will be divided into three classes with the distinction between classes based on the lateral restraint to splitting which is provided.

5.2 Design Recommendations

Standard hooks shall be considered to develop tensile stresses in bar reinforcement equal to $f_h = \xi\sqrt{f'_c}$ but not greater than f_y, where ξ is based on the confining conditions of the hooked anchorage.

Class 1. For all bar sizes f_h shall be computed using ξ not less than $700(1 - 0.3d_b)$.

Class 2. For bar sizes #11 or smaller, f_h may be computed using $\xi = 1000(1 - 0.3d_b)$, provided the lead straight embedment between the standard hook and the critical section is not less than 4 bar diameters or 4 in. whichever is greater, side concrete cover normal to the plane of the hooked bar is not less than 2.5 in., and cover on the tail extension is not less than 2 in.

Class 3. For bar sizes #11 or smaller, f_h may be computed using $\xi = 1250(1 - 0.3d_b)$, if in addition to meeting the requirements for Class 2 hooks the bar is enclosed by ties spaced apart not further than $3d_b$. Standard hooks embedded in mass concrete where side splitting is of no concern may be considered as Class 3.

No increase in f_h shall be permitted for tail extensions or bend radii greater than required for a standard hook. To meet development length requirements an equivalent embedment length ℓ_e may be computed using Sec. 12.5(a) by substituting f_h for f_y and ℓ_e for ℓ_d. Only straight lead embedments greater than $4d_b$ or 4 in., whichever is greater, shall be considered effective in computing development length requirements. For better control of deflections and cracking, 90° hooks are preferable.

5.3 Comparison of Proposed Recommendations with ACI 318-71 and with Test Results

The proposed design recommendations are summarized in Figs. 27 and 28. Both figures also show the computed strengths which could be obtained using current ACI 318-71 provisions. Figure 27 shows the ξ values for bar sizes from #3 to #18. For hooks which do not meet the lateral restraint requirements for Class 2 and Class 3 joints, the values of ξ

are very similar to those given in 318-71. For bars which meet Class 2 and Class 3 cover requirements and tie requirements through the joints, the values of ξ are increased roughly 40 percent for Class 2 and about 80 percent for Class 3 hooks. Furthermore, no distinction is made for anchorage capacities of bars with different yield stresses.

Figure 28 shows the computed hook strength for standard hooks based on the proposed design recommendations and on ACI 318-71 provisions. The stresses assigned to standard hook anchorages are considerably increased if the confinement conditions for Class 2 and Class 3 hooks are met. For example, under present Code restrictions the maximum stress on a #8 hook is roughly 38 ksi (50 ksi if the 30 percent increase is permitted for "enclosure" in Sec. 12.8.1); however, with proper attention to cover and tie requirements, that same hook would now be rated at 60 ksi if the concrete strength is 5000 psi. It should be noted that the type of confinement needed to increase the hook capacity for Class 2 and 3 is explicitly stated in the proposed recommendations, whereas the Code is vague about the type of enclosure needed.

Computed strengths using these recommendations for the bars tested in this study are listed in Table 5. Ratios of measured-to-computed strength vary from a minimum of about 1.15 to 1.7. Table 5 also lists the stress corresponding to 0.6 of the computed strength of the hooked bar anchorage and the measured lead slip for the test specimens at 0.6 of the computed strength. The rationale behind these comparisons is to give an indication of the possible crack width at the beam-column joint due to slip of the anchorage at service loads. The figures indicate that in none of the specimens was the slip greater than 0.016 at the assumed service load stress. The limit for crack width suggested in the Commentary (Sec. 10.6) is 0.016. The crack width at the face of the column is assumed to be about equal to the lead slip of the anchored bar. In general, for the #7 bars, measured slip values at "service loads" were in the range of 0.005 to 0.008 in. The only specimens which exceeded these values were those with 180° hooks and the specimens with 12 in. column thickness. In the case of the #11 bars, the slip at service loads tended to be higher but still less than the 0.016 in. crack width suggested in the Commentary.

In evaluating the proposed design recommendations or those contained in ACI 318-71 it is apparent that the main "gap" in the experimental evidence is the suggested computation for additional straight lead embedment to achieve desired stresses at critical sections. While the stress capacity of the hook can be reasonably estimated, the reliability of the equation for development length in computing embedment lengths for stress values less than f_y, say 10 or 20 ksi, cannot be verified from the available test results.

Based on these observations, it would appear that design recommendations can be adjusted to realistically reflect the actual strength of hooks and to satisfy required serviceability criteria.

REFERENCES

1. Minor, J., and Jirsa, J. O., "A Study of Bent-Bar Anchorages," Structural Research at Rice University, Report No. 9, Rice University, Houston, March 1971.

2. Marques, J. L. G., "A Study of Anchorage Capacities of Confined Bent-Bar Reinforcement," M.S. thesis submitted to Rice University, Houston, May 1972.

3. ACI Committee 318, Building Code Requirements for Reinforced Concrete (ACI 318-71), American Concrete Institute, Detroit, 1971.

4. ACI Committee 318, Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-71), American Concrete Institute, Detroit, 1971.

5. Untrauer, R. E., and Henry, R. L., "Influence of Normal Pressure on Bond Strength," ACI Journal, Proc. Vol. 62, May 1965, pp. 577.

6. Hribar, J. A., and Vasko, R. C., "End Anchorage of High Strength Steel Reinforcing Bars," ACI Journal, Proc. Vol. 66, November 1969, pp. 875.

TABLE 1. PROPERTIES OF TEST SPECIMENS

Specimen	Col. Size, in.	Hook Angle, deg.	Lead Embed., in.	Lateral Confinement*	N, Col. Load, k	Conc. Strength, f'_c, ksi		Bar Failed+
						Bottom Batch	Top Batch	
J7-90-15-1-H	12x15	90	9.5	1	545	4.6	4.6	Left
J7-90-15-1-M	12x15	90	9.5	1	269	5.0	5.1	Right
J7-90-15-1-L	12x15	90	9.5	1	145	4.8	4.8	Right
J7-90-12-1-H	12x12	90	6.5	1	420	4.4	3.9	Left
J7-180-15-1-H	12x15	180	9.5	1	545	3.9	4.1	Right
J7-180-12-1-H	12x12	180	6.5	1	425	4.0	4.7	Left
J7-90-15-2-H	12x15	90	9.5	2	545	4.8	4.7	Right
J7-90-15-2-M	12x15	90	9.5	2	274	4.7	4.8	Right
J7-90-15-3-H	12x15	90	9.5	3	555	4.5	4.8	Right
J7-90-15-3a-H	12x15	90	9.5	3a	535	3.9	3.6	Right
J7-90-15-4-H	12x15	90	9.5	4	548	4.4	4.6	Right
J11-90-15-1-H	12x15	90	6.0	1	540	4.8	5.0	Left
J11-90-15-1-L	12x15	90	6.0	1	154	4.8	4.7	Right
J11-90-12-1-H	12x12	90	3.0	1	437	4.5	4.7	Right
J11-180-15-1-H	12x15	180	6.0	1	540	4.2	4.6	Right
J11-90-15-2-H	12x15	90	6.0	2	540	5.1	4.9	Right
J11-90-15-2-L	12x15	90	6.0	2	125	4.6	4.4	Left
J11-90-15-3-L	12x15	90	6.0	3	150	4.9	4.8	Right
J11-90-15-3a-L	12x15	90	6.0	3a	175	5.0	5.0	Both

*Lateral Confinement:
1. Col. Bars + 2-7/8 in. cover
2. Only 2-7/8 in. cover
3. 2-7/8 in. cover + #3 ties at 5 in. through joint
3a. 2-7/8 in. cover + #3 ties at 2.5 in. through joint
4. Only 1-1/2 in. cover

+As viewed from back of column.

TABLE 2. SUMMARY OF MEASURED SLIP BEHAVIOR

Specimen	Stress, ksi, @ Lead slip of			Lead Slip, in. at $0.6(f_h + f_\ell)$ Lead Stress	Slip @ 2H, in. at $0.6(f_h + f_\ell)$ Lead Stress	Approx. Slip, in. @ Failure	Stress,ksi at Failure
	0.005 in.	0.016 in.	0.05 in.				
J7-90-15-1-H	35	56	79	0.006	0.002	0.14	91
J7-90-15-1-M	33	61	79	0.005	0.001	0.18	100
J7-90-15-1-L	30	52	78	0.006	0.001	0.18	97
J7-90-12-1-H	19	31	58	0.014	0.011	0.08	62
J7-180-15-1-H	24	45	73	0.009	0.003	0.15	87
J7-180-12-1-H	22	38	59	0.010	0.007	0.07	61
J7-90-15-2-H	32	52	75	0.007	0.003	0.22	99
J7-90-15-2-M	33	47	70	0.008	0.004	0.21	95
J7-90-15-3-H	39	65	80	0.005	0.002	0.21	104
J7-90-15-3a-H	38	63	80	0.003	0.002	0.18	98
J7-90-15-4-H	28	53	72	0.008	0.003	0.05	73
J11-90-15-1-H	19	32	48	0.006	0.003	0.055	48
J11-90-15-1-L	19	28	48	0.010	0.007	0.06	52
J11-90-12-1-H	18	30	Failed	0.006	0.001	0.04	42
J11-180-15-1-H	13	25	43	0.012	0.007	0.05	45
J11-90-15-2-H	22	34	48	0.006	0.001	0.06	49
J11-90-15-2-L	18	28	50	0.008	0.003	0.06	53
J11-90-15-3-L	15	26	52	0.012	0.007	0.09	62
J11-90-15-3a-L	24	42	66	0.005	0.003	0.065	69

TABLE 3. MEASURED AND COMPUTED+ ANCHORAGE STRENGTH

Specimen	f_h, comp., ksi+		f_h, meas. ksi	f_h: $\frac{\text{Meas.}}{\text{Comp.}}$*	f_ℓ, comp.+ ksi	Comp* $f_h + f_\ell$	Meas. f at failure	$\frac{\text{Meas.}}{\text{Comp.}}$+
	Top Cast	Other						
J7-90-15-1-H	24.4	36.5	66	1.8	26.8	59.3	91	1.53
J7-90-15-1-M	25.8	38.5	65	1.7	28.3	62.6	100	1.59
J7-90-15-1-L	25.2	37.8	65	1.7	27.6	61.2	97	1.58
J7-90-12-1-H	23.2	34.9	62	1.8	17.4	48.4	62	1.29
J7-180-15-1-H	22.7	34.1	64	1.9	24.9	55.2	87	1.58
J7-180-12-1-H	23.7	35.6	61	1.7	17.8	49.4	61	1.23
J7-90-15-2-H	24.8	37.3	73	2.0	27.3	60.4	99	1.64
J7-90-15-2-M	24.8	37.3	70	1.9	27.3	60.4	95	1.57
J7-90-15-3-H	24.6	36.8	70	1.9	27.0	59.7	104	1.73
J7-90-15-3a-H	22.1	33.1	66	2.0	24.3	53.7	98	1.82
J7-90-15-4-H	24.1	36.1	62	1.7	26.5	58.6	73	1.25
J11-90-15-1-H	24.9	29.0	48	1.6	6.6	35.6	48	1.36
J11-90-15-1-L	24.8	28.9	52	1.8	6.6	35.5	52	1.46
J11-90-12-1-H	24.5	28.6	42	1.5	3.3	31.9	42	1.30
J11-180-15-1-H	23.9	27.9	45	1.6	6.4	34.3	45	1.31
J11-90-15-2-H	25.6	29.8	49	1.6	6.8	36.6	49	1.33
J11-90-15-2-L	24.2	28.2	53	1.9	6.4	34.6	53	1.52
J11-90-15-3-L	25.2	29.4	62	2.1	6.7	36.1	62	1.71
J11-90-15-3a-L	25.5	29.7	69	2.3	6.8	36.5	69	1.88

+ACI 318-71

*Using values for other bars.

TABLE 4. SHEAR IN CORE OF JOINT

Specimen	V @ bar failure, k	$v = V/bd$ ksi	$\frac{v}{\sqrt{f'_c}}$	V_c at shear* cracking k	$v_c = V/bd$ ksi	$\frac{v_c}{\sqrt{f'_c}}$	$\frac{v_c}{\sqrt{f'_c}\,(1 + .002N/A_g)}$ +
J7-90-15-1-H	109	0.73	10.7				
J7-90-15-1-M	120	0.80	11.2				
J7-90-15-1-L	116	0.77	11.5				
J7-90-12-1-H	75	0.67	10.3				
J7-180-15-1-H	105	0.70	11.1				
J7-180-12-1-H	73	0.65	9.8				
J7-90-15-2-H	119	0.79	11.4				
J7-90-15-2-M	114	0.76	11.0				
J7-90-15-3-H	124	0.83	11.6				
J7-90-15-3a-H	118	0.79	12.9				
J7-90-15-4-H	88	0.59	8.8				
J11-90-15-1-H	151	1.01	14.4				
J11-90-15-1-L	162	1.08	15.7	143	0.96	13.9	8.4
J11-90-12-1-H	129	1.13	16.7				
J11-180-15-1-H	140	0.94	14.2				
J11-90-15-2-H	152	1.02	14.4				
J11-90-15-2-L	164	1.10	16.4	132	0.88	13.2	8.5
J11-90-15-3-L	193	1.29	18.5	142	0.95	13.6	8.3
J11-90-15-3a-L	214	1.43	20.2	131	0.87	12.4	7.2

*Only listing is for those specimens which cracked in shear prior to anchorage failure of bars.

+Eq. 11-7 (ACI 318-71).

TABLE 5. COMPARISON OF MEASURED AND COMPUTED ANCHORAGE STRENGTH USING PROPOSED DESIGN RECOMMENDATIONS

Specimen	Computed stress, ksi	Stress at failure, ksi	Ratio: $\frac{\text{Meas.}}{\text{Comp.}}$	0.6 Comp.	Measured Lead Slip, in., at 0.6 Comp.
J7-90-15-1-H	60	91	1.6	36	0.006
J7-90-15-1-M	60	100	1.7	36	0.005
J7-90-15-1-L	60	97	1.6	36	0.006
J7-90-12-1-H	50	62	1.2	30	0.015
J7-180-15-1-H	60	87	1.4	36	0.012
J7-180-12-1-H	51	61	1.2	31	0.011
J7-90-15-2-H	60	99	1.6	36	0.007
J7-90-15-2-M	60	95	1.6	36	0.008
J7-90-15-3-H	60	104	1.7	36	0.005
J7-90-15-3a-H	60	98	1.6	36	0.005
J7-90-15-4-H	50	73	1.4	30	0.006
J11-90-15-1-H	41	48	1.15	25	0.009
J11-90-15-1-L	40	52	1.3	24	0.012
J11-90-12-1-H	28	42	1.5	17	0.005
J11-180-15-1-H	39	45	1.15	24	0.015
J11-90-15-2-H	41	49	1.2	25	0.007
J11-90-15-2-L	39	53	1.3	24	0.011
J11-90-15-3-L	42	62	1.5	25	0.014
J11-90-15-3a-L	52	69	1.3	31	0.010

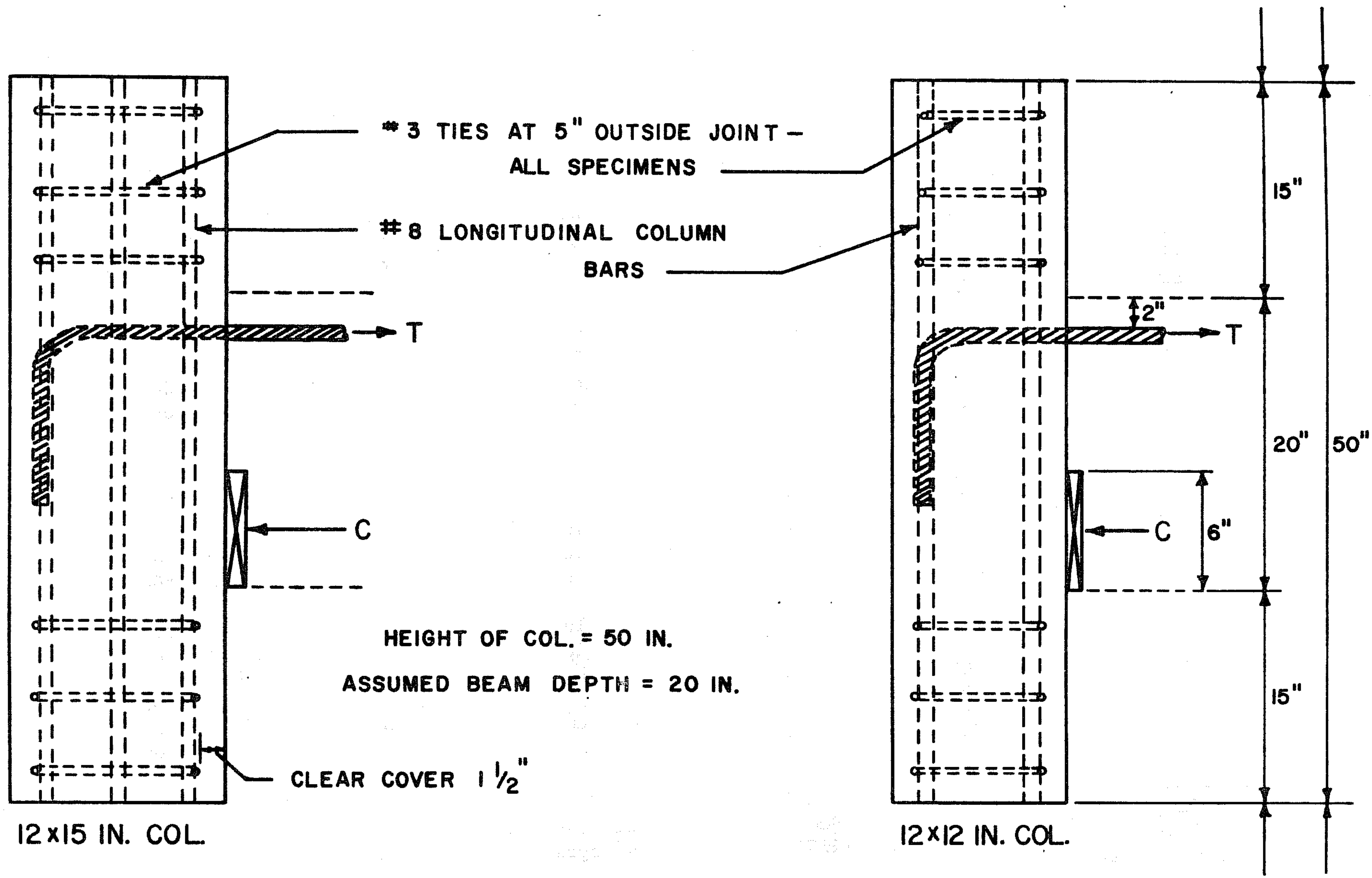


FIG. 1. DETAILS OF TEST SPECIMENS.

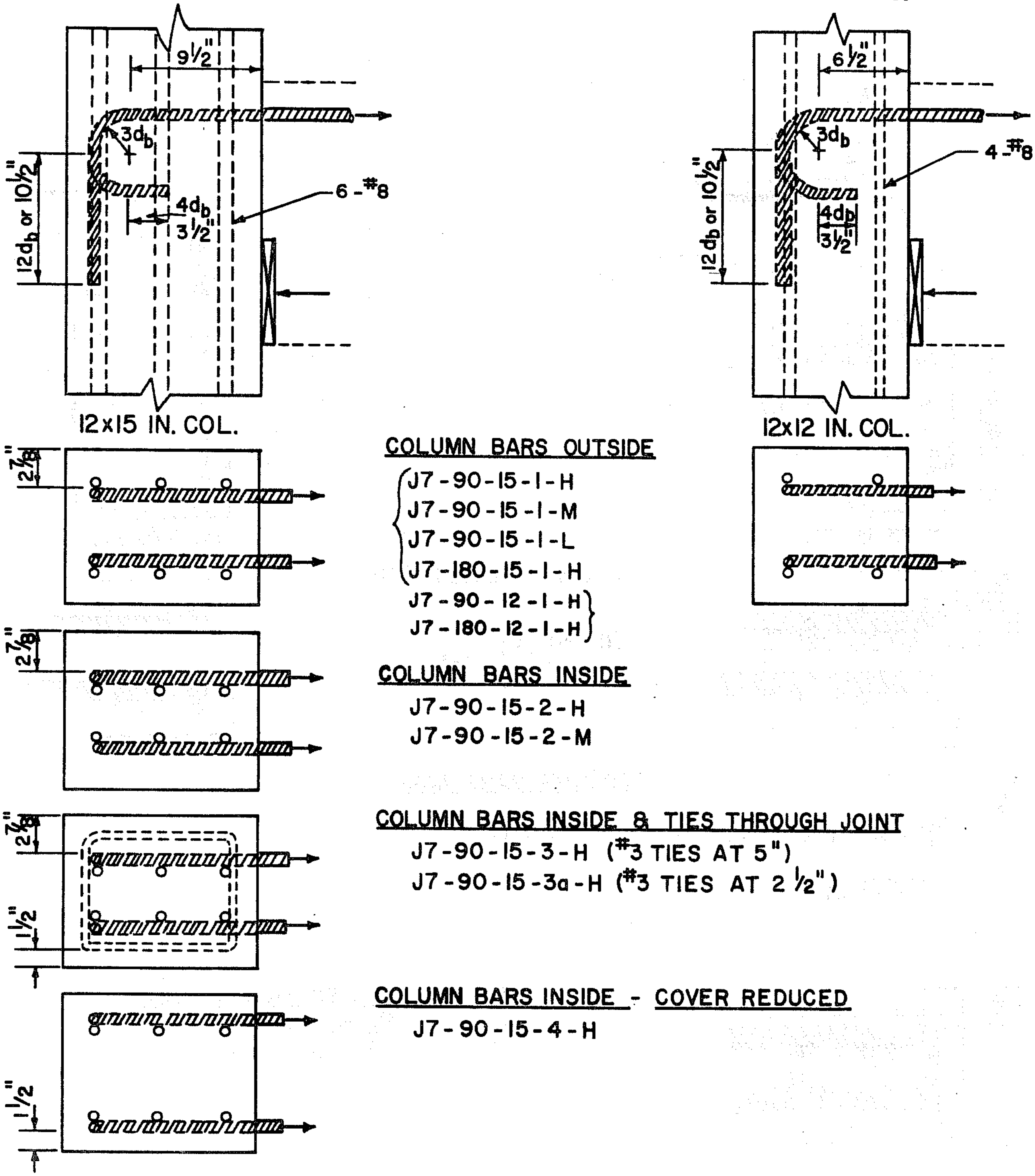


FIG. 2. JOINT DETAILS--J7 SERIES (#7 BARS).

