DEVELOPMENT LENGTH FOR ANCHOR BOLTS

by

John E. Breen

Final Report for Project No. 3-5-63-53
"Development Length for Anchor Bolts"

REPORT 53-1F

Conducted for

TEXAS HIGHWAY DEPARTMENT
Interagency Contract No. 4413 - 830

and

U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

by

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS
Austin, Texas

April 15, 1964

PREFACE

On June 1, 1963, a research program to study design criteria for the proper lengths for anchor bolt embedment into footings was undertaken by the University of Texas Center for Highway Research in cooperation with the Texas Highway Department and the Bureau of Public Roads. The program included a series of tests on thirty-six anchor bolts with varying embedment conditions. The anchor bolts were embedded near the edge of a square footing specimen and were under a combination of flexural tension, bond, and splitting conditions closely approximating those in the prototype footing specimens. In these tests loaded end slip behavior was determined at nominal working loads, at first yielding, and in many cases at ultimate capacity. This report presents the results of these tests and a survey of the trends indicated thereby.

ACKNOWLEDGEMENTS

The tests described herein were conducted as a part of the over-all research program of the University of Texas Center for Highway Research under the administrative direction of Dean John J. McKetta. The work was sponsored jointly by the Texas Highway Department and the Bureau of Public Roads under an interagency contract between the University of Texas and the Texas Highway Department. Liaison with the Texas Highway Department was maintained through the advisory committee for this project consisting of Mr. Wayne Henneberger, Mr. Larry G. Walker, and Mr. De Leon Hawkins, the contact representative. The study was directed by John E. Breen, Assistant Professor of Civil Engineering.

CHAPTER 1

INTRODUCTION

Object and Scope

The general objective of this investigation was the establishment of the development length or the embedment length required to develop the tensile yield capacity of ASTM A-7 anchor bolts with a yield strength of 33 ksi. At the same time it was to explore the effect of various types of end anchorages and certain other variables. A substantial amount of recent test data has been compiled on the proper development length for the ASTM A305 deformed reinforcing bars. Very little material is available concerning the length required to develop the strength of large size anchor bolts. The main information existing is a series of concentric pullout tests reported by Duff Abrams[1] which involved plain bars anchored with both nuts and nuts with washers. In this type test the concrete around the bolt is under compression. As has been shown by Ferguson, Thompson, and Turpin[2] the results of such concentric pullout tests do not reflect the bond-slip behavior of elements which are in the tension zone of beam or footing members such as the prototype sign support footings. In addition the only slips which were measured in the Abrams series were those at the free end of the embedded bolt. Tests on anchorage lengths of high strength reinforcement using a modified eccentric pullout specimen have indicated that the measurement of free end slips is very unreliable in members which have sizable water gain effects.[3]

Since the requirements of modern sign structures for the interstate highway program require a great number of such anchor bolts, it was felt desirable to obtain basic information concerning the required development length as affected by present design and construction practices. In order to closely simulate the behavior as found in typical footing structures, the test specimen shown in Fig. 1 was developed. The bolt is embedded near the edge and tension is applied through a flexural loading. In this type specimen, the anchor bolt is under a combination of tension, bond, and splitting conditions closely approximating those in the prototype footing structures.

This investigation reports on tests of twelve footing specimens of this type. In some specimens four separate anchor bolts were tested, while in others

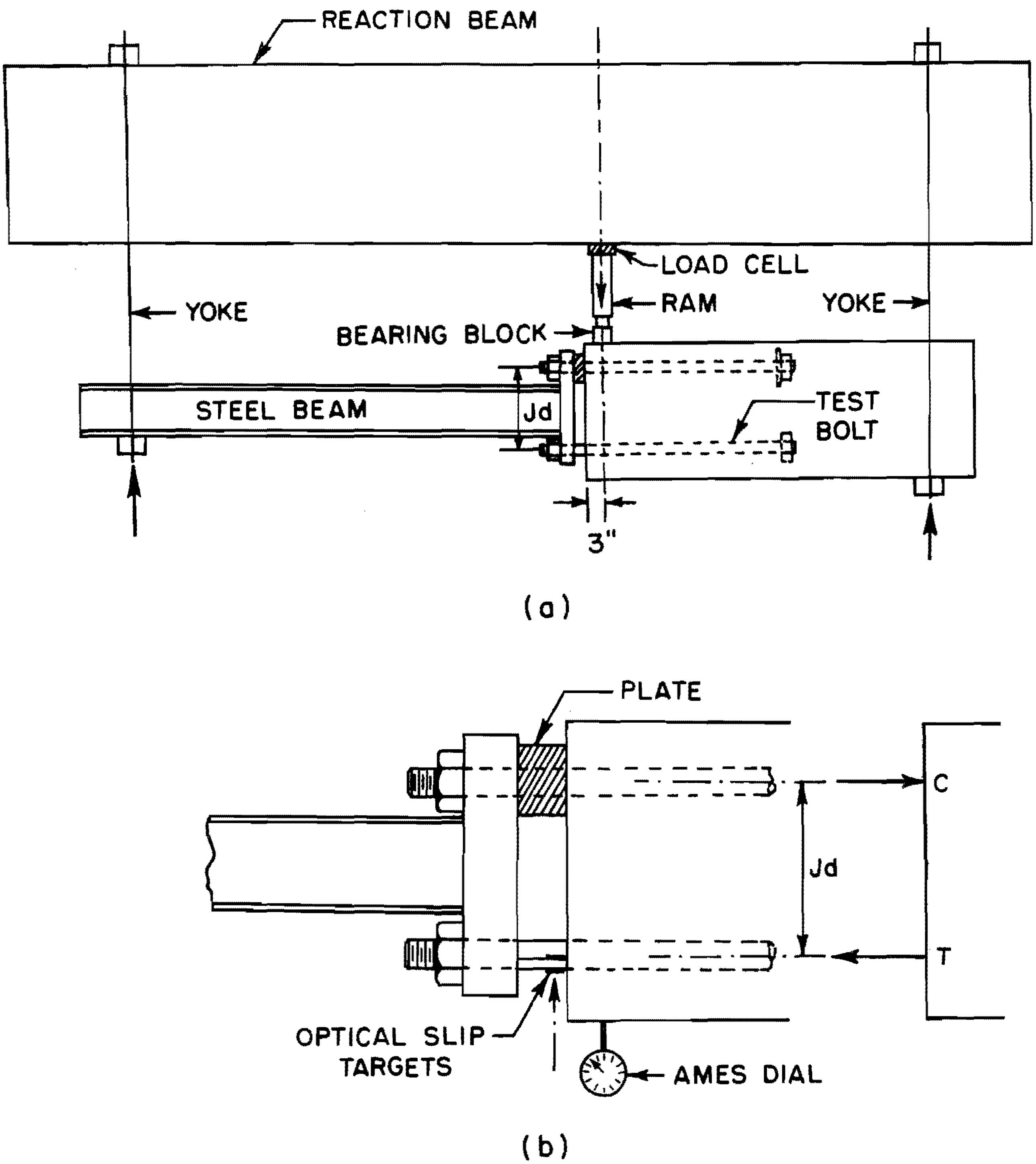


FIG. 1 TEST ARRANGEMENT

only two bolts were used. The total number of bolts involved was thirty-six. The anchor bolt sizes ranged from 1 1/4" to 3" with embedment lengths of 10 and 15 diameters. End anchorages consisted of either a standard nut or a combination of a standard nut and washer.

CHAPTER 2
TEST PROGRAM

Specimens

All of the footing specimens used in this program were of the same general type. When 1 1/4" through 2" diameter bolts were used, four anchor bolts were embedded in each footing specimen. However, only two bolts were used in each specimen with the 2 1/2" and 3" diameter anchor bolts. The individual anchor bolt specimens are designated by a four part code symbol. The first number represents the nominal bolt diameter. This is followed by the letter N for a specimen with a nut anchorage only, W for a specimen with a nut and washer anchorage and S for the series with special type of end anchorage. This letter is followed by a number which is the embedment length in terms of number of bar diameters. The final letter indicates the designation of individual companion specimens. For example, the designation 1-1/4 W 10a designates a specimen in which a 1-1/4" bolt with a nut and washer anchorage was embedded for 10 diameters in a footing specimen. The a indicates that it was the first of this type tested. In a few cases this code designation is followed by a -2. This indicates a retest to failure of a specimen which was taken up only to yield in the initial testing.

Figs. 2 through 4 show the entire set of test specimens with critical dimensions. A tabulation of the important variables and a summary of the test results is provided in appendix Table A. The specimens have been grouped according to bolt diameter and have been subdivided in these groups according to the type of end anchorage.

Materials

Concrete. In all test specimens a concrete conforming to Texas Highway Department Standard Specifications for Road and Bridge Construction "Class A" concrete was used.[4] The 1-1/4" and 1-1/2" specimens were cast with a job mixed concrete. The 1-3/4" through 3" specimens were cast with a ready mixed concrete procured from a local source. The cement used in all test specimens was a high-early strength cement (type III). The fine aggregate met the specifications for aggregate grade no. 1 of Section 421.2.4. The coarse aggregate met the requirements of aggregate grade no. 3 of Section 421.2.3. The maximum