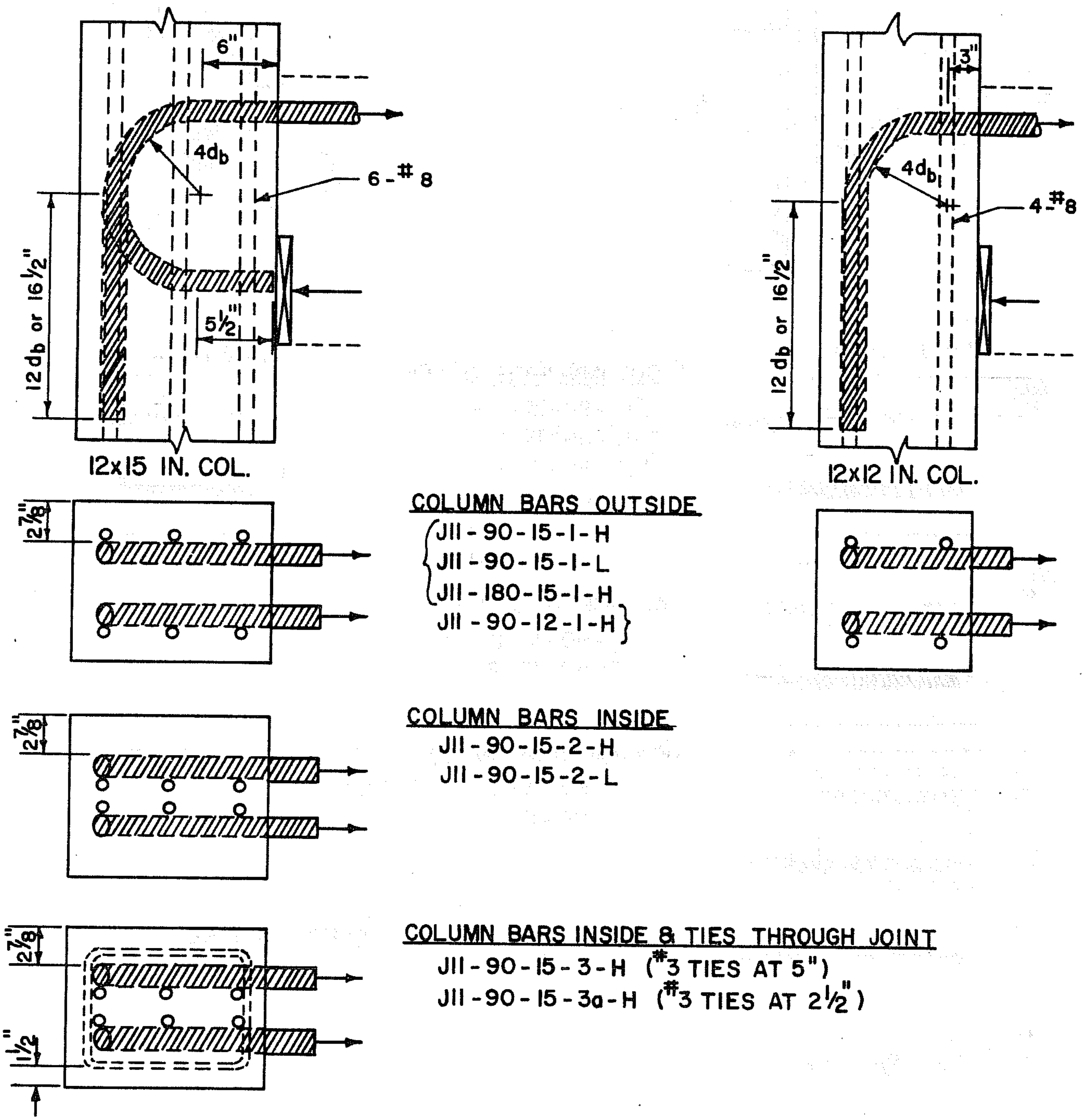


FIG. 3. JOINT DETAILS--J11 SERIES (#11 BARS).

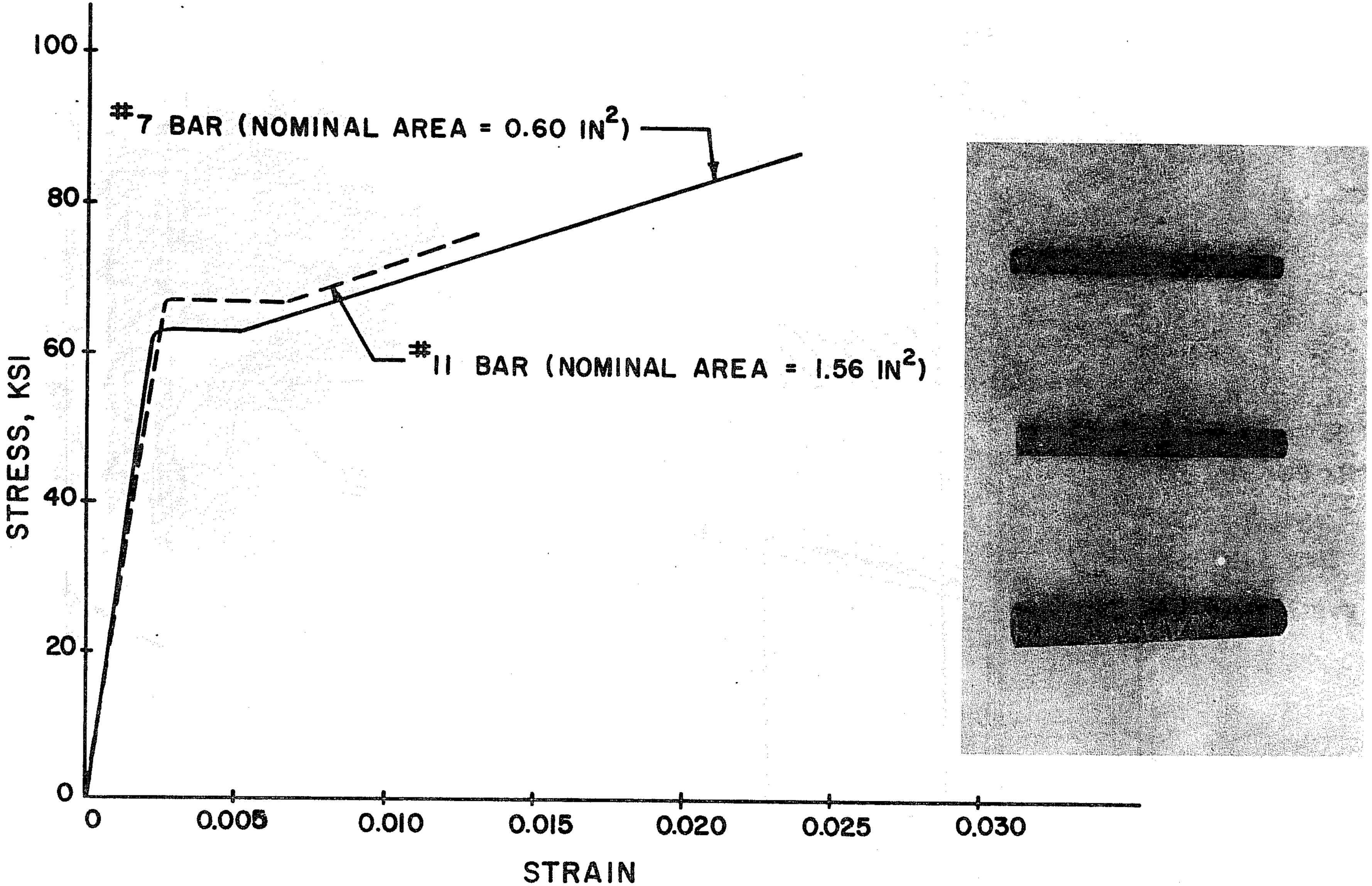


FIG. 4. TYPICAL STRESS-STRAIN CURVES FOR #7 AND #11 GRADE 60 BARS.

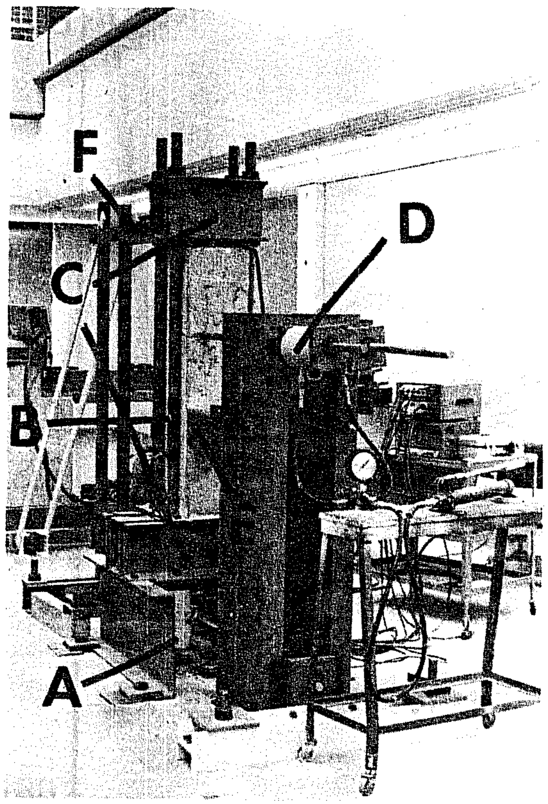


FIG. 5a. TEST SETUP.

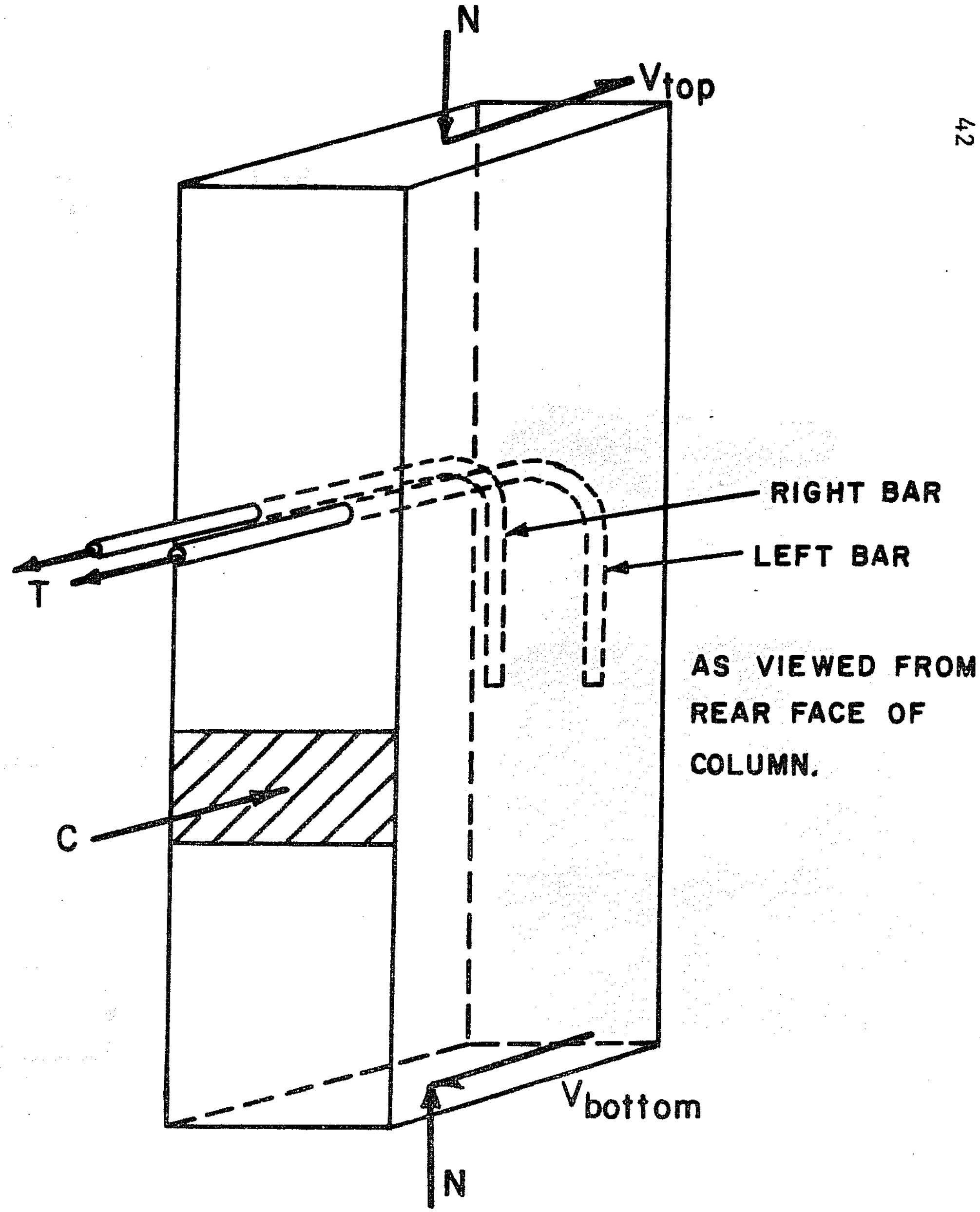
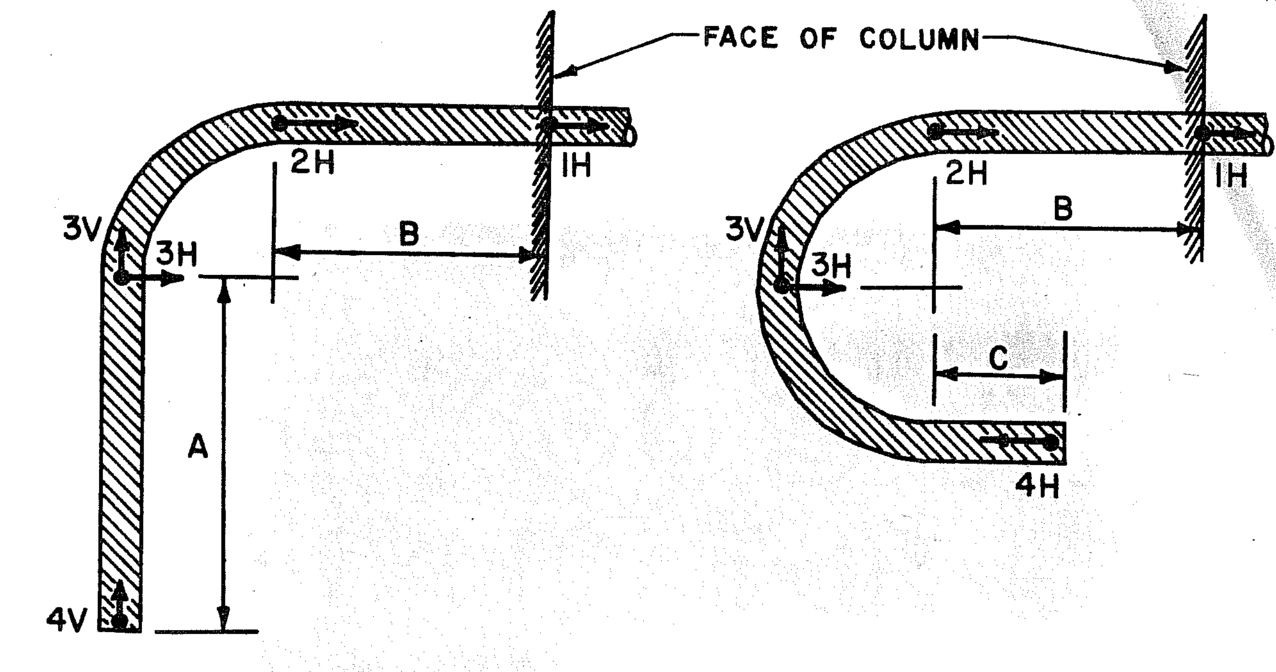
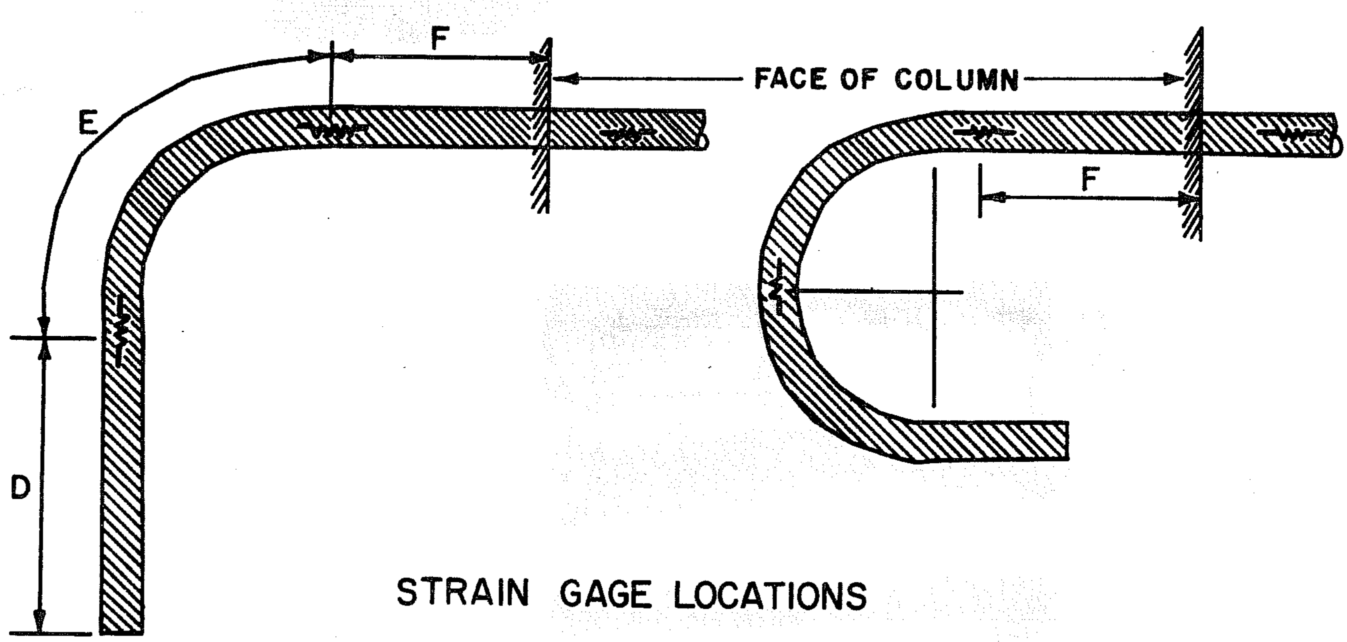


FIG. 5b. FORCES ON TEST SPECIMEN.



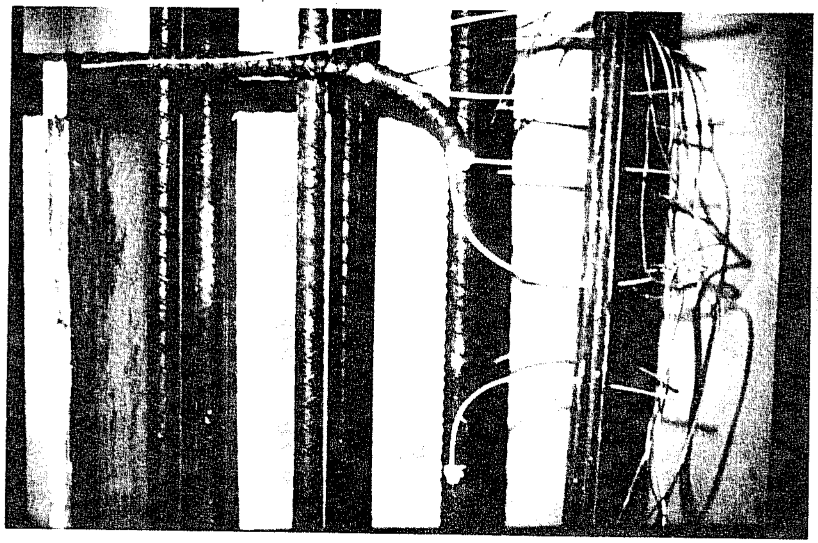
SLIP MEASUREMENT POINTS



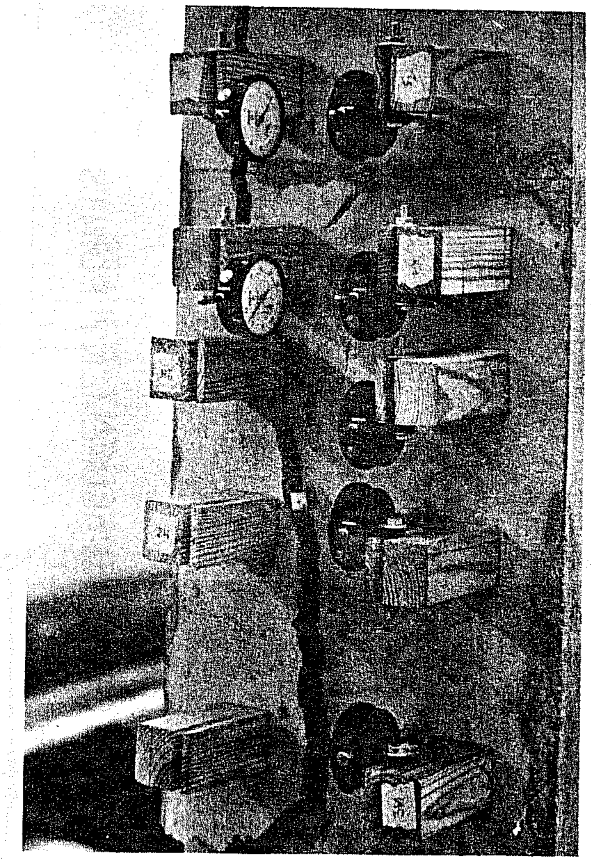
STRAIN GAGE LOCATIONS

	A	B	C	D	E	F
J7 Series						
12x15	10 1/2	9 3/8	3 1/2	8 9/16	8 9/16	8 9/16
12x12	10 1/2	6 3/8	3 1/2	8 9/16	8 9/16	5 9/16
J11 Series						
12x15	16 1/2	6 1/4	5 1/2	16 1/2	9	6 1/4
12x12	16 1/2	3 1/4	5 1/2	16 1/2	9	3 1/4

FIG. 6. INSTRUMENTATION.



(a) Wires Attached to Beam Bars



(b) Dial Gages in Place

FIG. 7. SLIP MEASURING SYSTEM.

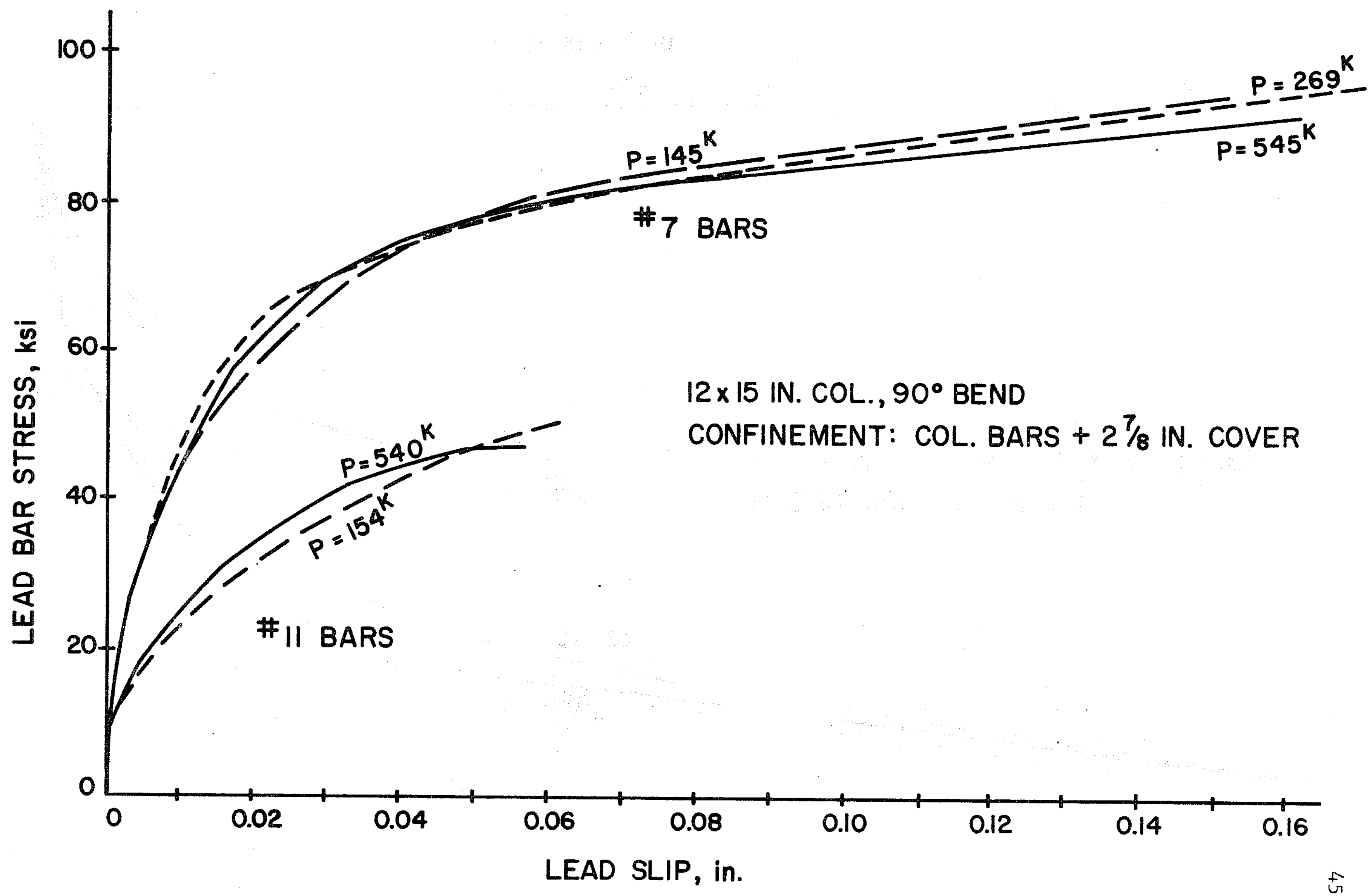


FIG. 8. INFLUENCE OF COLUMN AXIAL LOAD ON SLIP.

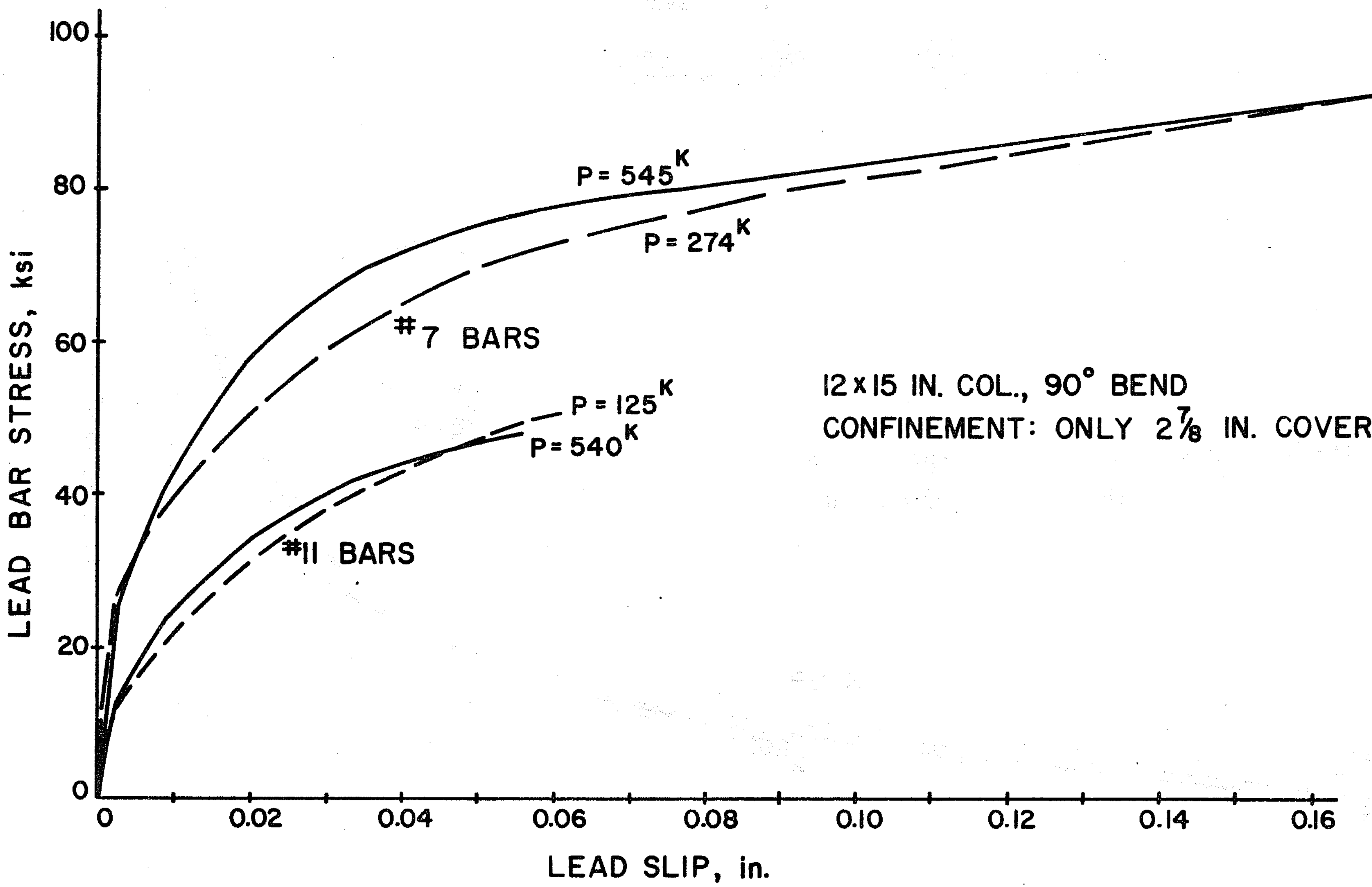


FIG. 9. INFLUENCE OF COLUMN AXIAL LOAD ON SLIP.

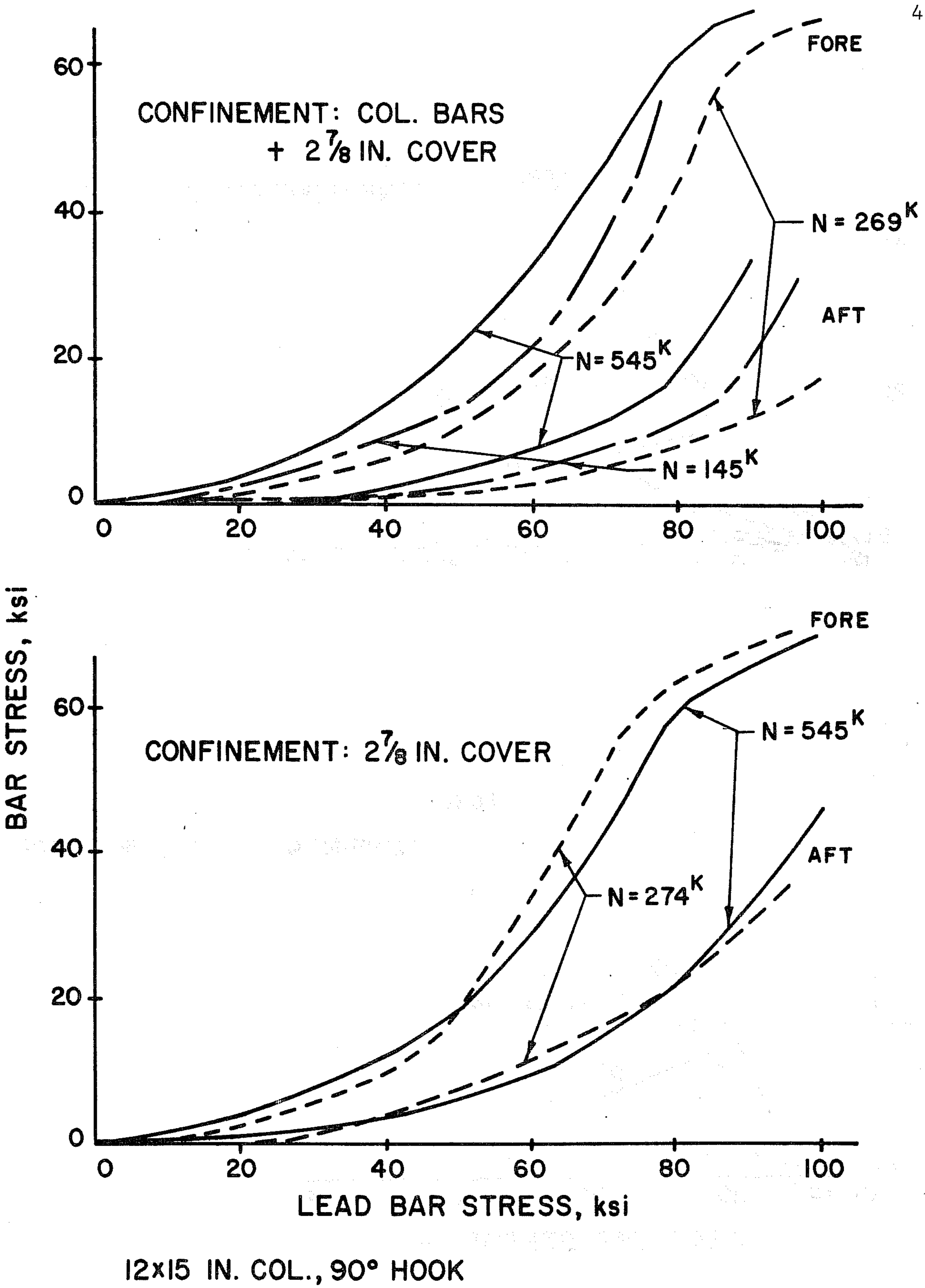


FIG. 10. INFLUENCE OF COLUMN AXIAL LOAD ON STRESSES, #7 BARS.

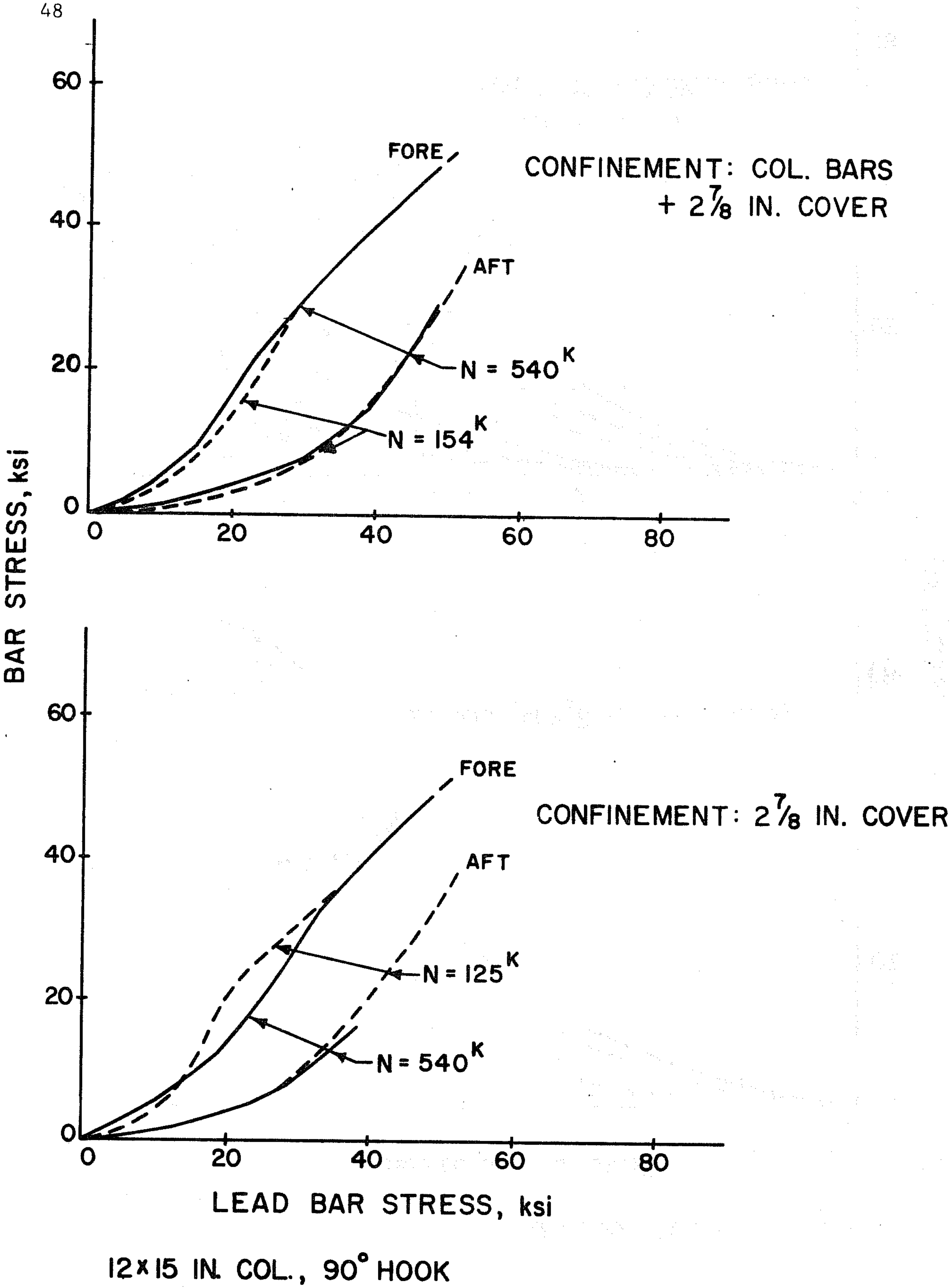


FIG. 11. INFLUENCE OF COLUMN AXIAL LOAD ON STRESSES, #11 BARS.

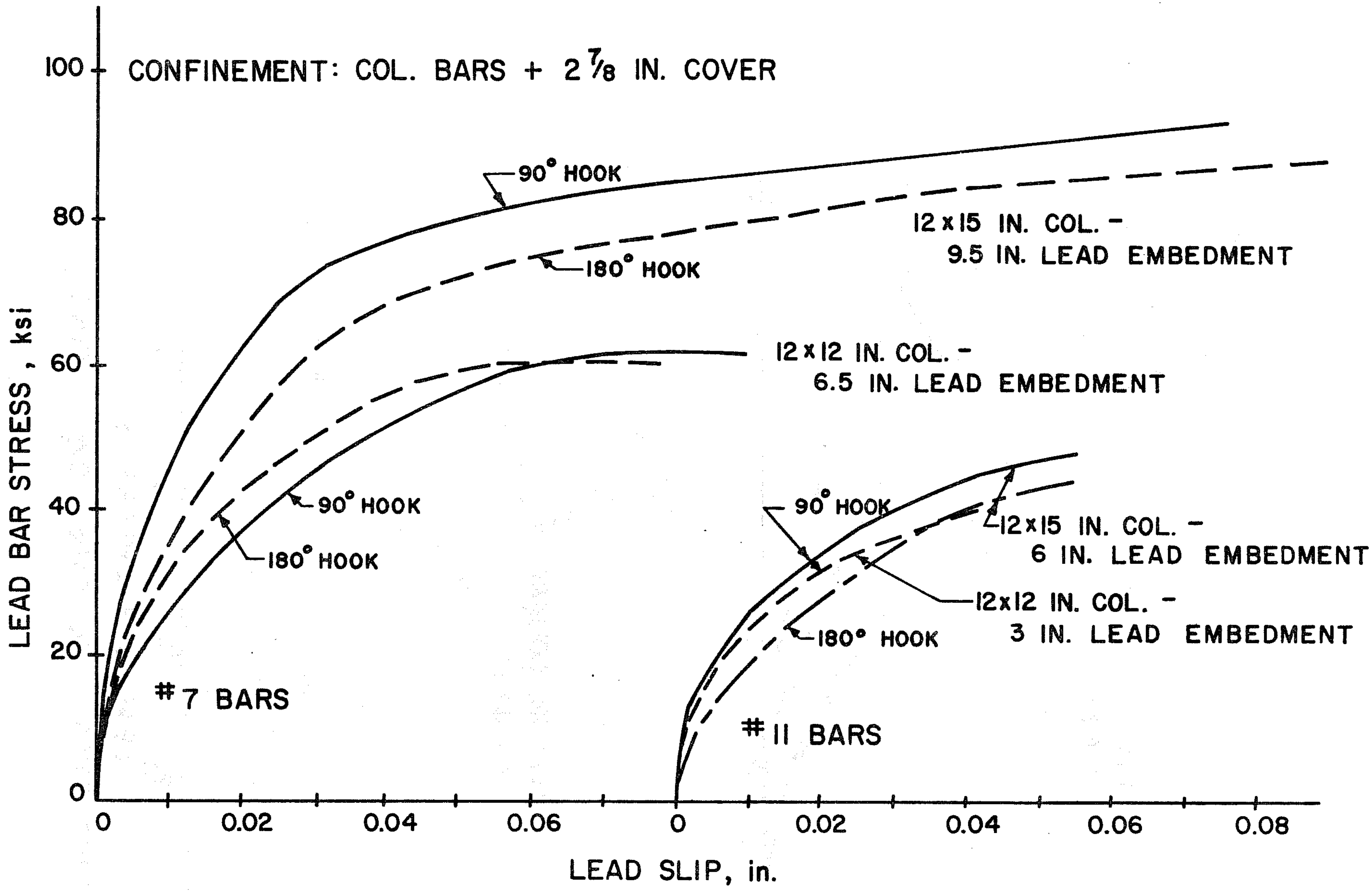


FIG. 12. INFLUENCE OF BEND ANGLE AND LEAD EMBEDMENT ON SLIP.

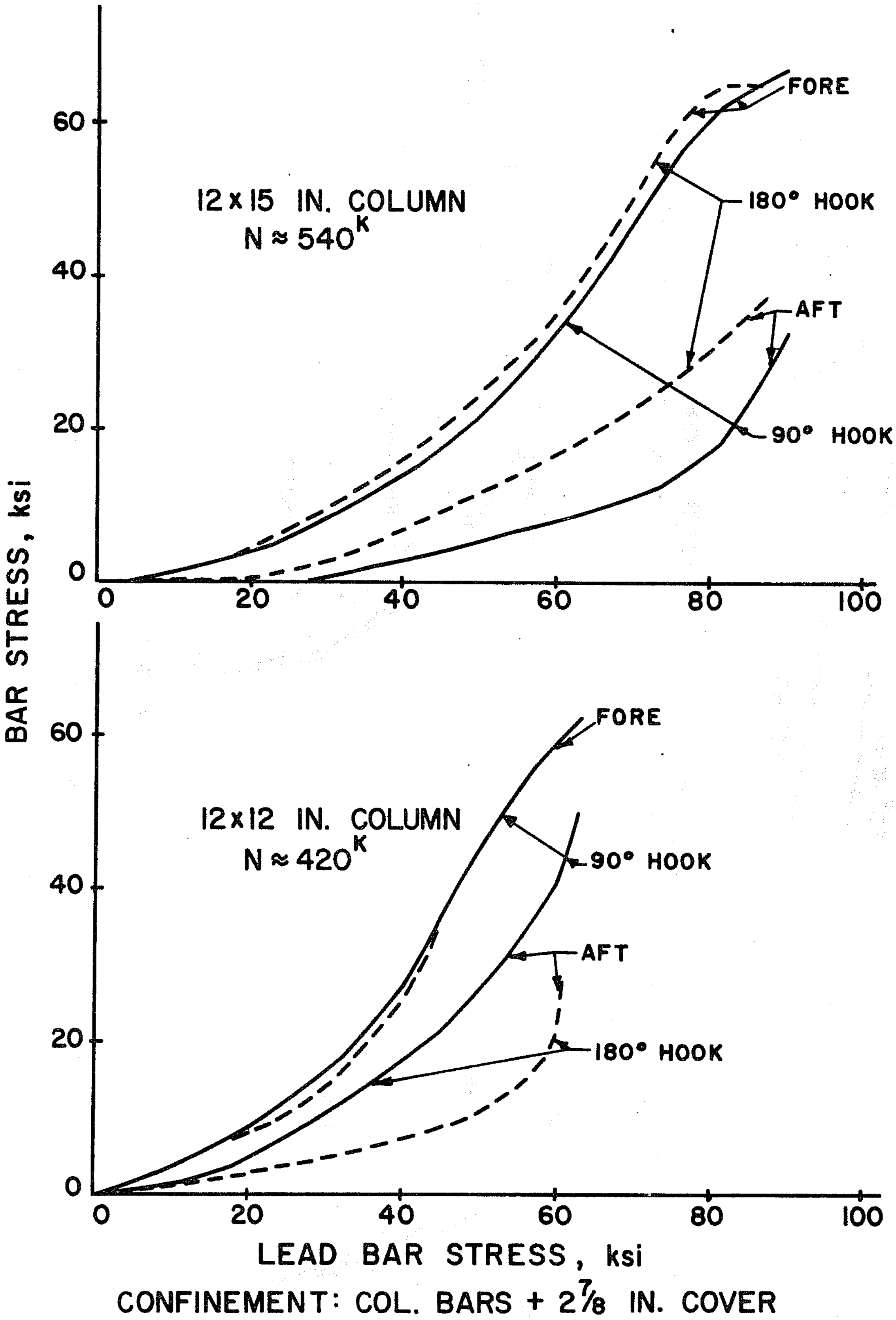


FIG. 13. INFLUENCE OF BEND ANGLE ON STRESSES, #7 BARS.