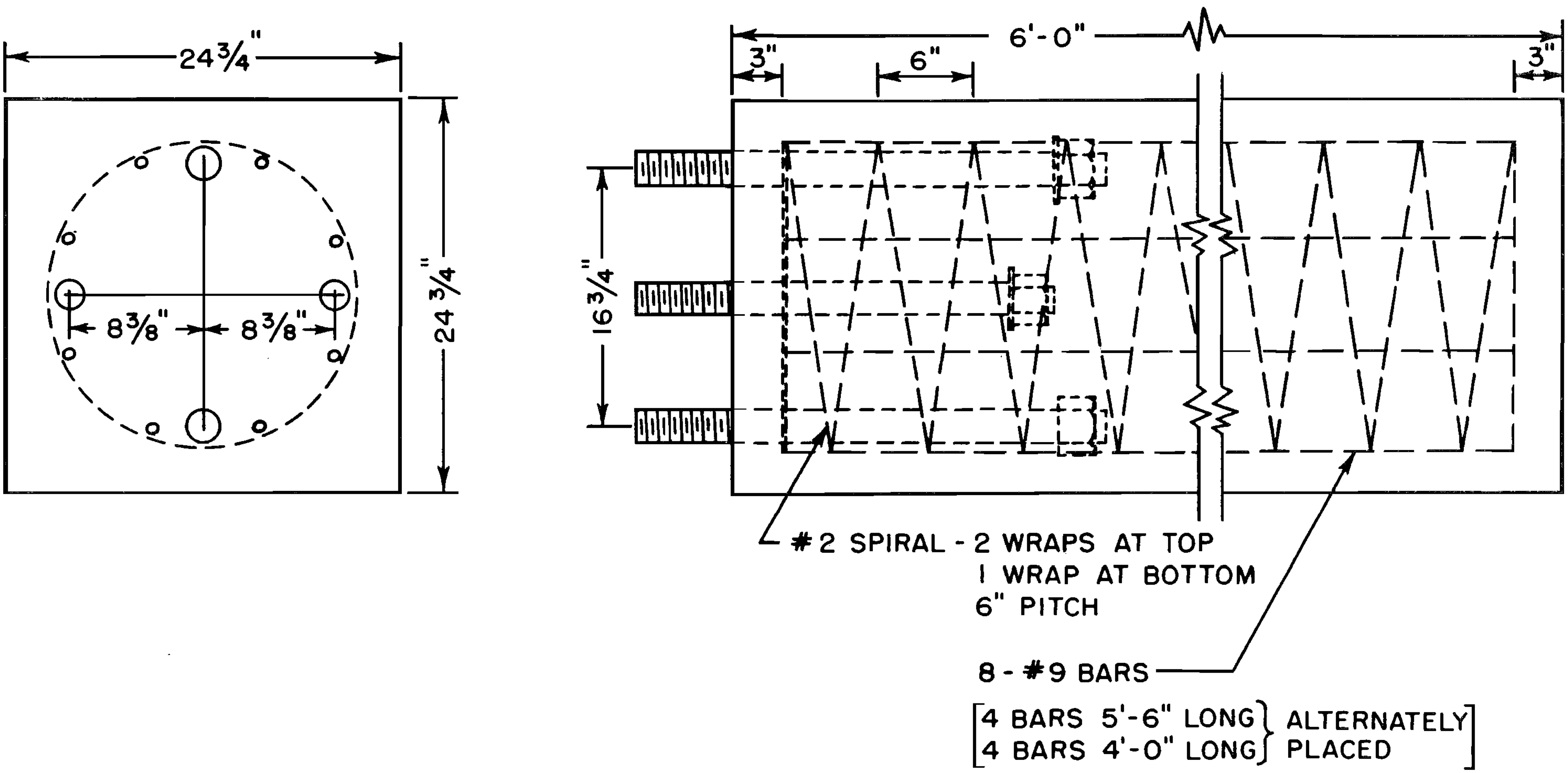
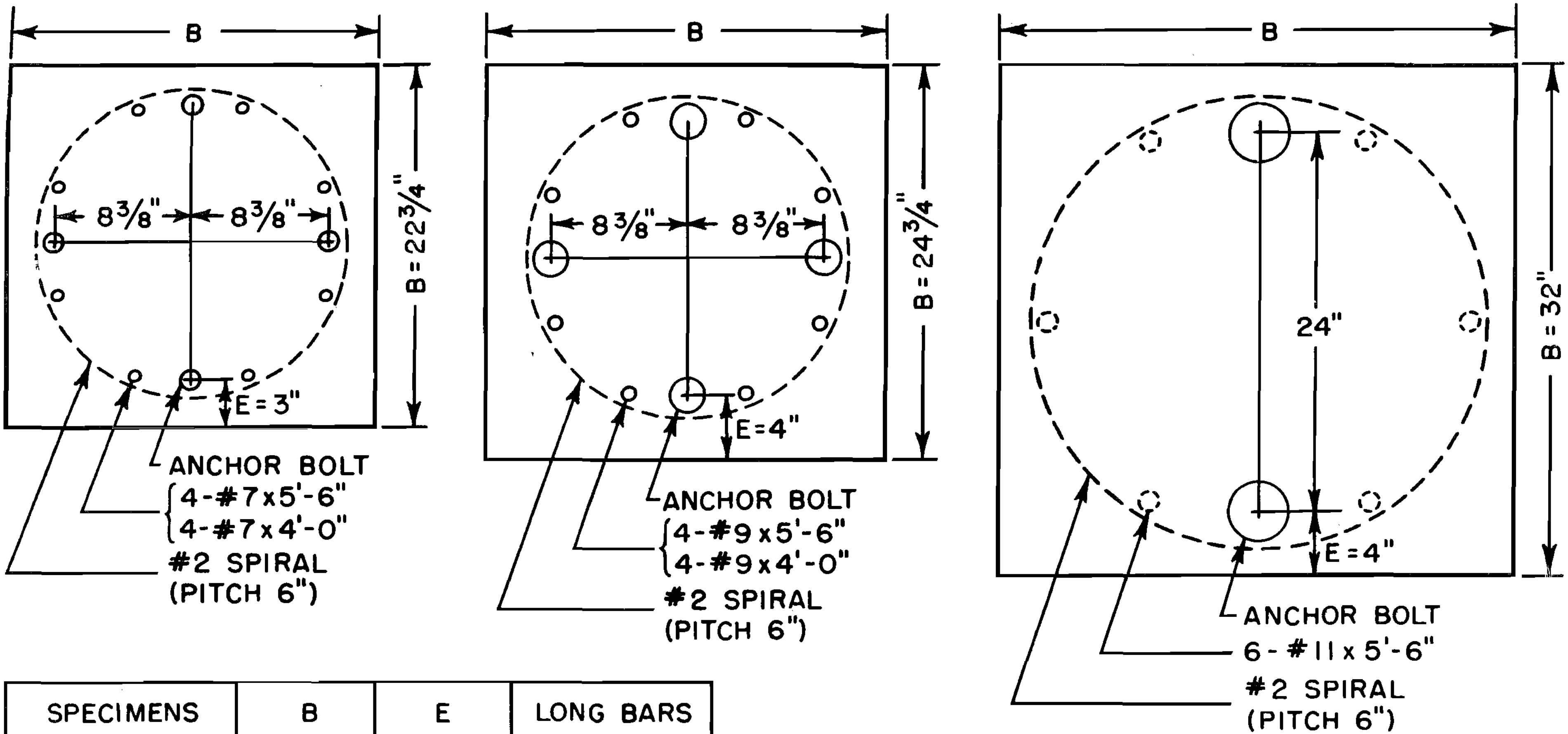
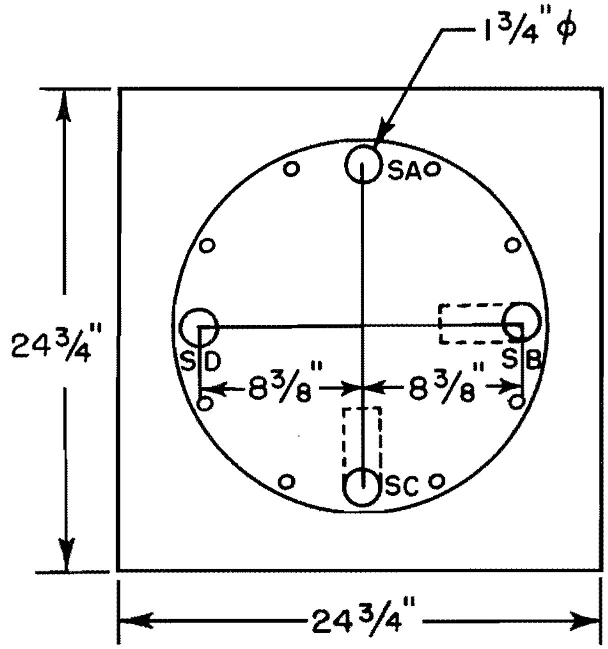


FIG. 2 TYPICAL SPECIMEN DETAIL



SPECIMENS	B	E	LONG BARS
ALL 1¼", 1½"	22¾"	3"	8-#7
ALL 1¾", 2"	24¾"	4"	8-#9
ALL 2½"	32"	4"	8-#9
ALL 3"	32"	4"	6-#11

FIG. 3 SPECIMEN DETAILS



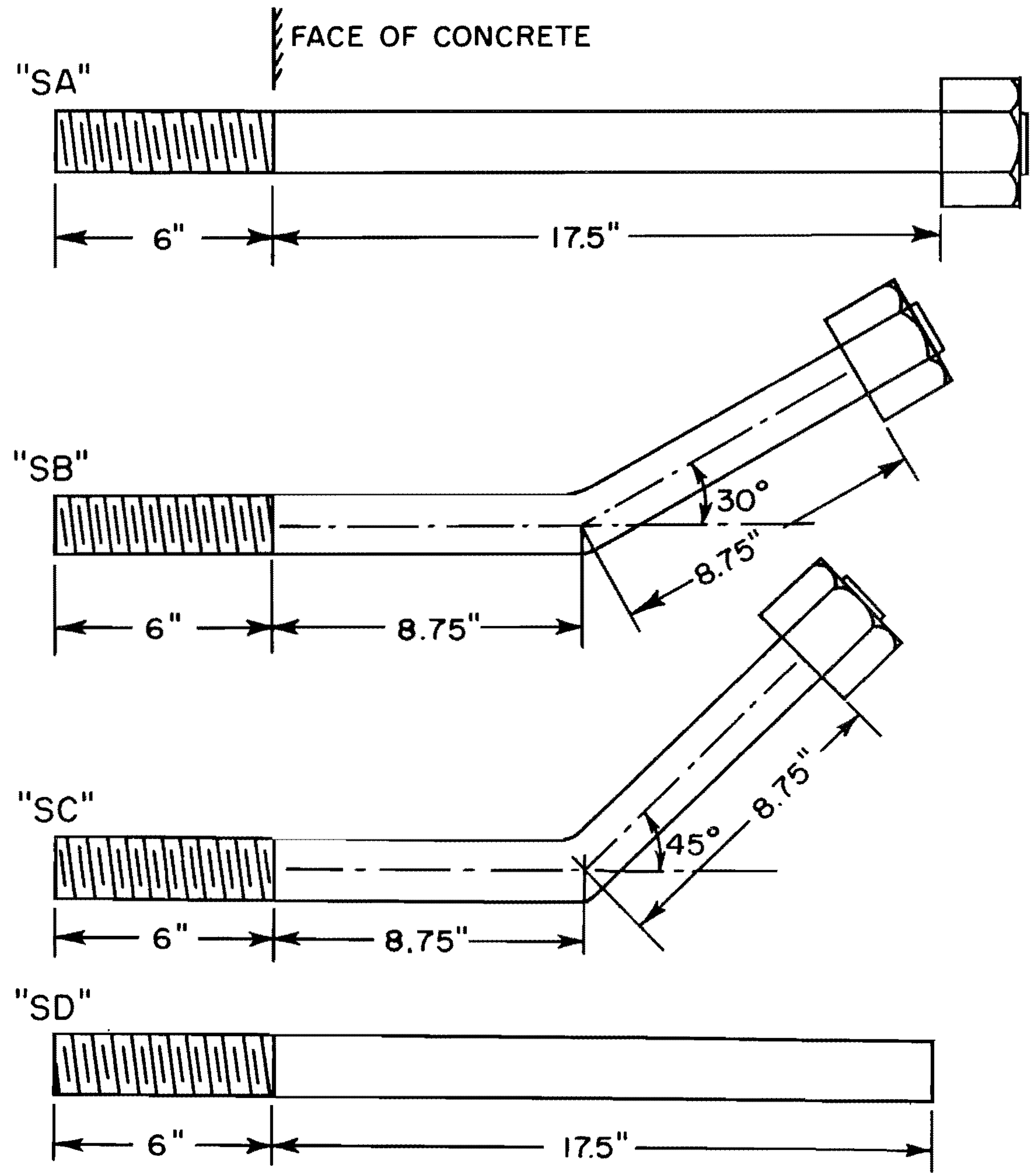


FIG. 4 DETAILS OF SPECIAL ANCHORAGE

SERIES "S"

size was 1". The cement factor was five sacks per cubic yard and the slump range was 3" to 4". In general, the water—cement ratio was seven gallons per sack. The water used was Austin city water with no admixtures added. When ready mixed concrete was used inspectors were provided at the plant to insure proper batching and mixing procedures.