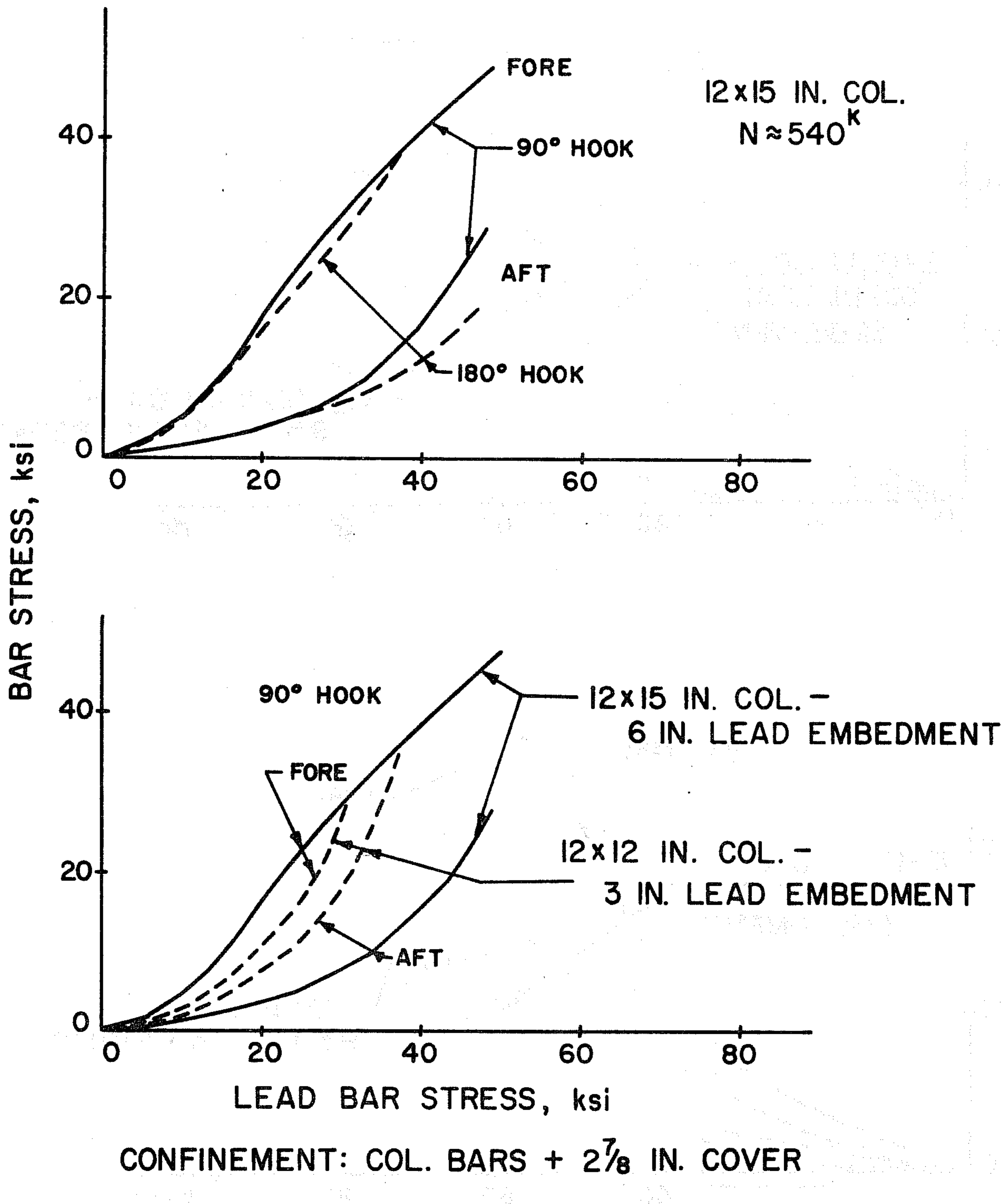


FIG. 14. INFLUENCE OF BEND ANGLE AND LEAD EMBEDMENT ON STRESSES, #11 BARS.

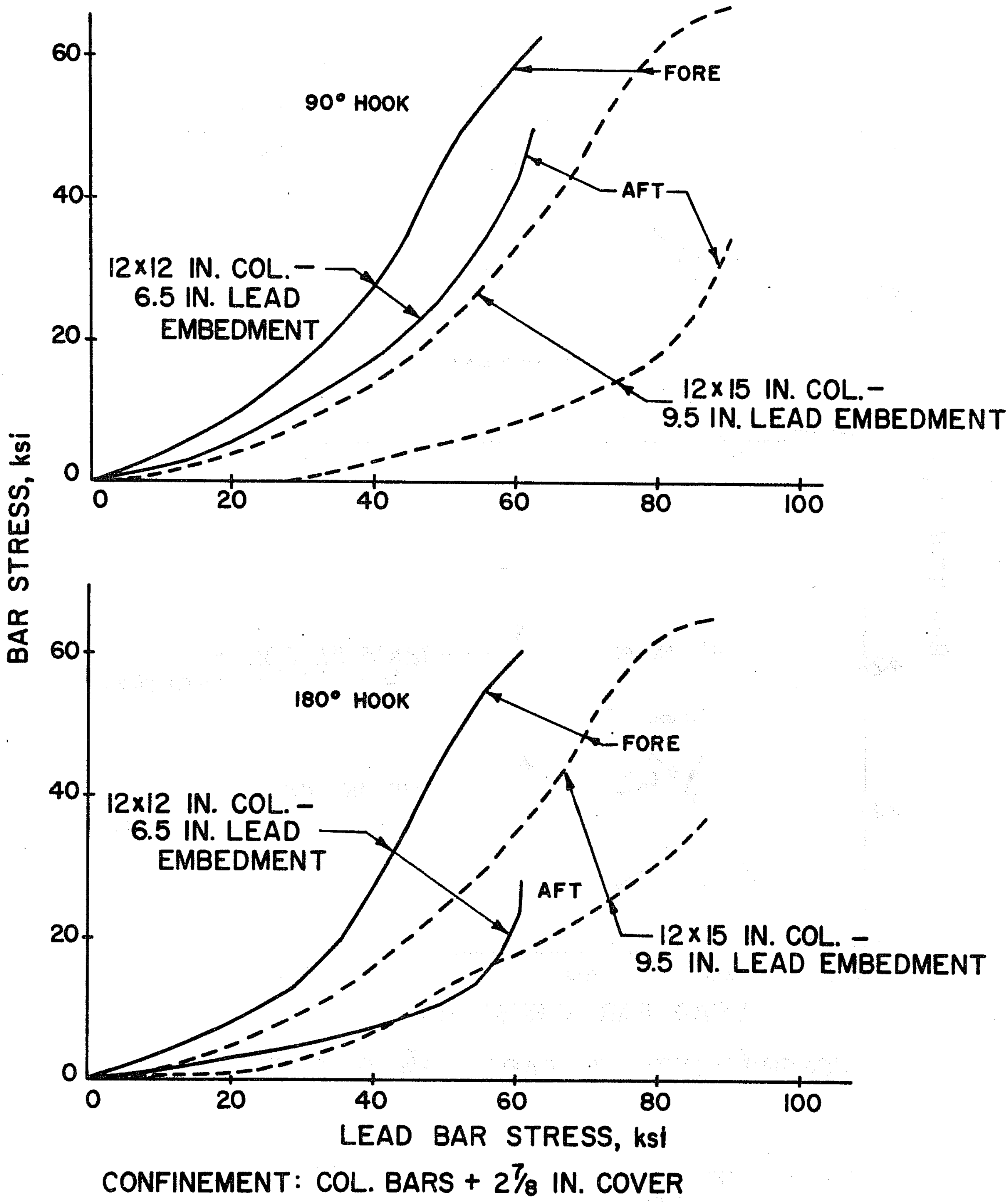


FIG. 15. INFLUENCE OF LEAD EMBEDMENT ON STRESSES, #7 BARS.

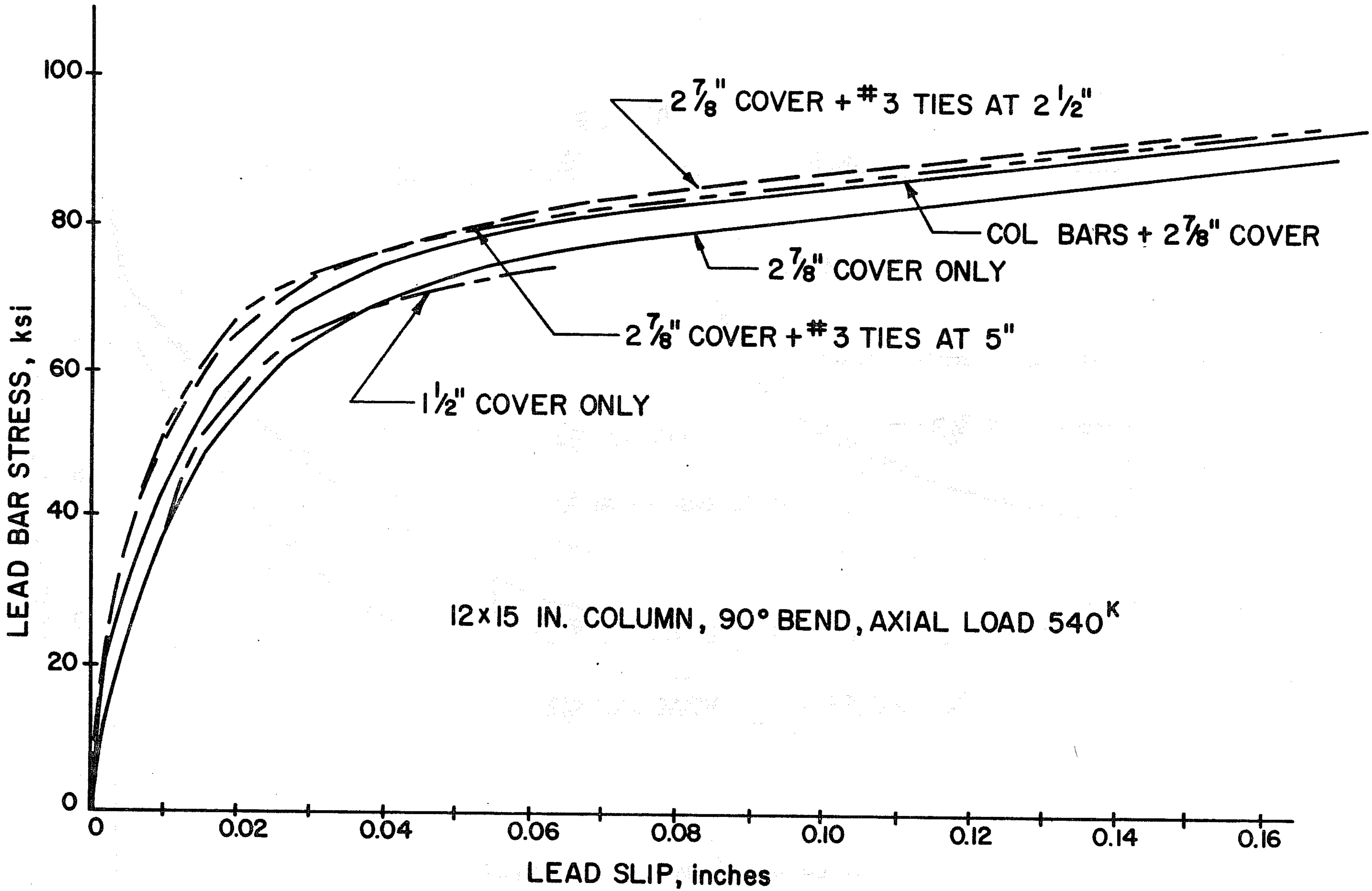


FIG. 16. INFLUENCE OF CONFINEMENT ON SLIP--#7 BARS.

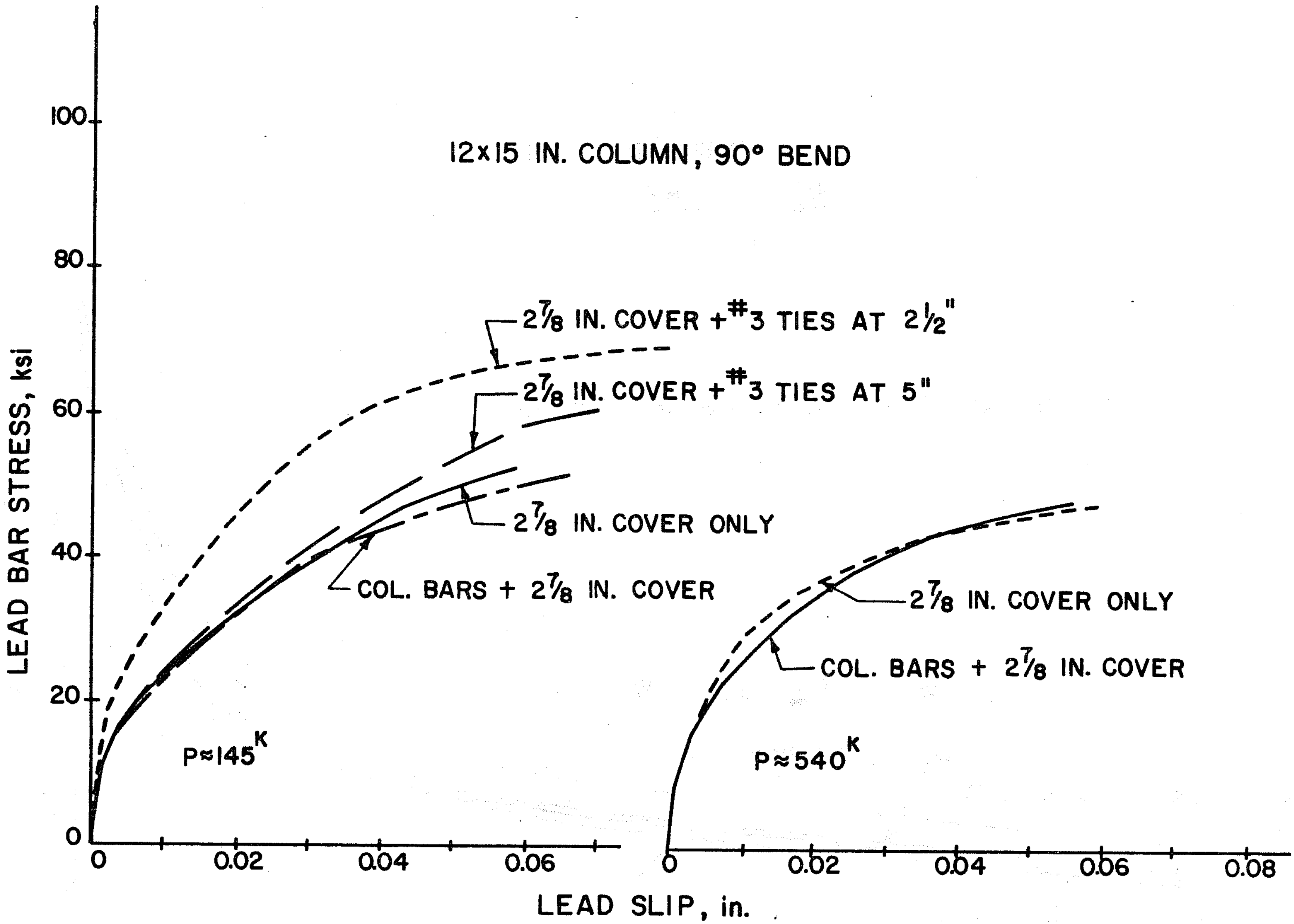


FIG. 17. INFLUENCE OF CONFINEMENT ON SLIP--#11 BARS.

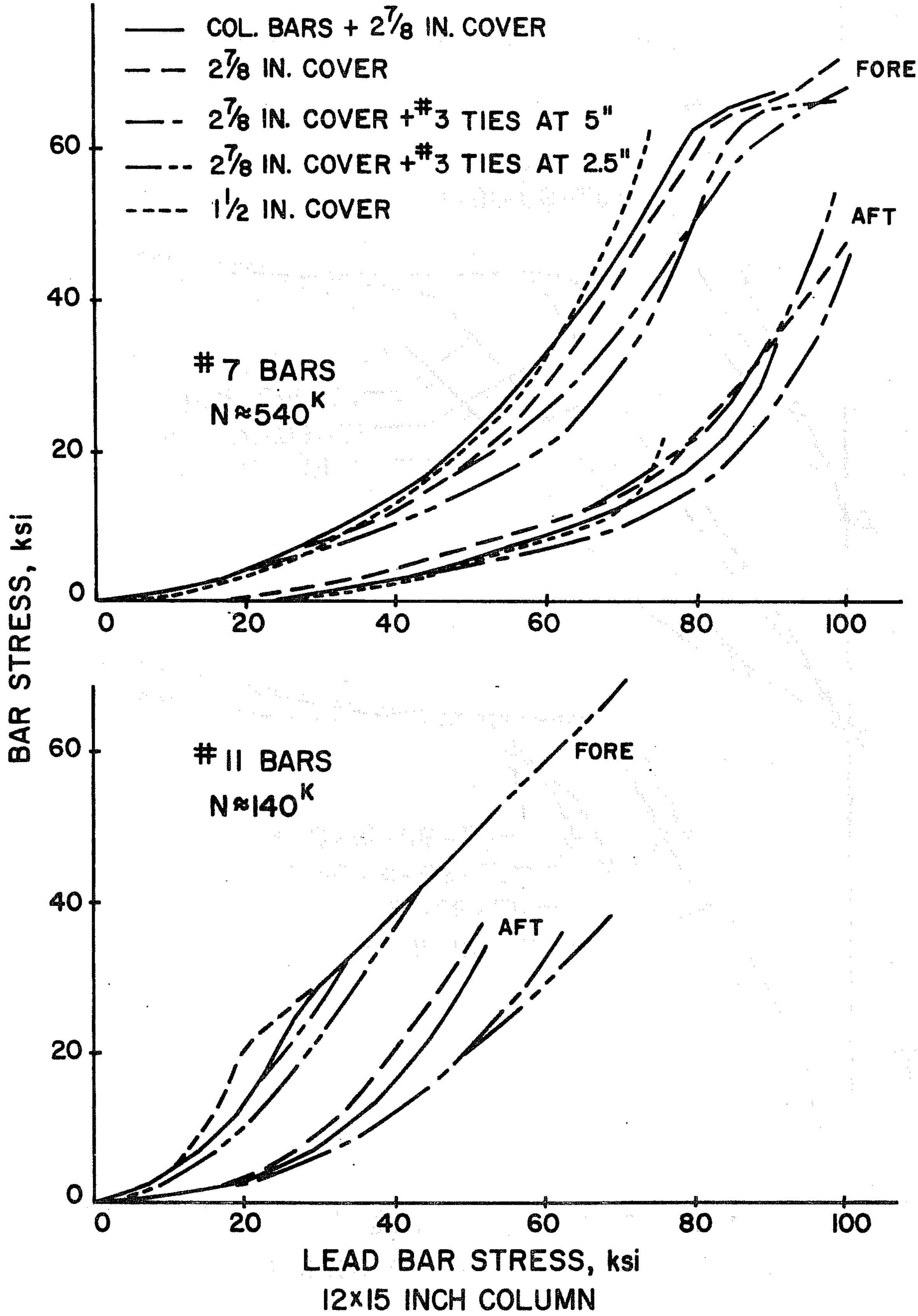


FIG. 18. INFLUENCE OF CONFINEMENT ON STRESSES.