Anchor Bolts. The anchor bolts used were specified to be manufactured from ASTM A7 steel with a minimum yield strength of 33 ksi. In all cases the bolts were fabricated in local commercial machine shops from round bar stock. In one set of 1-3/4" bolts, the companion specimen indicated that a high carbon or other alloy steel was substituted and the yield point was substantially above that required. Appendix Figs. A1 and A2 show typical stress strain curves for the bolts used. The threaded sections were machined to conform to USS thread specifications. The anchor bolts were not galvanized.

Reinforcing Steel. The reinforcing steel used was deformed (ASTM A305) bars meeting ASTM A15 intermediate grade specifications. The spirals were wound and furnished by a commercial supplier using number 2 (not deformed) bars of ASTM A15 intermediate grade steel.

Forms. All the footing specimens were cast in wooden forms as shown in Fig. 5. The bolts were cast in a vertical position at the top of the forms. In all specimens the height of the footing specimen was maintained at 6 feet. This insured that the water gain effect would remain relatively constant in all tests and would closely approximate that found under field conditions. The form joints were taped inside and the forms were wiped with form oil prior to each use.

Reinforcing Cage. Reinforcing steel was saw cut to length and the spirals were procured wound to the proper diameter. The main longitudinal reinforcement was arranged in the proper configuration inside the spirals and then the whole was tied with wire into a stiff cage which could be set directly into the form. After the reinforcing cage was in place in the form, the anchor bolts were set into position. In order to position the bolts properly a set of cross members was used on top of the forms. These cross members had holes of exactly the same diameter as the bolts drilled in the proper position. The bolts were then positioned and held firmly in place with a nut above and below the cross member. This insured that the anchor bolt would be correctly positioned in the specimen after casting.

Casting Procedure. Concrete was placed in the forms using a standard

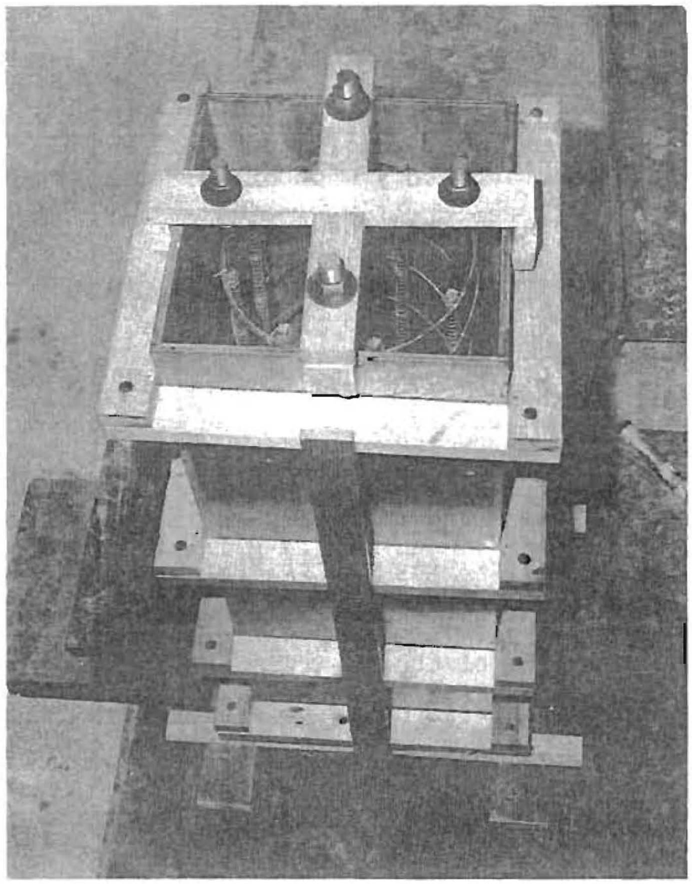


Fig. 5. Specimen prior to concrete placement

concrete bucket and an overhead traveling crane. The concrete was vibrated into place with a large internal vibrator. Standard compression test cylinders and modulus of rupture beams were cast from each of the mix batches. In general the concrete samples were taken from the portion of the batch which was placed in the top half of the footing specimen. This was the portion of the concrete surrounding the test bolts. Because of the slumps used good compaction was obtained.

Curing. The specimens were trowled to a smooth finish and then left overnight in the forms in the laboratory without any special protection other than a piece of moist burlap placed over the open end of the form. In general this burlap covering was kept in place until the sixth day. Then the curing cover and wooden side forms were removed and the specimen was moved into position and prepared for testing. Wire rope slings were utilized for handling the specimens. At no time was the specimen lifted by using the embedded anchor bolts.

Test Procedure. While the general procedure used in testing all of the bolts was identical, it was found necessary to increase the capacities of some of the elements of the testing rig to handle the 2-1/2" and 3" diameter bolts. However, this was mainly a scaling operation. No distinction will be made between the two sets of equipment in the text or figures other than to note that no monitor load cell could be used in the 2-1/2" and 3" diameter bolt tests since the ranges of loads applied were beyond the capacities of available load cells.

On the day of test a calibrated hydraulic ram (A) and an electronic load cell (B) were inserted between the specimen and a reaction beam as shown in Fig. 6a. The ram load was distributed to the specimen through a thick steel bearing plate. (C) This plate was plastered onto the specimen to insure an even bearing surface. Then a specially fabricated steel wide-flange section (D) was attached to the footing specimen as shown in Fig. 6b. Reaction straps (E,F) were positioned to yoke the test assembly to the heavy reaction beam.(G) It can be seen in Fig. 6c that the footing specimen was mounted on large rollers. In addition it should be noted that a gap was left between the heavy plate on the end of the wide-flange and the concrete surface around the tension anchor bolt. This was to permit visual observation of the loaded end slip. In Fig. 6d the mounting on the compression face of the footing specimen can be seen. A heavy plate assembly was provided to concentrate the compression force over a relatively shallow but wide area. This insured an accurate determination of the location

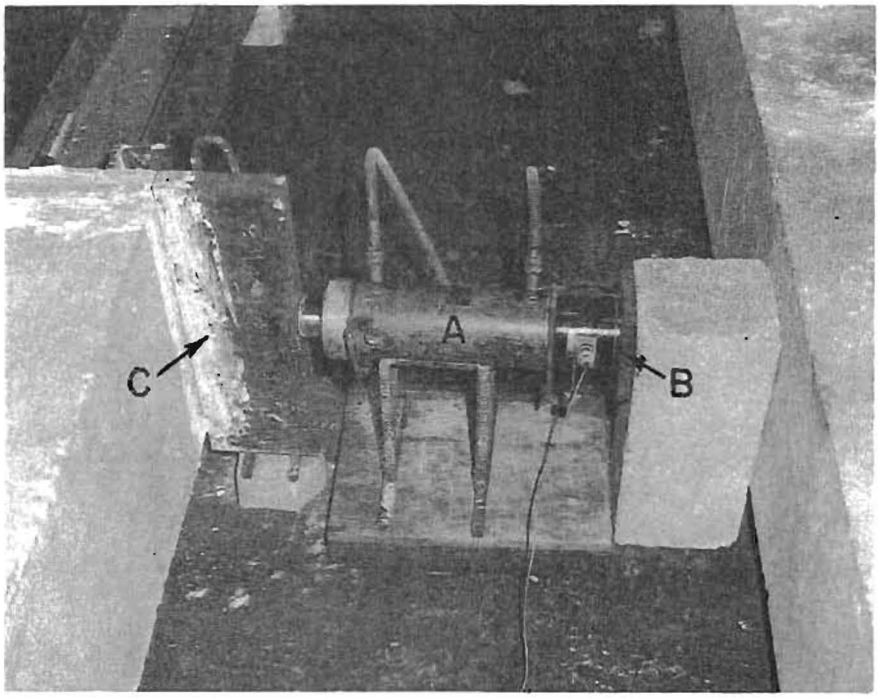


Fig. 6a. Loading system

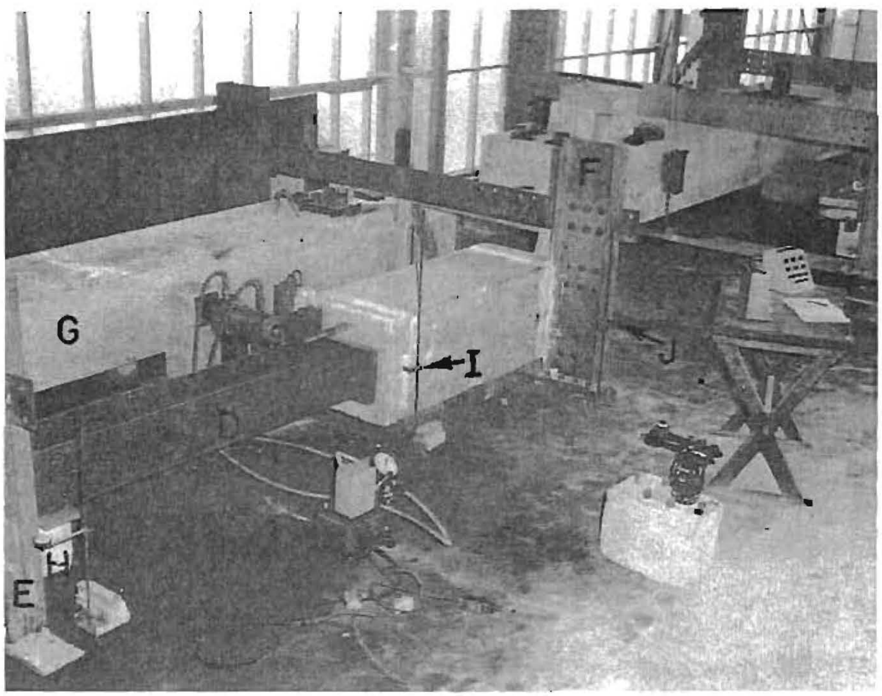


Fig. 6b. Loading arrangement

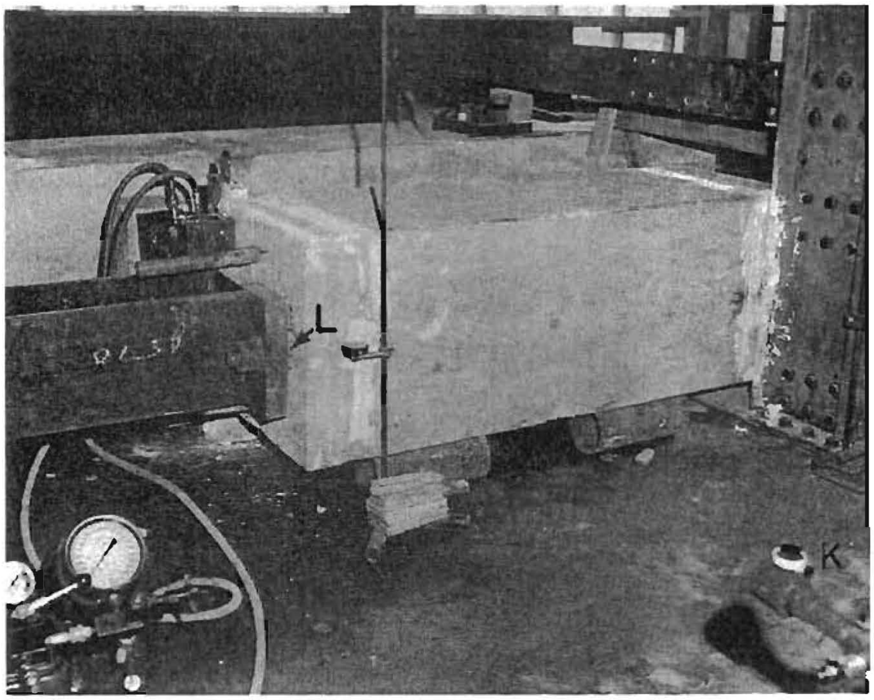


Fig. 6c. Slip measurement system

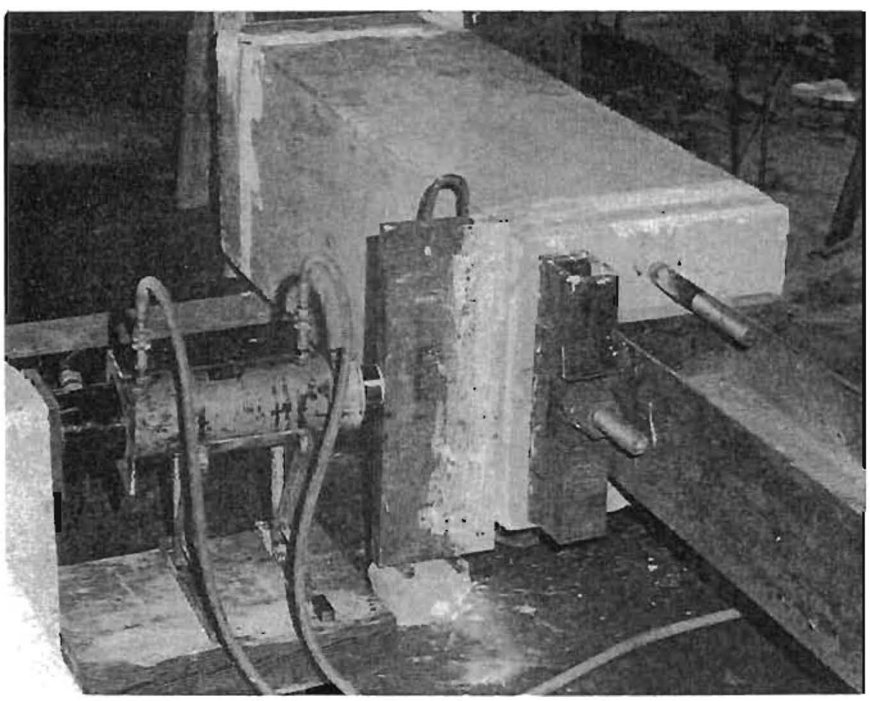


Fig. 6d. Compression zone loading

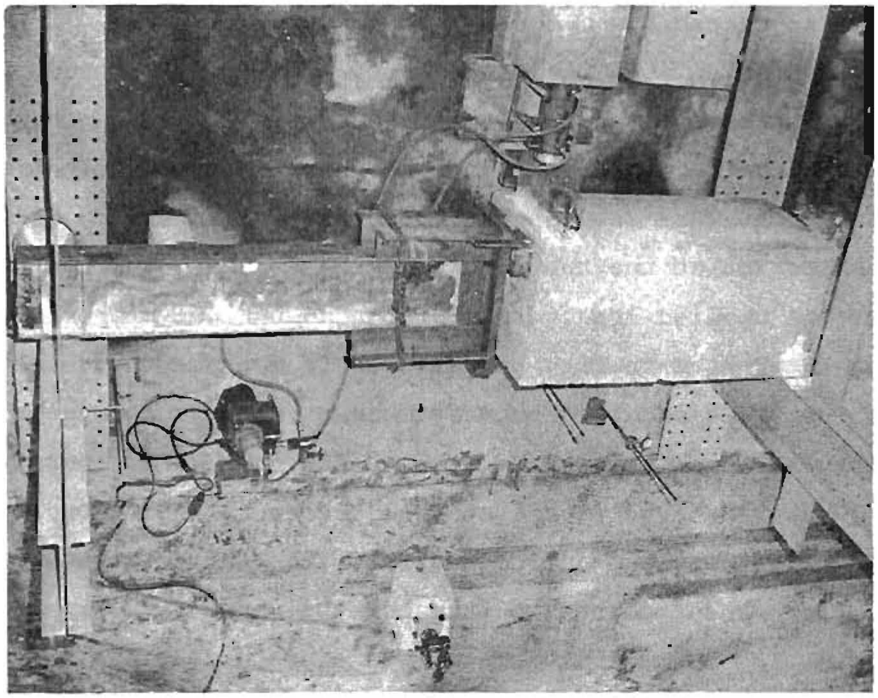


Fig. 6e. Test arrangement (large bolts) as viewed from above specimen

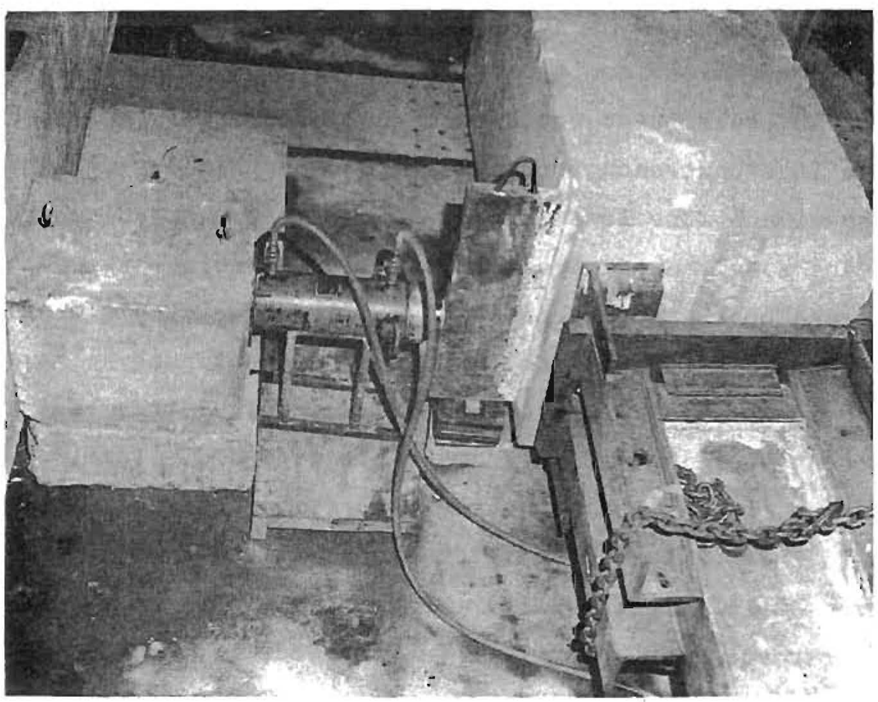


Fig. 6f. Compression zone (large bolts)

of the compression centroid.

The instrumentation utilized was relatively simple. In addition to the pressure gage to measure the hydraulic pressure applied to the ram, and a load cell to check the applied load, measurements were made of deflections and loaded end slip. Dial gages were used as indicated at H, I, and J in Fig. 6b to measure the deflection at the two reaction straps and in the footing specimen directly under the applied load. In addition an optical micrometer mounted on the theodolite shown at K in