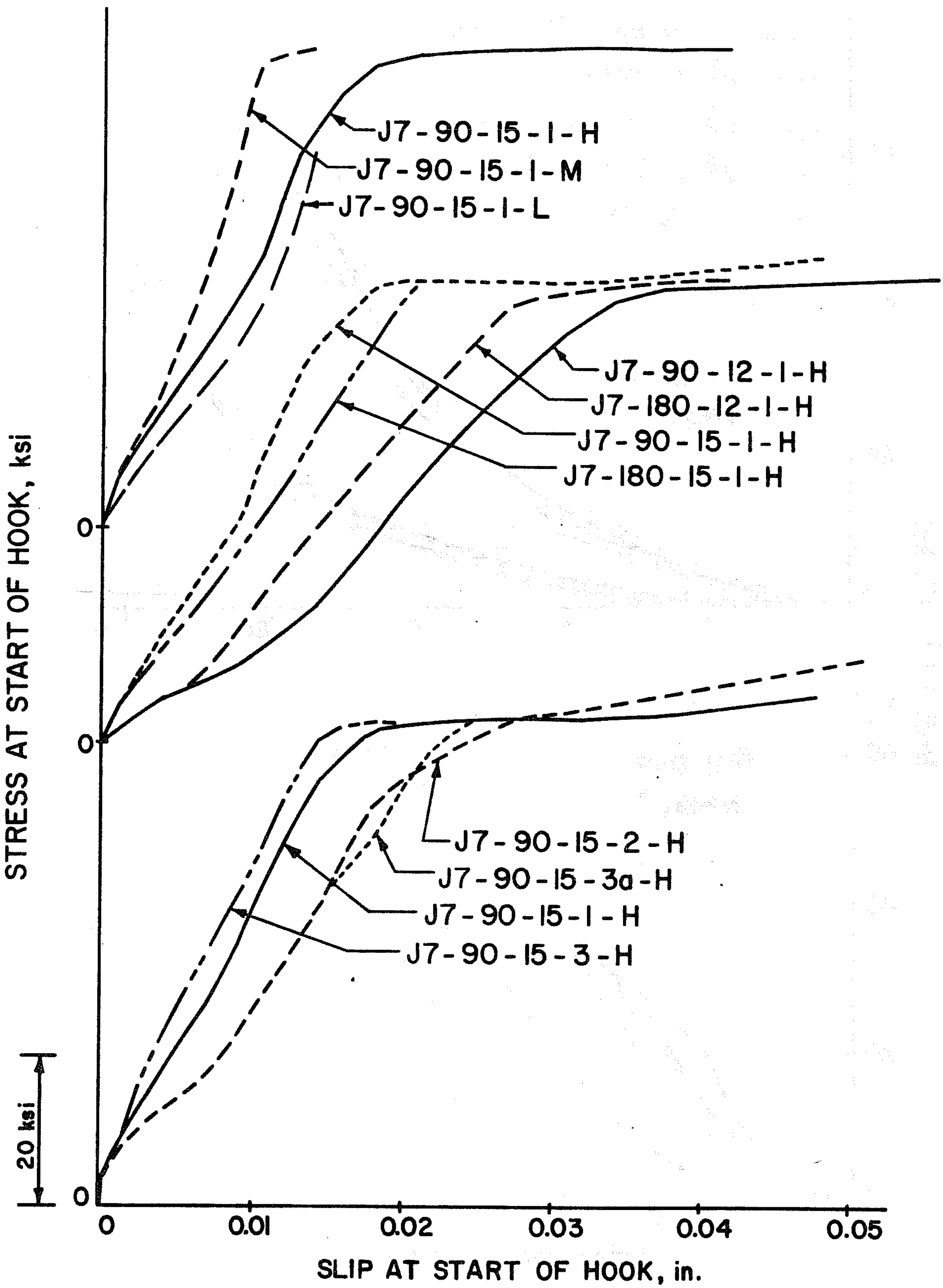


FIG. 19. STRESS-SLIP CURVES FOR HOOKS, #7 BARS.

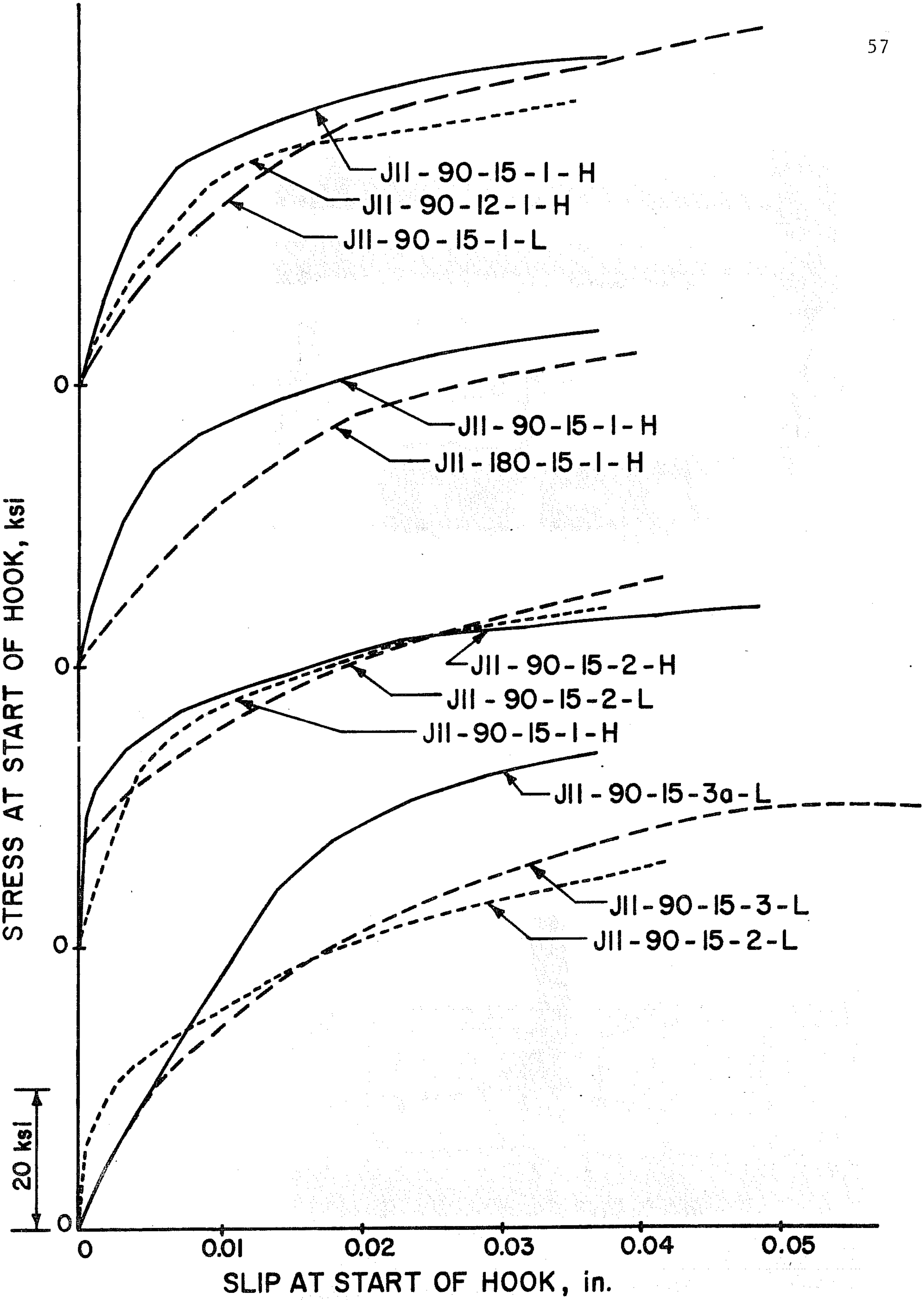
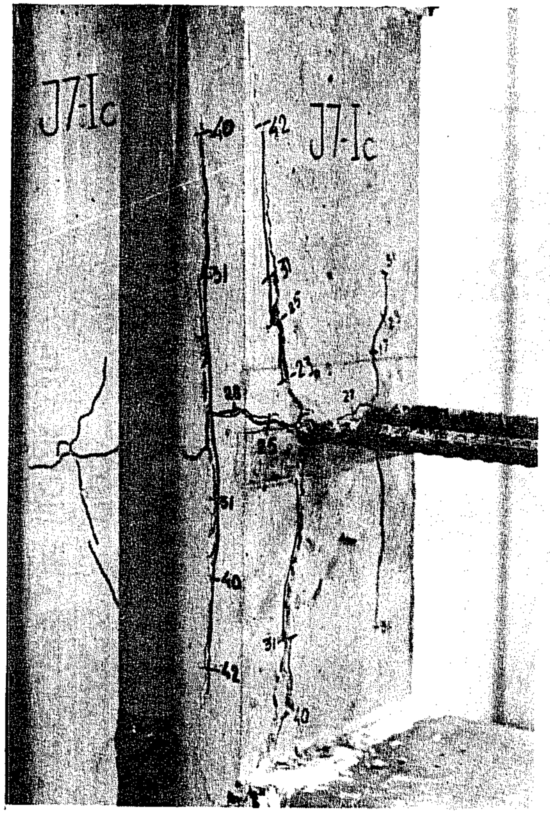
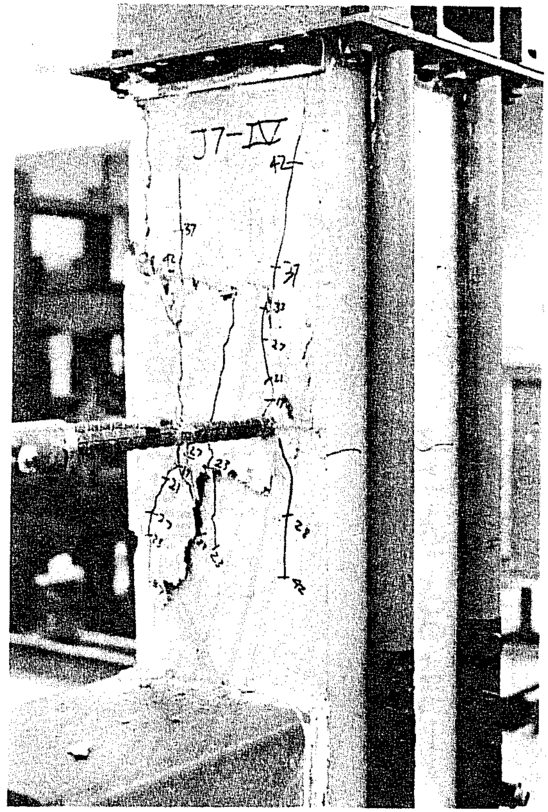


FIG. 20. STRESS-SLIP CURVES FOR HOOKS, #11 BARS.



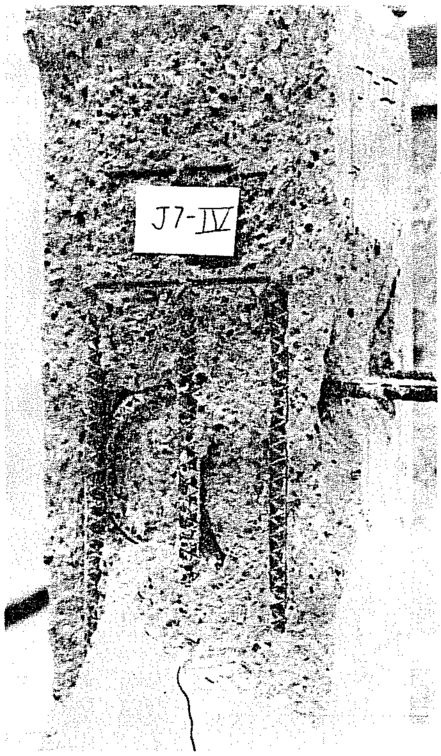
J7-90-15-1-L



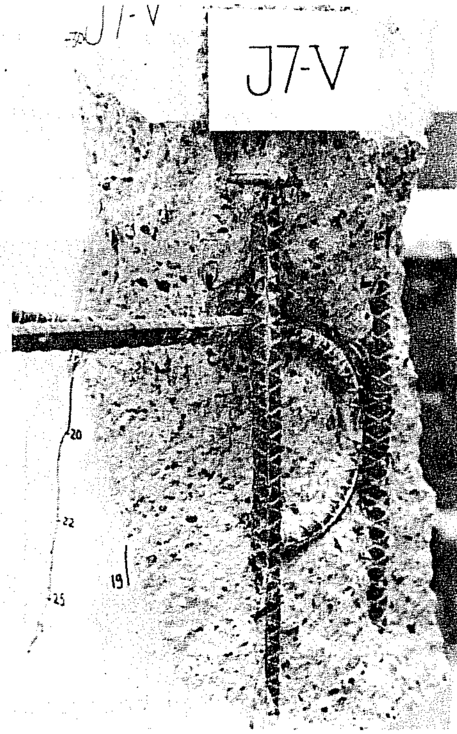
J7-180-15-2-H

FIG. 21. CRACKING PRIOR TO FAILURE

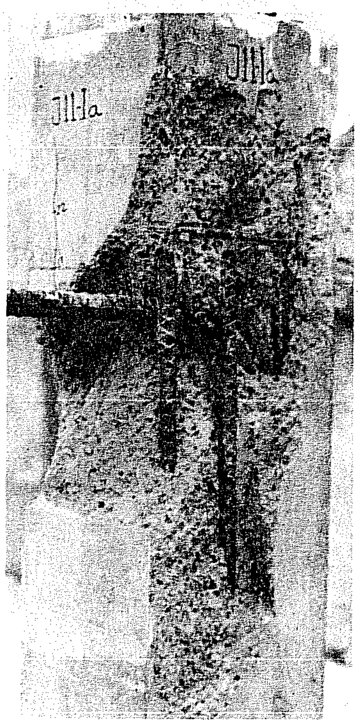
J7-90-12-1-H



J7-180-15-1-H



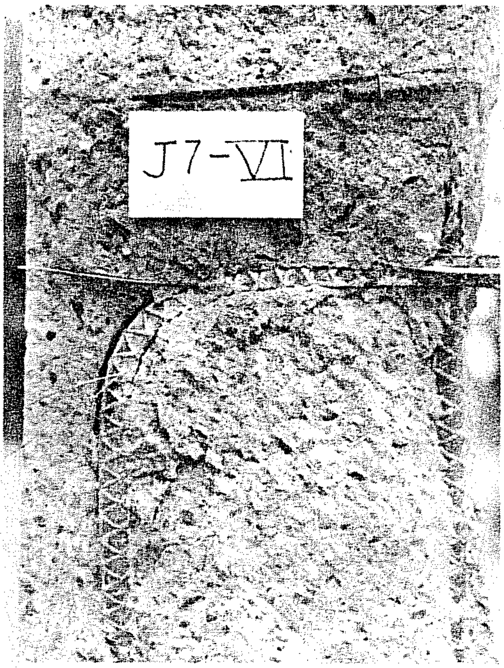
J7-180-12-1-H



J11-90-15-1-H

J11-90-15-2-H

FIG. 22. APPEARANCE OF SPECIMENS AFTER FAILURE



J7-90-15-2-H

J7-90-15-4-H

J7-90-15-3a-H

J7-90-15-3-H

FIG. 23. APPEARANCE OF SPECIMENS AFTER FAILURE.

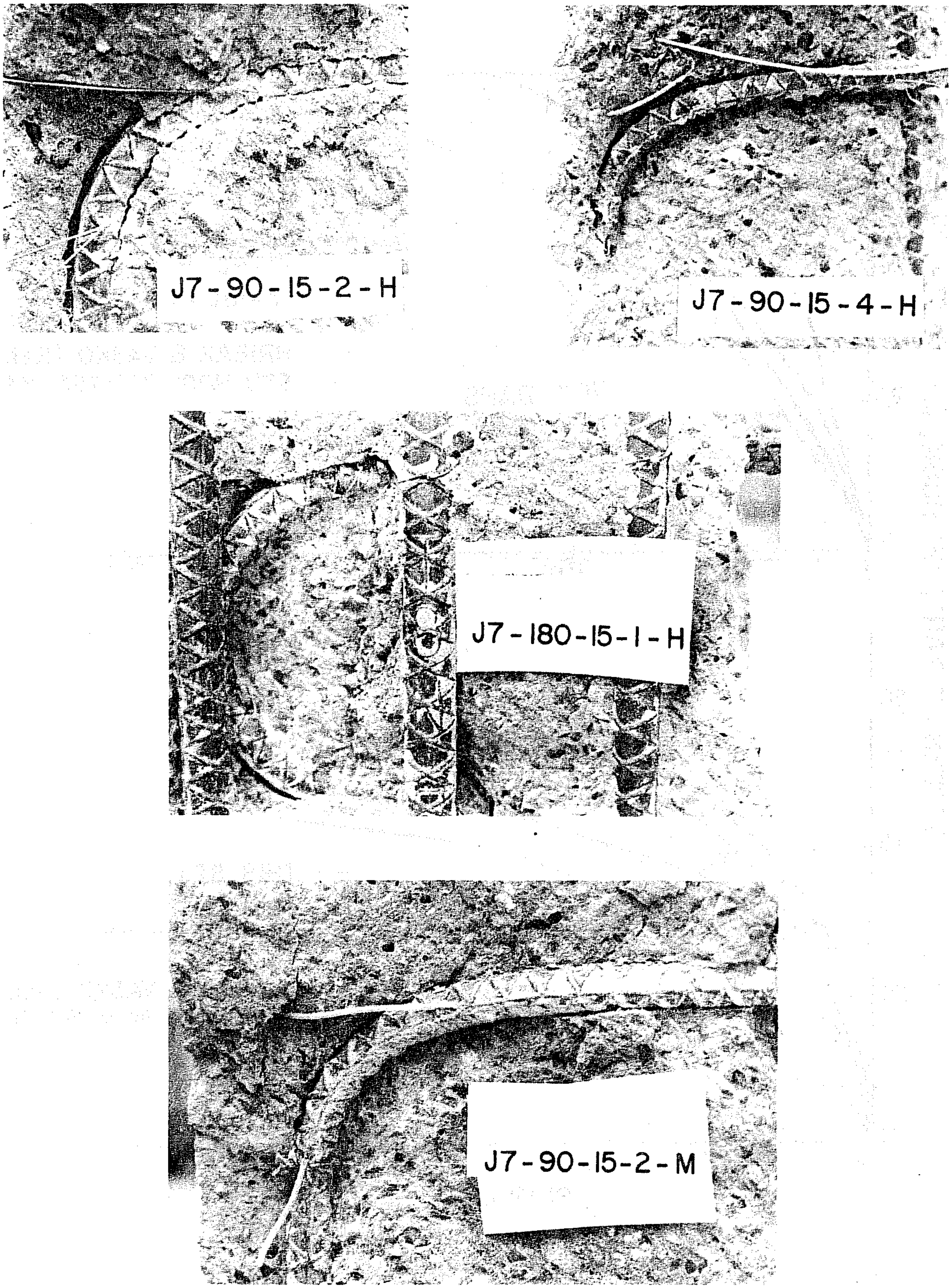


FIG. 24. SLIP AND CRUSHING IN VICINITY OF BEND.

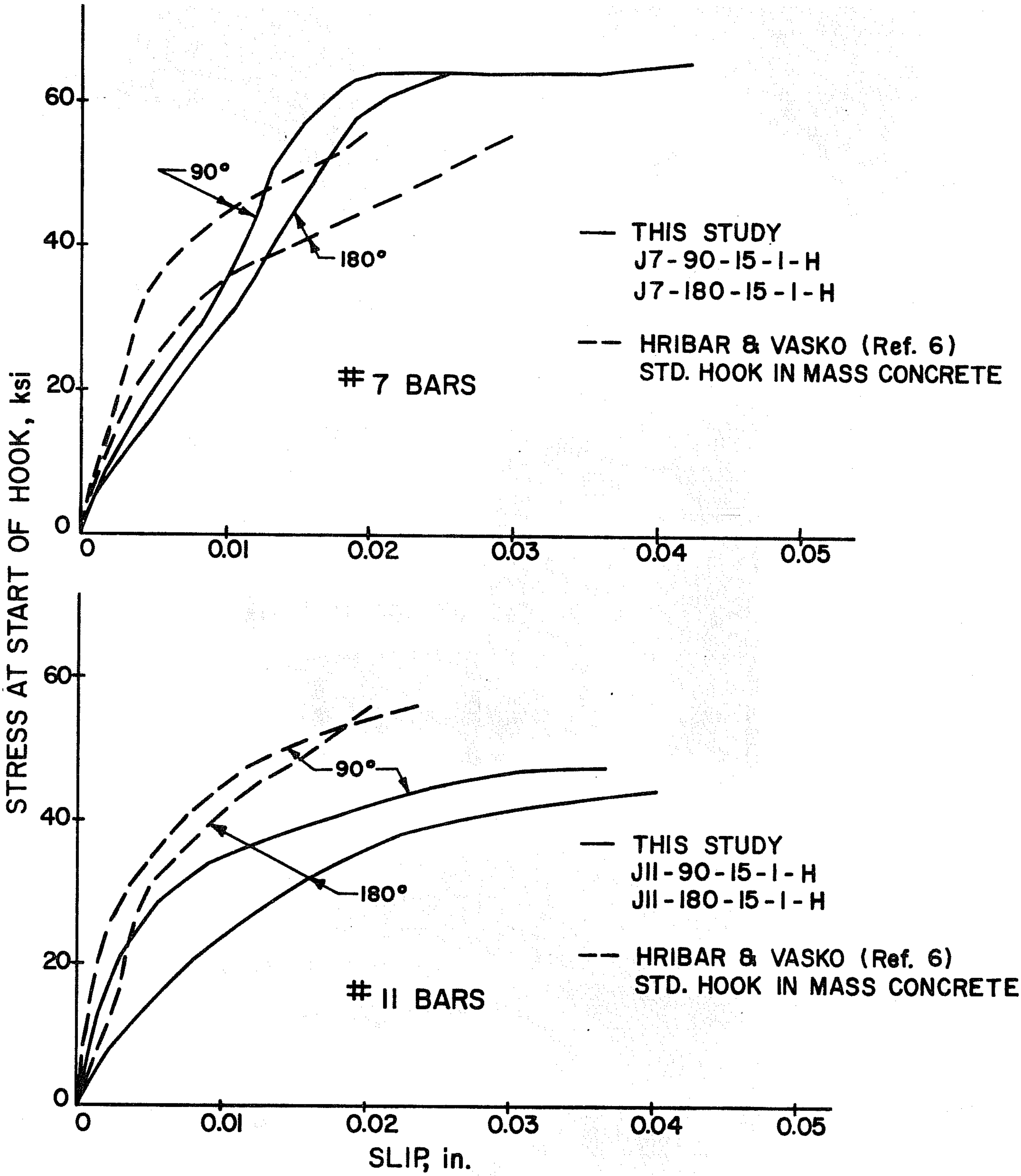
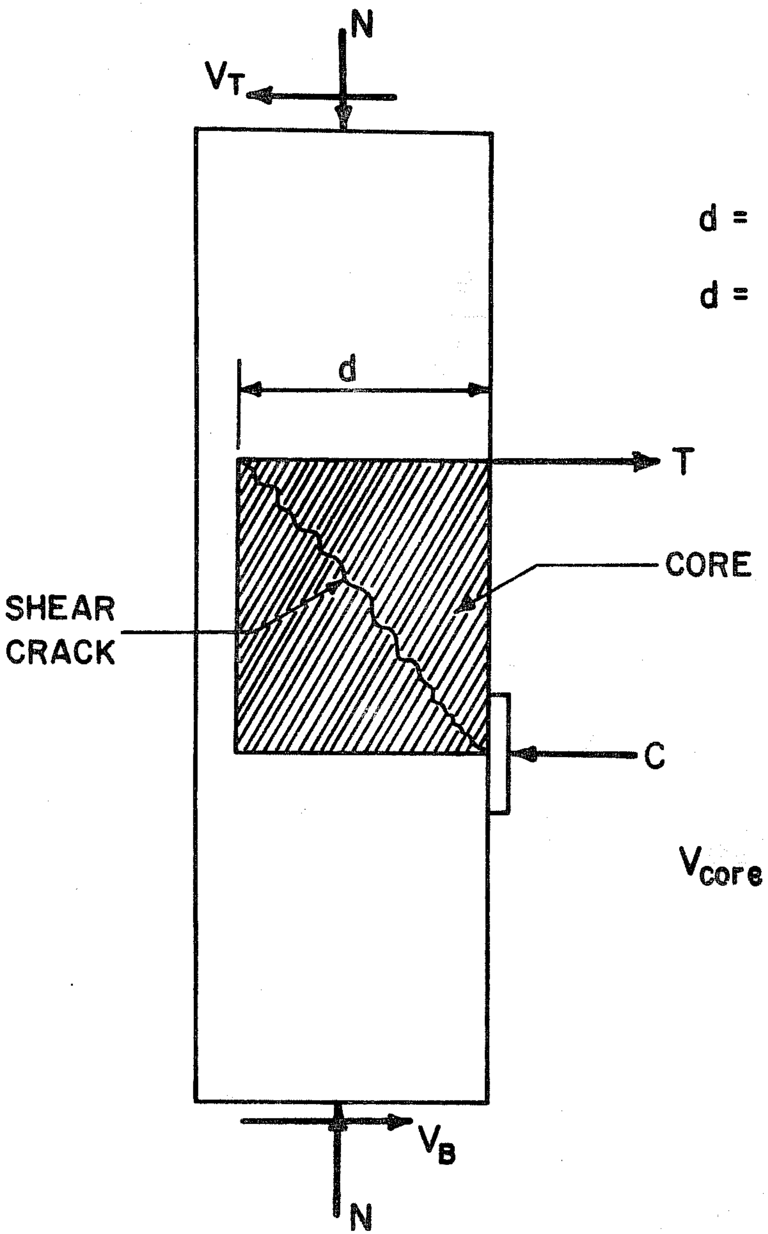


FIG. 25. COMPARISON OF STRESS-SLIP CURVES WITH HOOKS IN MASS CONCRETE.

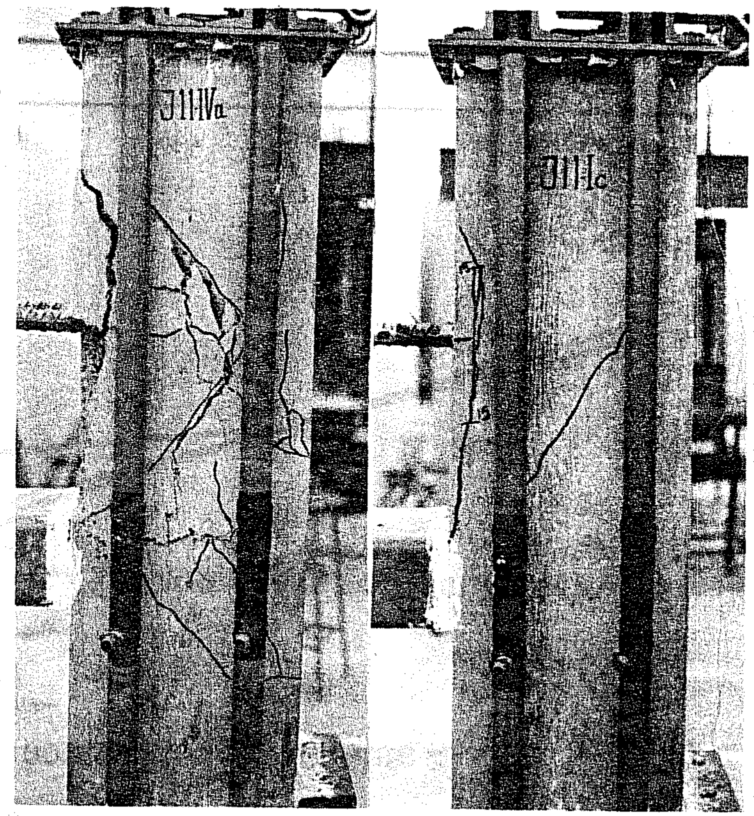


SHEAR FORCES IN CORE

d = 12.5" FOR 15" COL.

d = 9.5" FOR 12" COL.

$V_{core} = T - V_T$ OR $C - V_B$



JII-90-15-2-L JII-90-15-1-L

LOCATION OF SHEAR CRACKS

FIG. 26. SHEAR IN CORE.

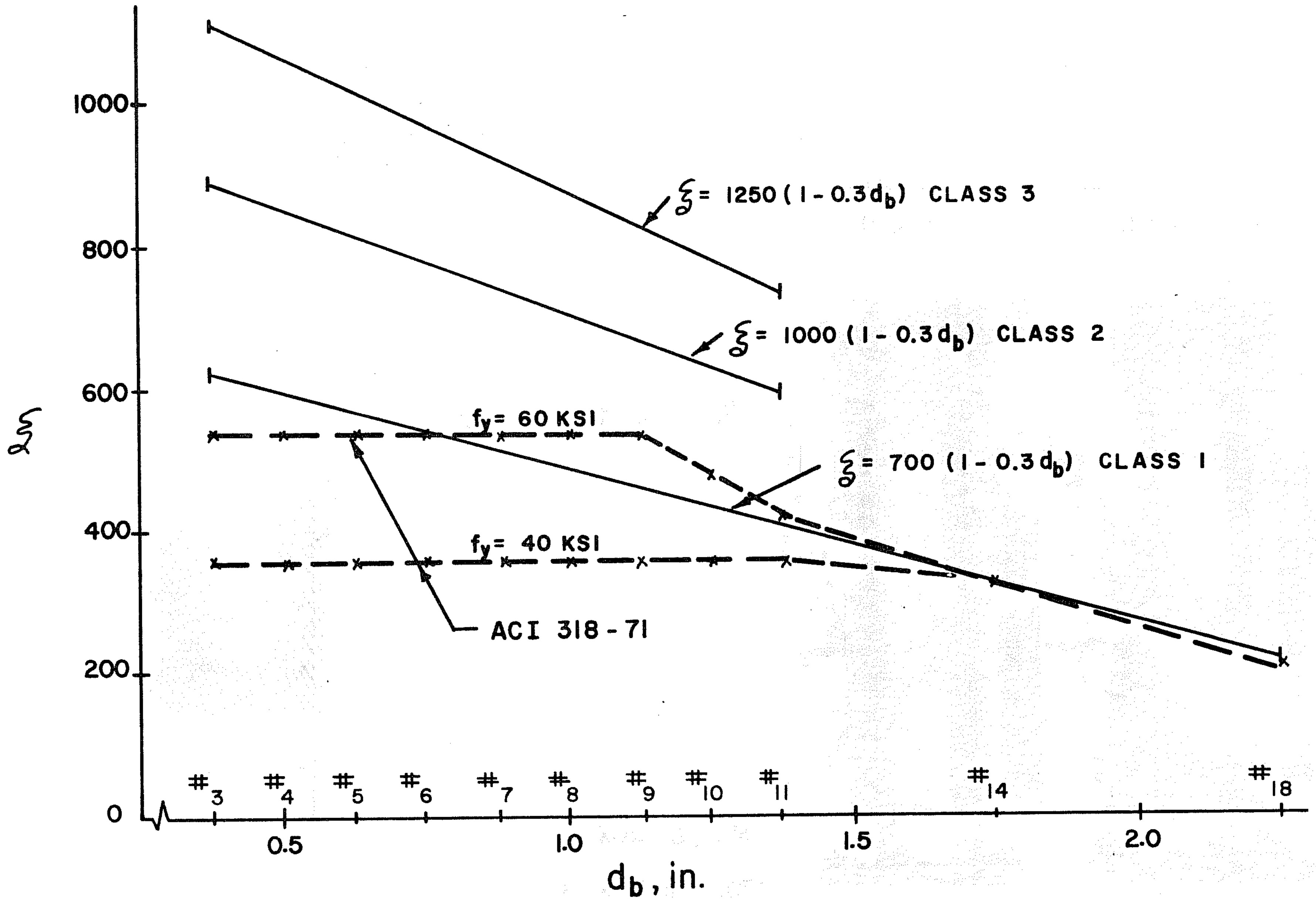


FIG. 27. PROPOSED AND CURRENT ξ VALUES FOR STRENGTH OF STANDARD HOOKS.

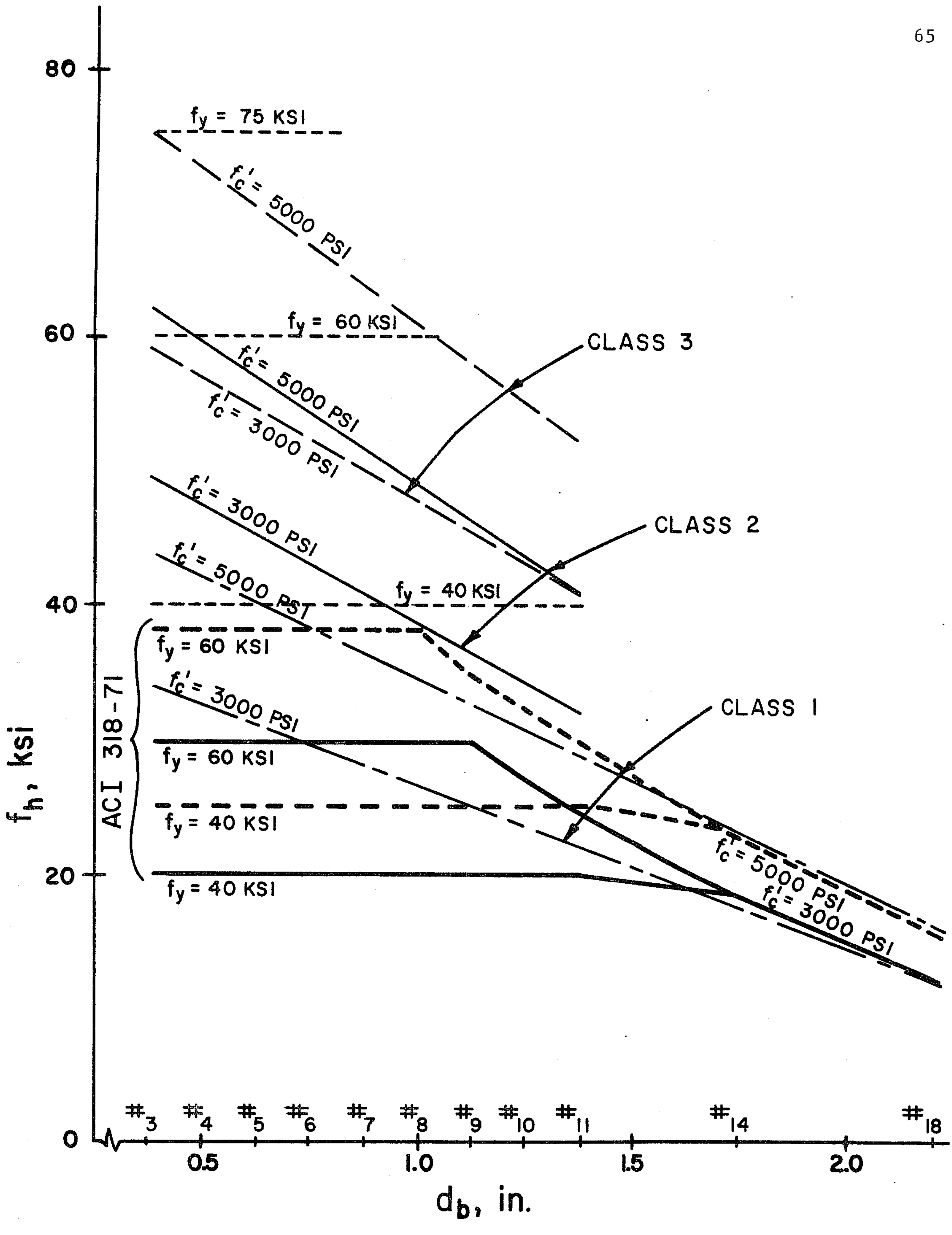


FIG. 28. PROPOSED AND CURRENT VALUES FOR STRENGTH OF STANDARD HOOKS.

APPENDIX

TEST RESULTS

A.1 Introduction

To determine the behavior of the anchored bars stress and slip were measured at various points along the bars. In the Appendix data are shown for both bars in each test specimen, thereby providing a complete record of test results.

A.2 Lead Bar Stress-Slip Curves

The slip measured at five points (shown in Fig. 6) along each bar is plotted against applied bar stresses in Figs. A.1 through A.8. The slip of the bar was measured relative to the back surface of the column. The lead bar stress refers to the stress applied to the bar protruding from the column. The bar stress at the lead end of the anchorage was obtained using measured hydraulic pressure to the rams loading the bars. Strain gages were mounted on the bars outside the column; however, the measured strains tended to be inconsistent at these points. The inconsistency was due to bending stresses on the bars produced during application of load. Even though the bars were carefully aligned in the form prior to casting and the threaded bars aligned with the reinforcing bars before splicing operations proceeded, it was virtually impossible to eliminate bending in the bars. As a result a slight discrepancy betwen applied bar stress measured using hydraulic pressure and stresses measured using strain gages was noted. The discrepancies between these values are more pronounced in the case of #7 bars. Therefore, the stresses in all cases are those obtained using measured hydraulic pressure and checked against measured strains. In Figs. A.1 through A.8, applied bar stress versus slip curves are shown for both bars anchored in the joint. In several instances loading of one of the bars was not satisfactory and the curves for only one bar are plotted. The designation of left and right bars are as viewed from the back surface of the column, that is, the face opposite to that on which the bars protrude from the column.

A.3 Stress Transfer Along the Bar

Strains were measured at three locations on all bars. The location of the strain gages is shown in Fig. 6. The gages were located to provide information regarding the stress transfer to the concrete over the straight embedment before the hook, the 90° bend portion between horizontal and vertical tangents to the hook (the 90° circular length of the hook), and the length of tail embedment (straight for a 90° hook and a quarter circle with straight tail length for 180° hooks). It should be noted that as the bars pulled out bending stresses were induced on the bars near the bend and at large slips the measured stresses may not be accurate. This discrepancy was particularly evident in the case of the #11 bars.

The measured stresses indicate that the maximum stress transferred over the straight lead embedment varied from 25 to 40 ksi for #7 bars in 15 in. columns and about 10 to 15 ksi in the 12 in. columns. In most cases the stress at the end of the curved portion, that is, at the start of the straight tail section remained very low, less than 20 ksi, until lead bar stresses in excess of 60 ksi were reached. Near failure the stresses in the #7 bar tail section increased rapidly and in some cases nearly reached yield. In the J11 series the stresses transferred to the concrete along the straight lead embedment were very small. In most tests the stress at the start of the hook was equal to the lead stress when stresses of about 30 ksi were reached. Stress at the start of the tail section of the hook generally was at a level less than half the lead bar stress.

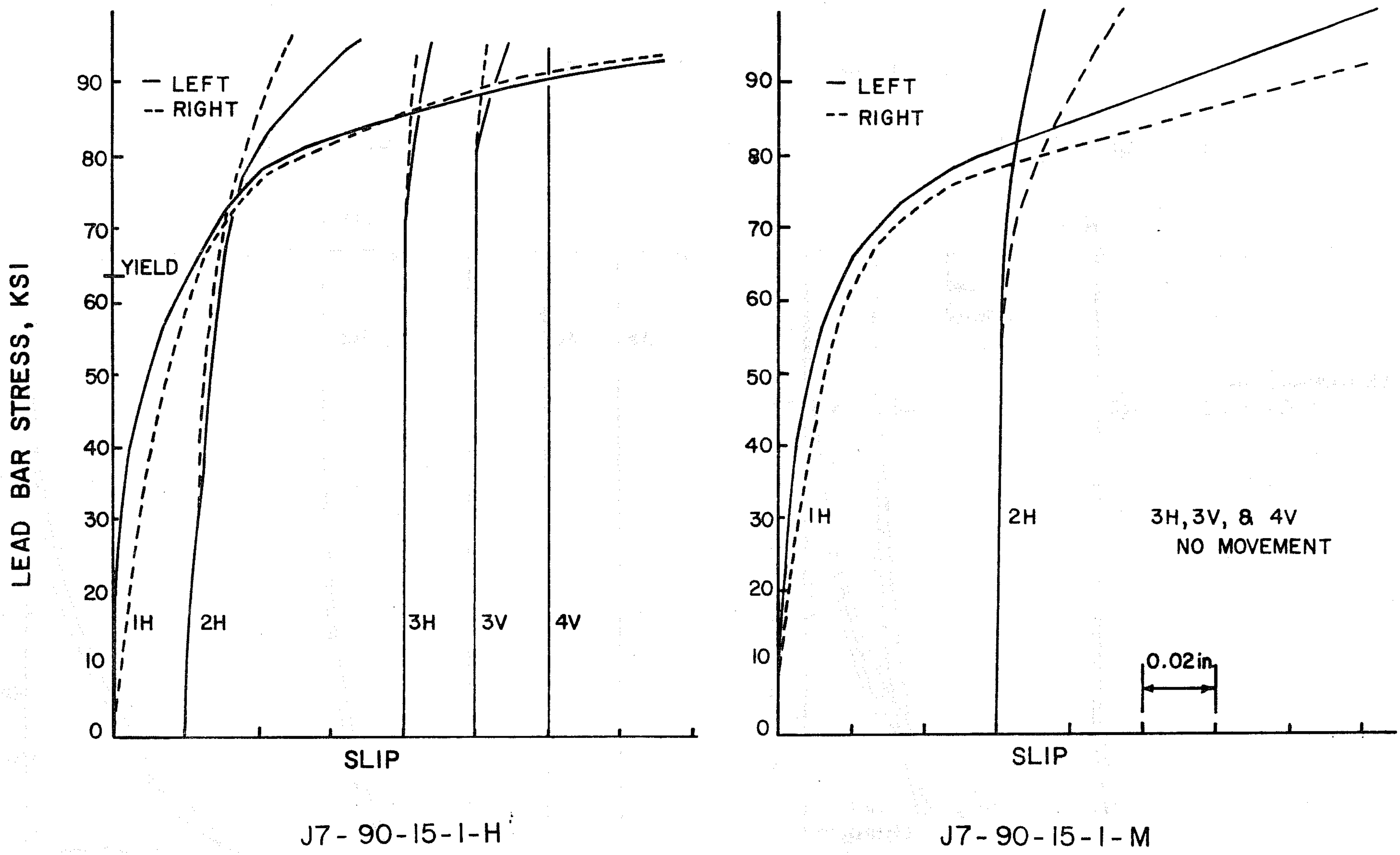
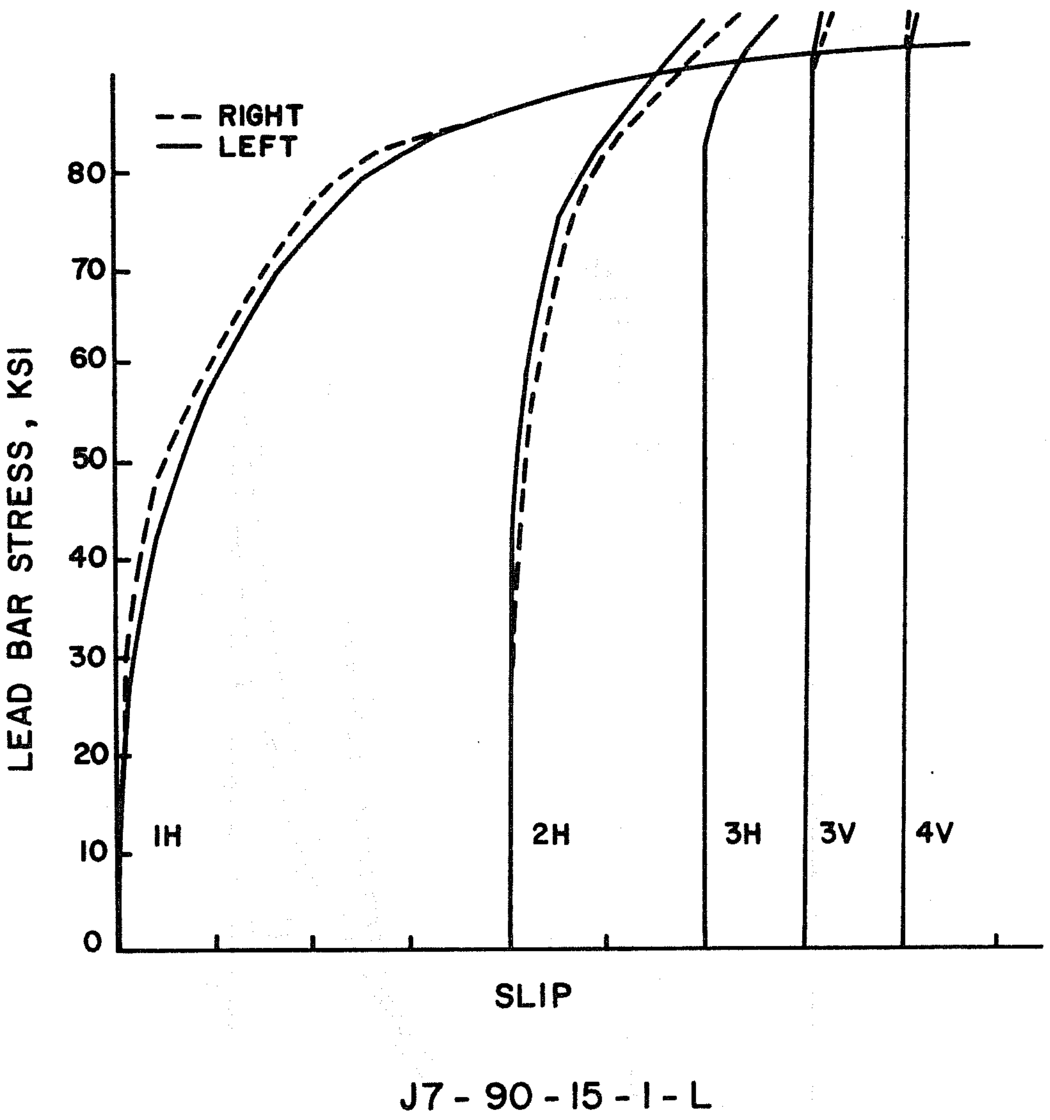


FIG. A.1. MEASURED STRESS-SLIP RELATIONSHIPS--J7 SERIES.