Fig. 6c was used to measure the relative slip between the anchor bolt and the concrete face. In order to make this measurement, three targets were installed on the specimen. Two of these targets were mounted (one 1" above the bolt, the other 1" below the bolt) on steel rods that were embedded in the concrete during casting. The third target was mounted (using a strong permanent magnet) on the bolt one half inch from the face of the concrete. (L). The optical micrometer was used to read the relative displacement between the target mounted on the bolt and the targets which were fixed to the concrete. This micrometer attachment has a sensitivity of 0.001". This system proved to be a reasonably satisfactory method of taking the slip measurements in an otherwise remote and fairly inaccessible region.

The loads were applied in increments and held until the deflection dials and the optical micrometer could be read and any cracking marked and recorded. Loading for a particular test would require approximately one to two hours. Because of the fact that each footing specimen contained a multiple number of anchor bolt specimens, the loading pattern had to be varied to preserve the over-all footing specimen until all bolts were tested. This was accomplished by initially loading the specimens only up till apparent yielding. At this point the test on an individual bolt would be discontinued. The specimen would be rotated and the testing begun on a different bolt. This bolt would then be taken to apparent yielding and the same cycle followed. This would continue until the testing of the last anchor bolt specimen for a particular footing. This bolt would then be loaded until complete failure occurred. If the condition of the specimen still permitted after this, it would be rotated and another bolt would be retested until complete failure occurred. This sequence was followed as long as the specimen was operative. Thus, most of the initial data obtained is for ranges up to and slightly over the yield point. A fairly large number of the tests were ultimately taken to failure. Much of this data represents retesting of a fairly well damaged specimen. However, since the specimens with 2-1/2" and 3" diameter anchor bolts only contained two bolts in each footing, these tests were taken to failure without retesting.