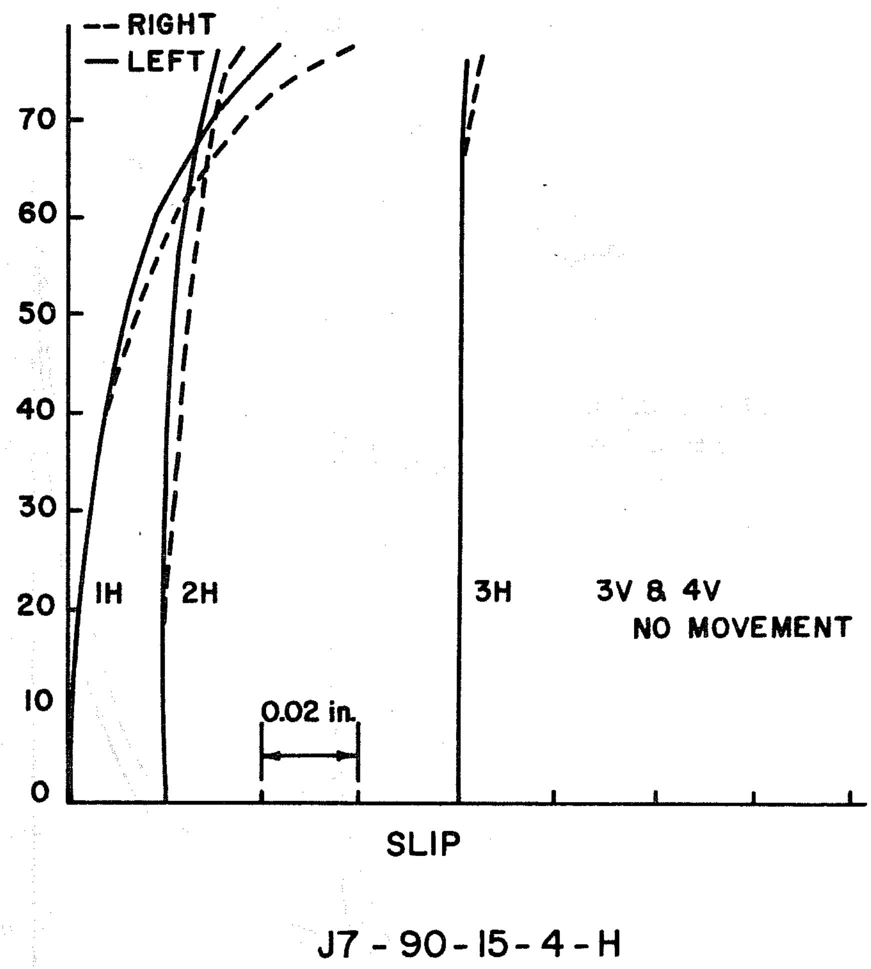
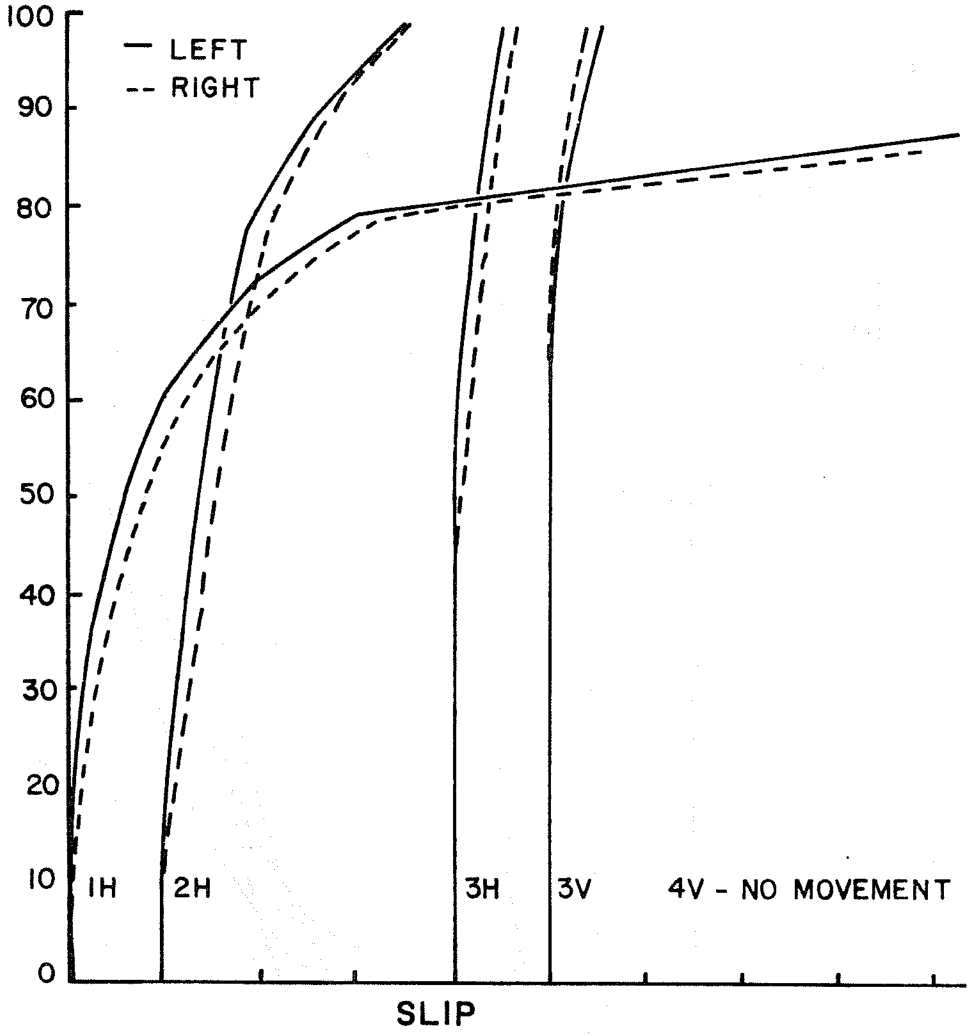
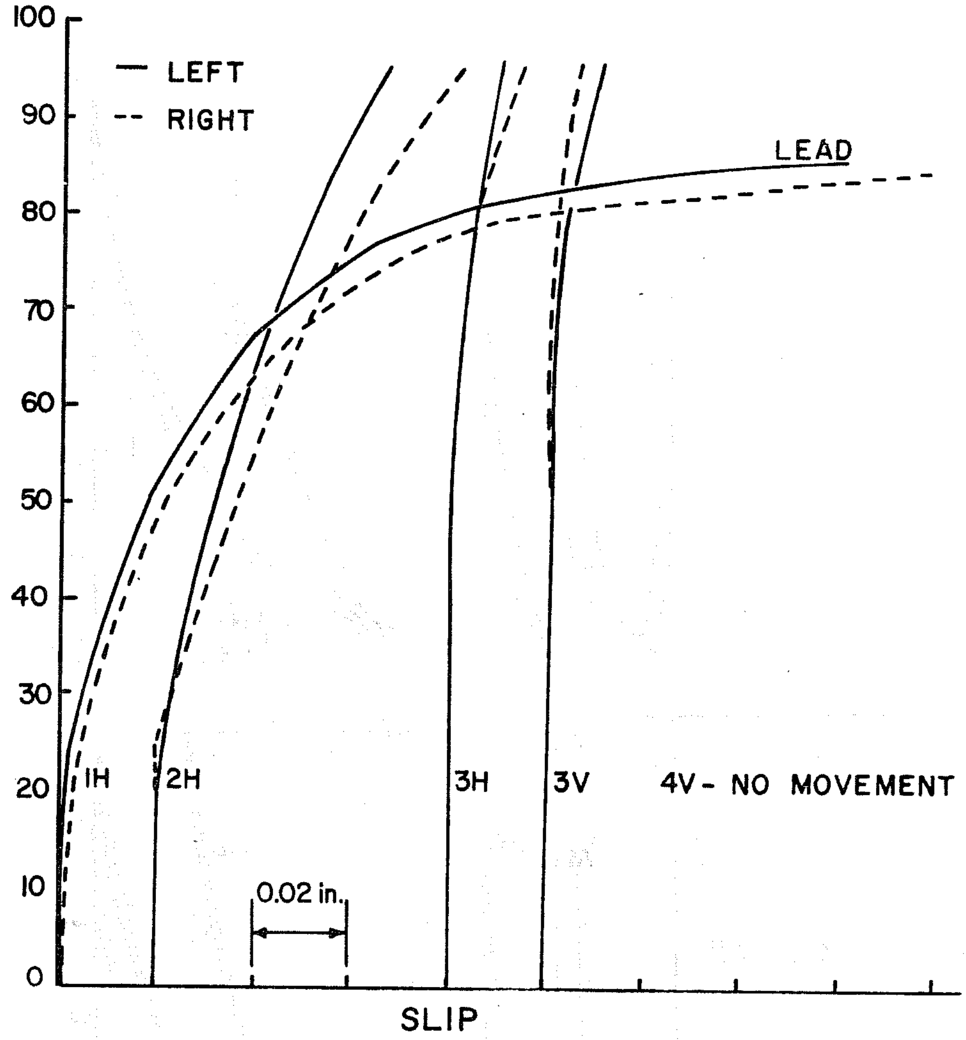


FIG. a.2. MEASURED STRESS-SLIP RELATIONSHIPS--J7 SERIES.



J7-90-15-2-H



J7-90-15-2-M

FIG. A.3. MEASURED STRESS-SLIP RELATIONSHIPS--J7 SERIES.

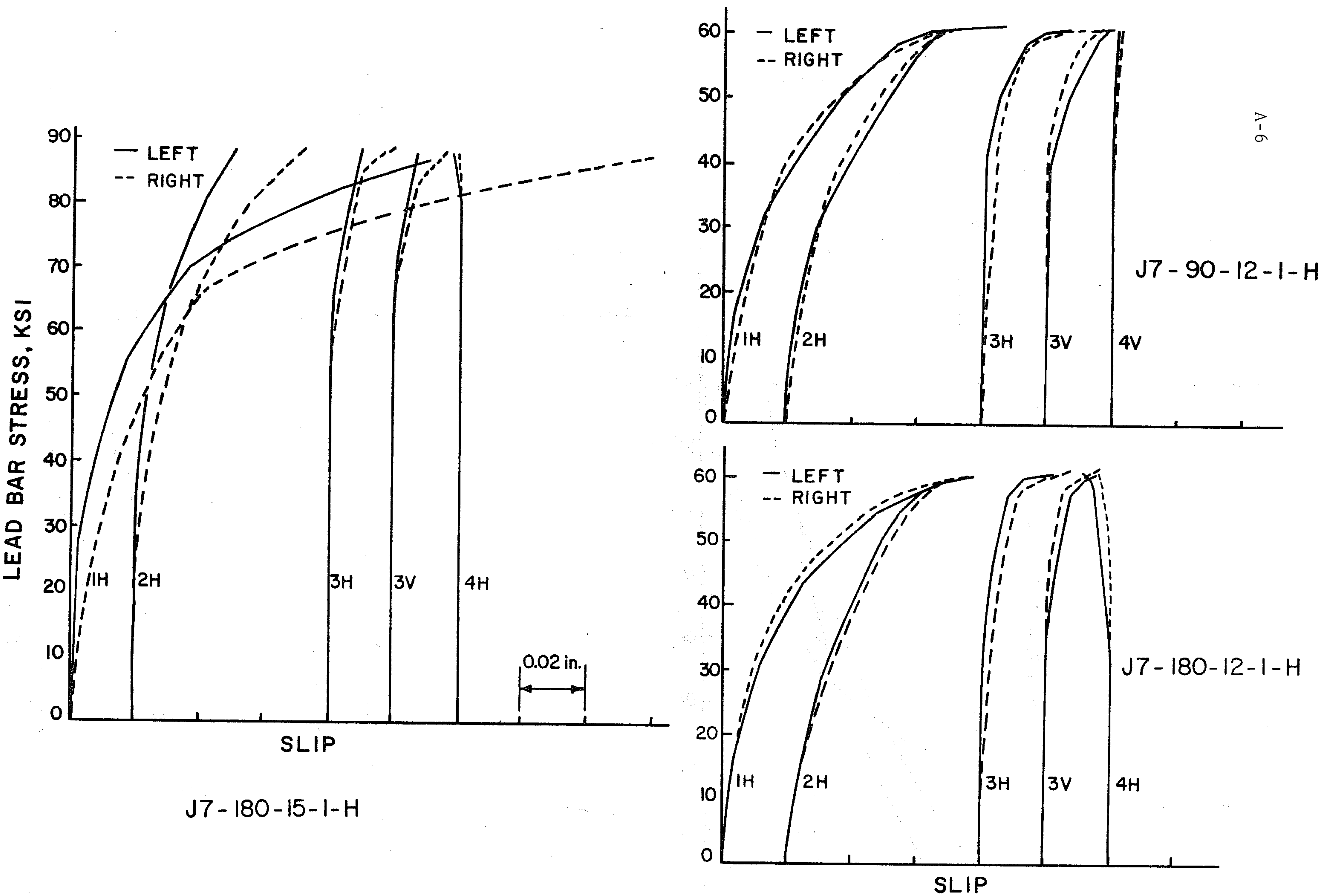
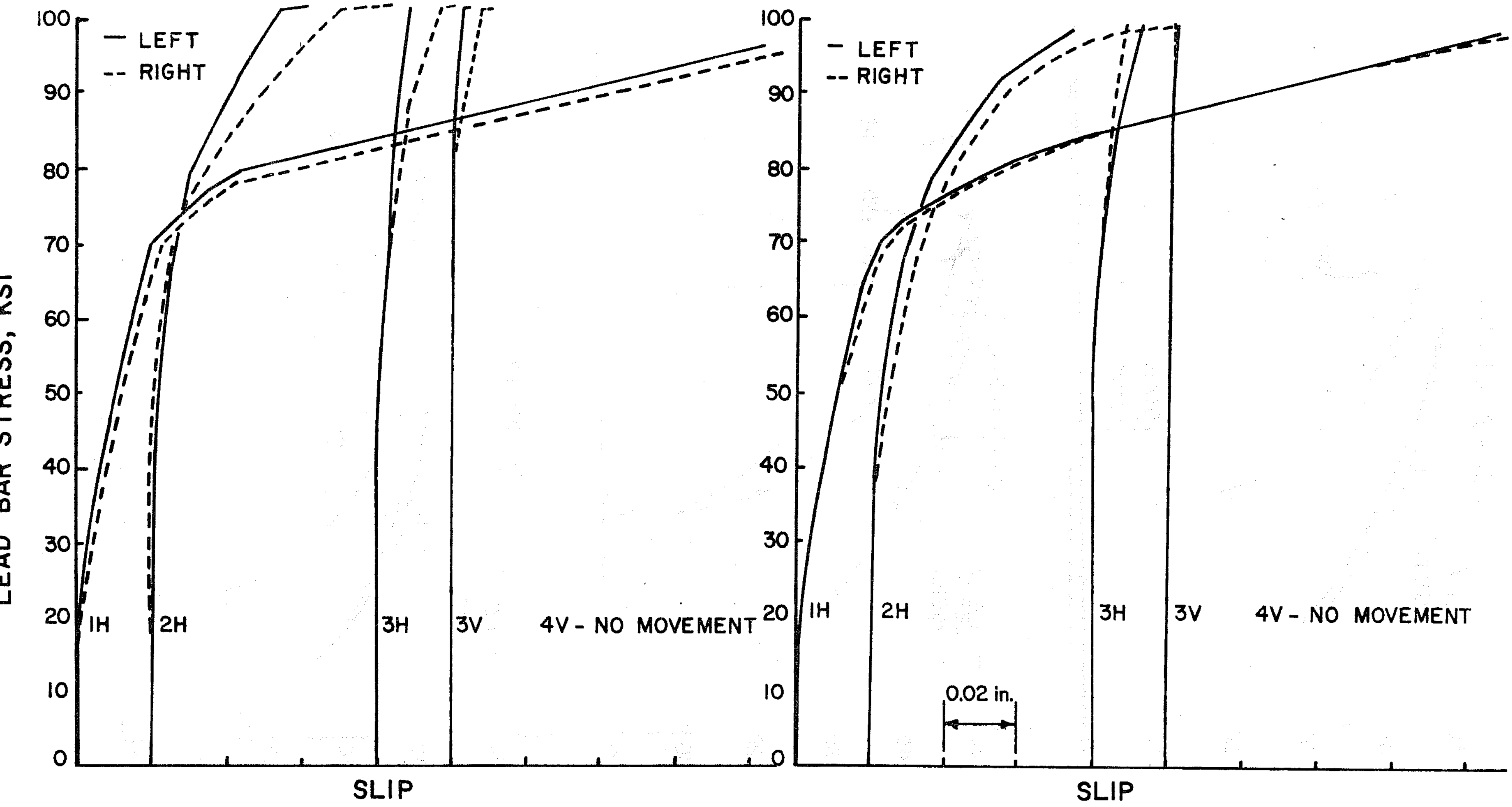


FIG. A.4. MEASURED STRESS-SLIP RELATIONSHIPS--J7 SERIES.



J7-90-15-3-H

J7-90-15-3a-H

FIG. A.5. MEASURED STRESS-SLIP RELATIONSHIPS--J7 SERIES.

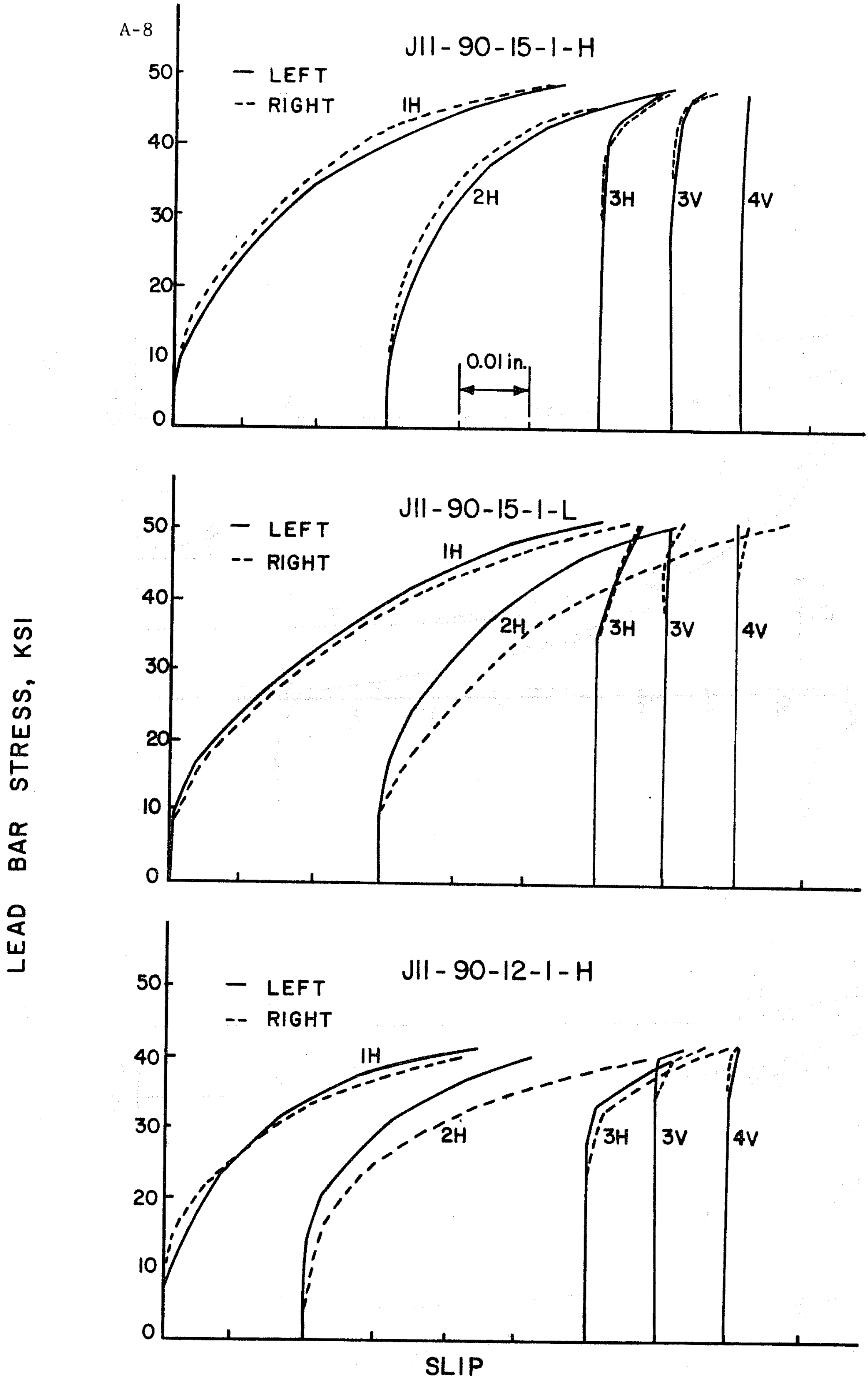


FIG. A.6. MEASURED STRESS-SLIP RELATIONSHIPS--J11 SERIES.

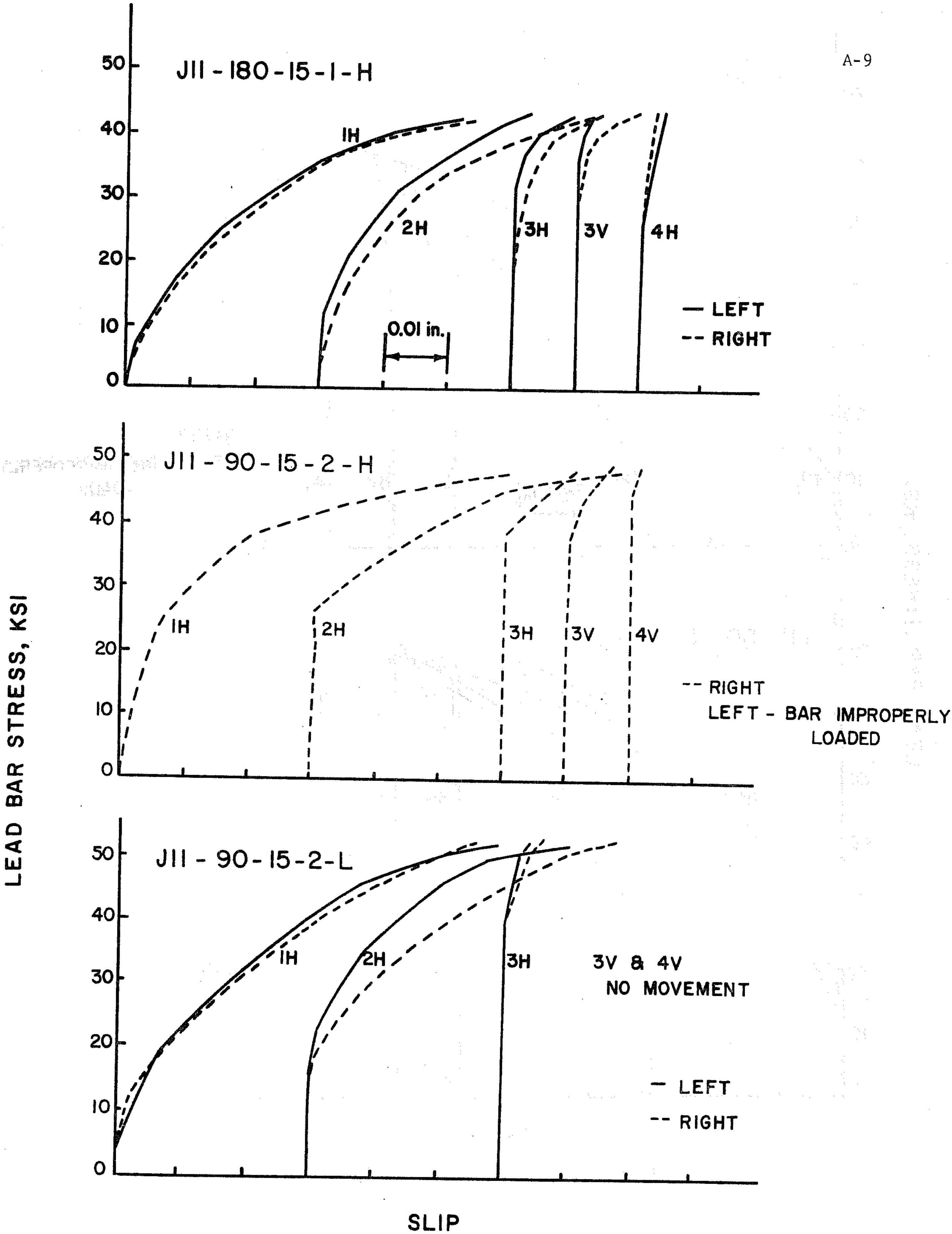


FIG. A.7. MEASURED STRESS-SLIP RELATIONSHIPS--J11 SERIES.

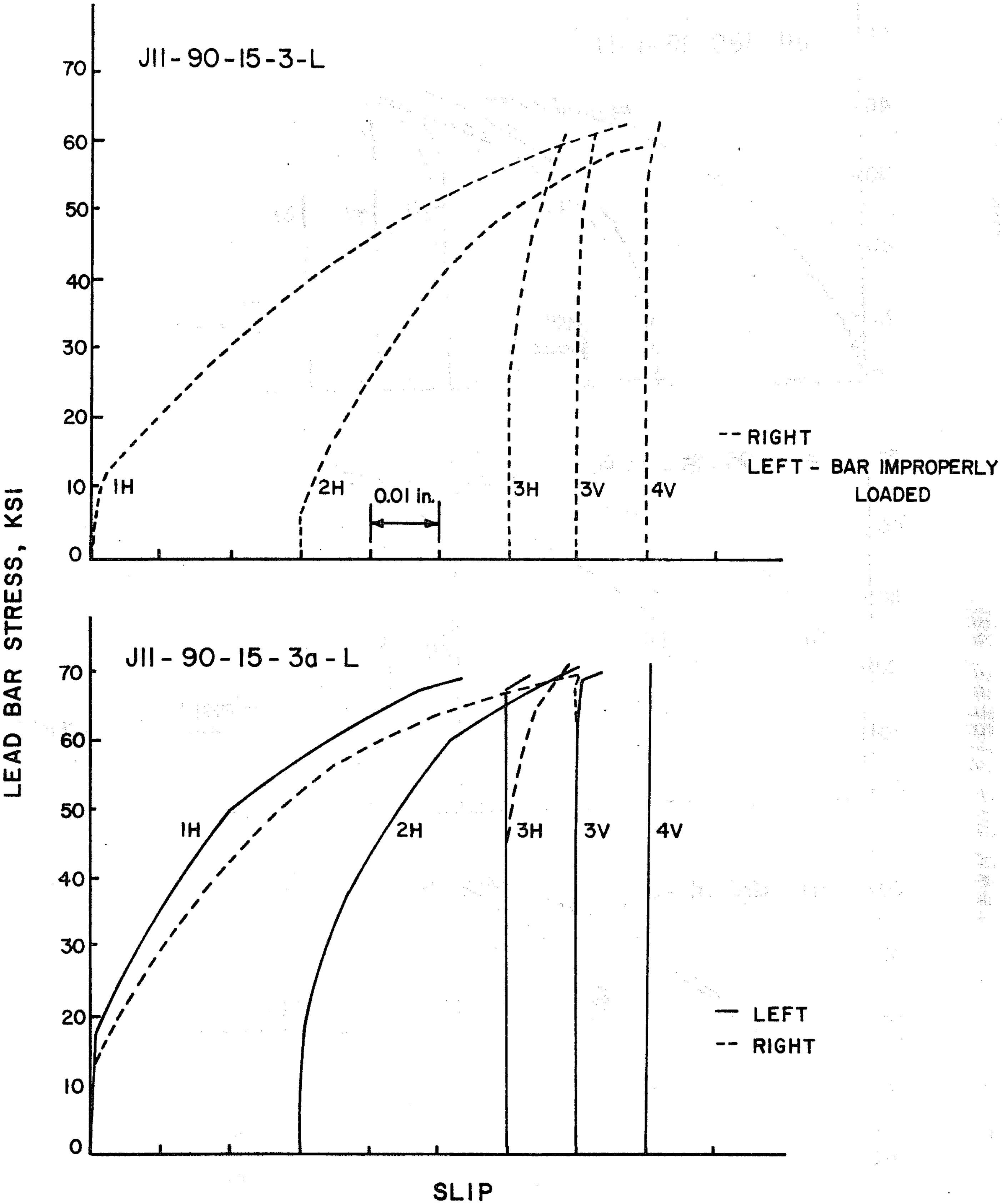


FIG. A.8. MEASURED STRESS-SLIP RELATIONSHIPS--J11 SERIES.

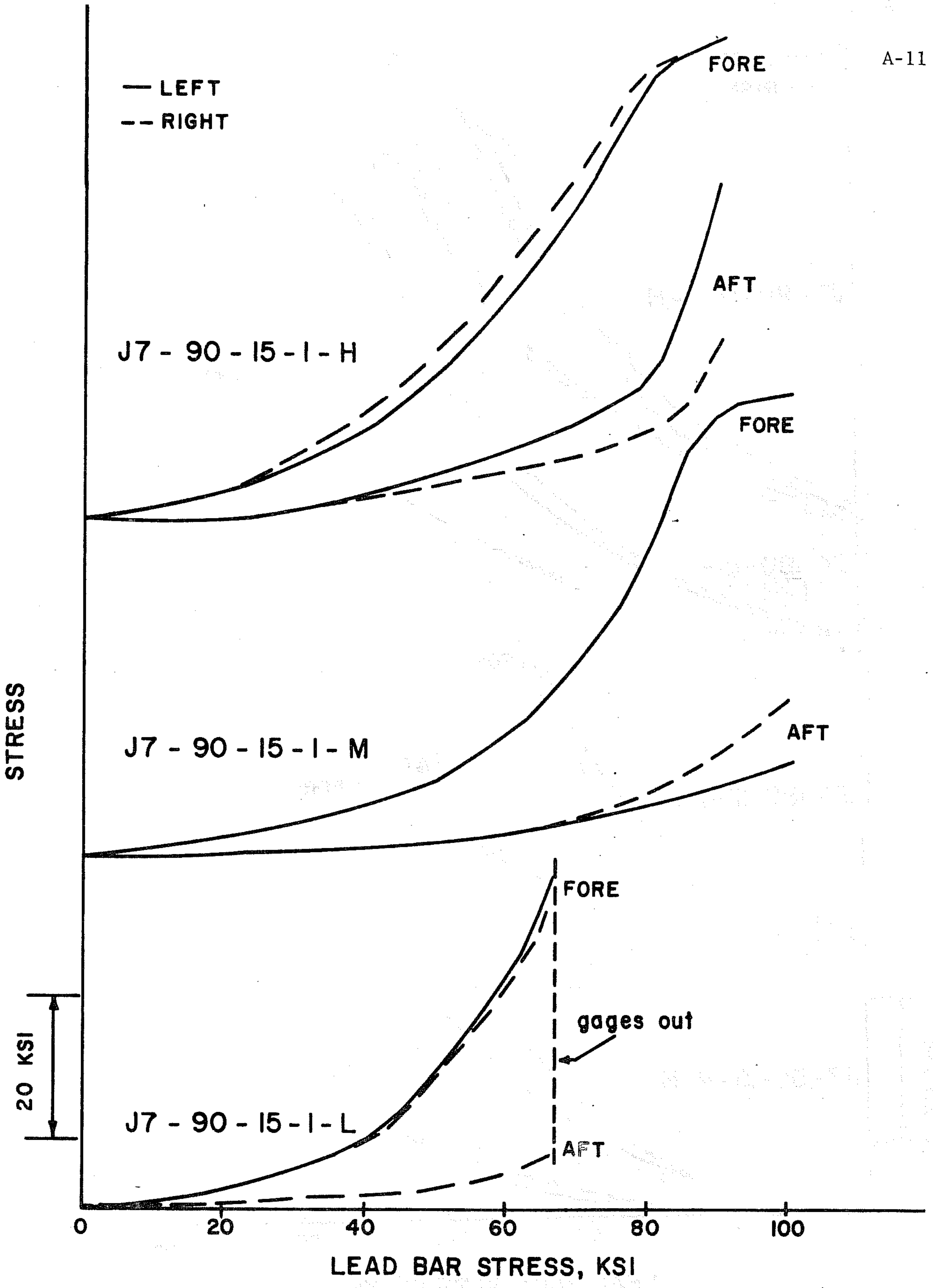


FIG. A.9. MEASURED STRESSES--J7 SERIES.

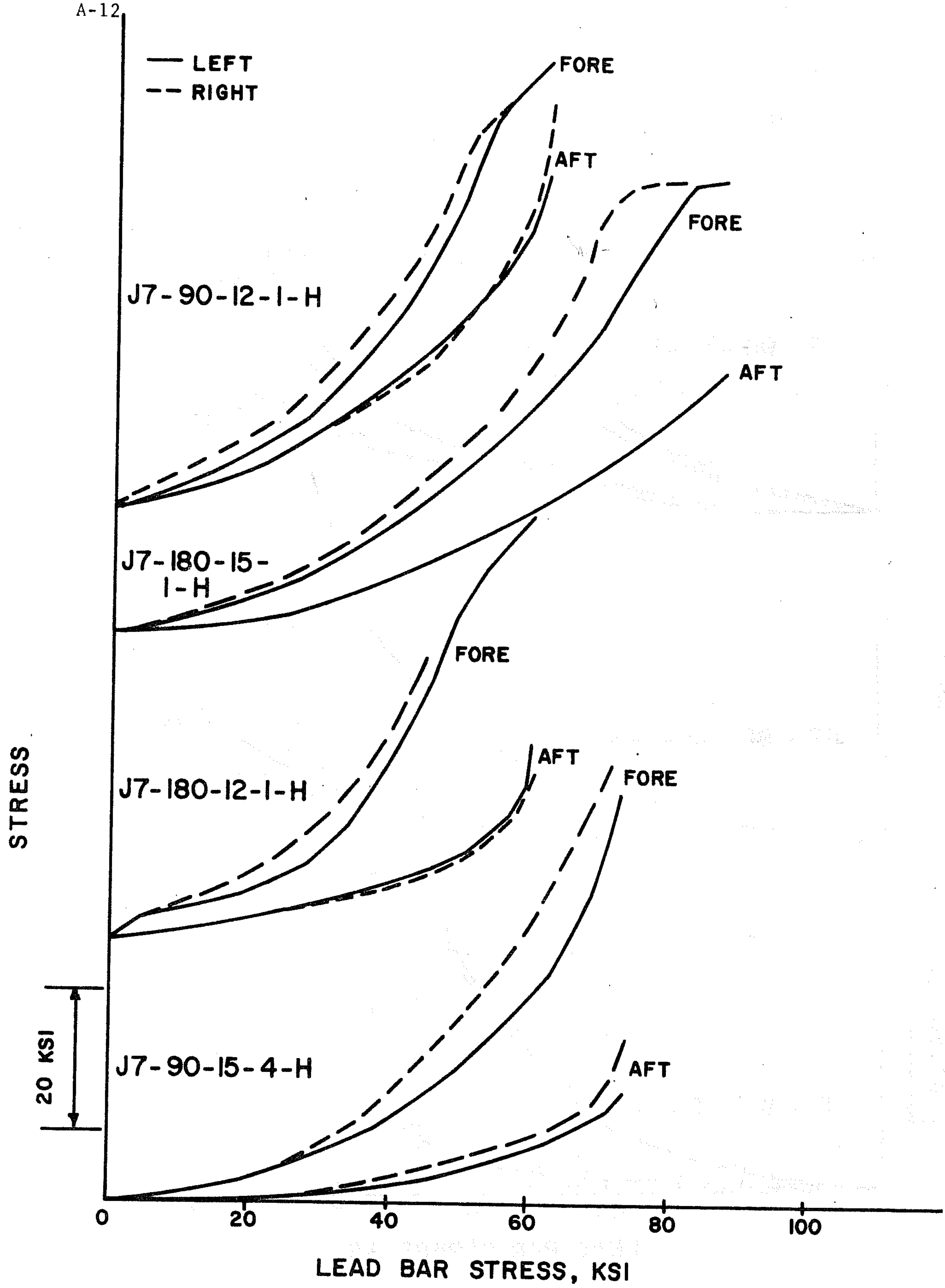


FIG. A.10. MEASURED STRESSES--J7 SERIES.

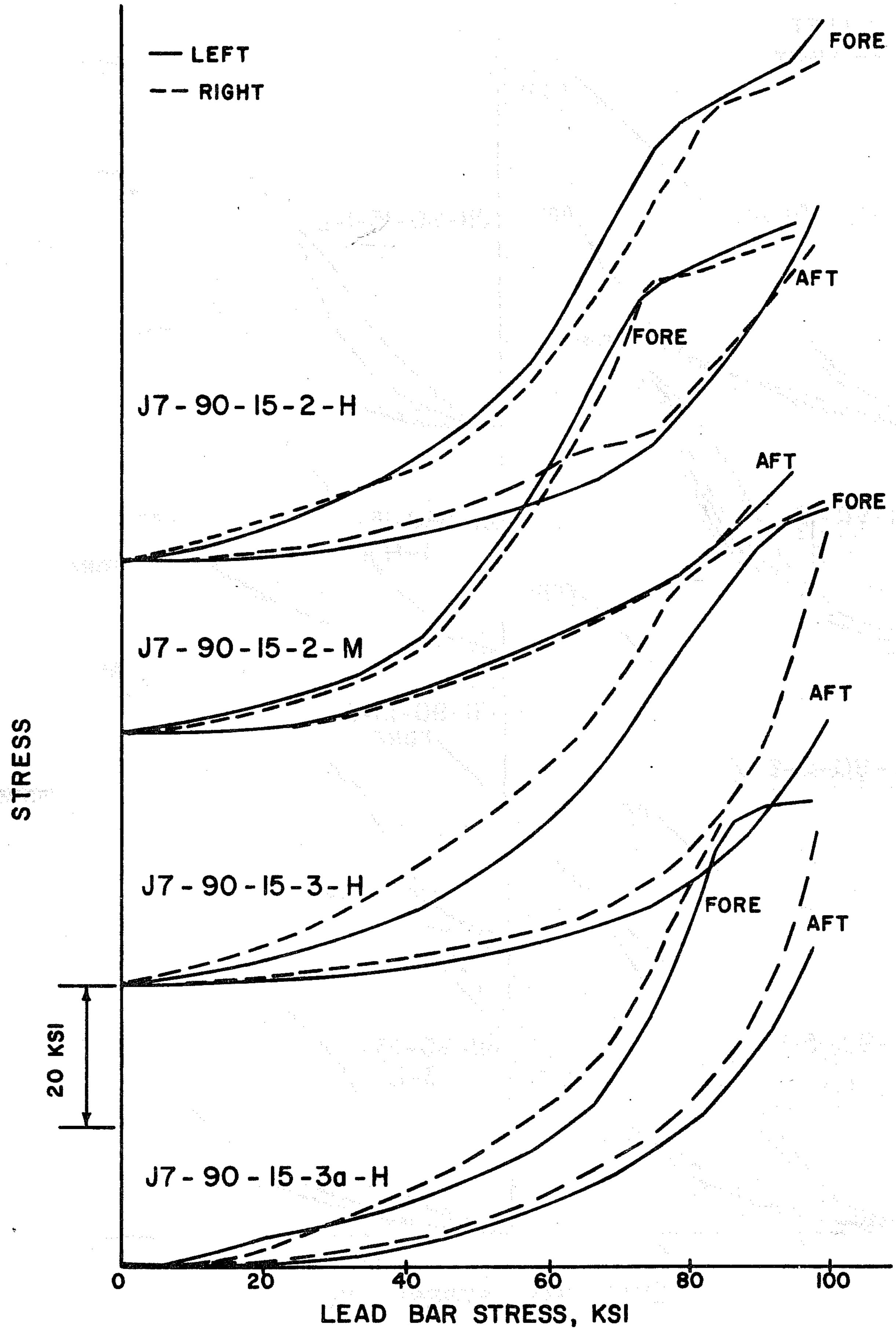


FIG. A.11. MEASURED STRESSES--J7 SERIES.

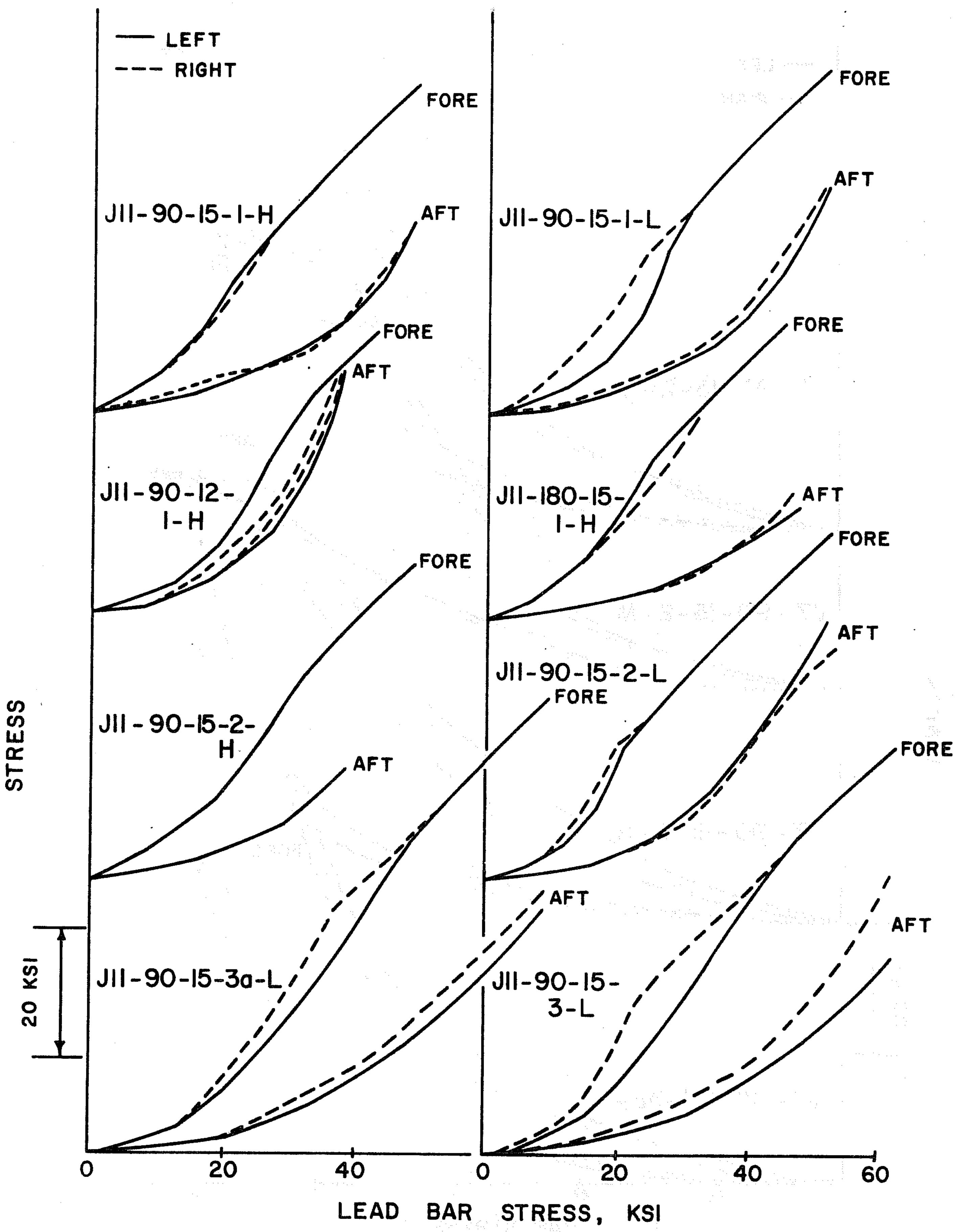


FIG. A.12. MEASURED STRESSES--J11 SERIES.