Specimen Behavior and Failure

Loaded end slip started almost immediately upon initial loading and progressed with gradually increasing increments. In a few cases splitting of the cover started over the bolt at the loaded end and progressed towards the anchored end. In most cases very definite flexural cracks opened up across the face of the beam. Usually the largest flexural crack occurred at the vicinity of the anchorage of the bolt (which represents the "cutoff point"). The final failure of those specimens which were taken to ultimate fell into three different classes;

1. Specimens in which very definite and pronounced splitting appeared along the full length of the bolt at ultimate load, as shown in Figs. 7a and 7b. It can be seen in Fig. 7b that a very wide splitting crack extends from bolt 1-1/2 N 10a to the face of the specimen. It can also be seen that the concrete cover over this bolt is breaking off into two distinct wedges. This is similar to failures found in pullout tests of deformed bars.

2. Specimens in which very little longitudinal splitting appeared prior to or at failure but in which very intensive spalling and crushing of the concrete in the vicinity of the anchorage of the bolt appeared. Most of the cracking and crushing indicated did not become apparent until failure occurred. A typical case is shown in Fig. 8a for specimen 2-1/2 W 15. Fig. 8b shows this same specimen with the concrete cover chipped off following the test. A slight gap can be seen between the head of the nut and the concrete.

3. Specimens in which the full tensile strength of the bolt was developed and failure occurred in the steel anchor bolt itself. Two types of such failures were observed as follows:

   a. In two specimens the bolt failed inside the concrete in the root of the thread immediately in front of the anchor nut. One of these cases is shown in Fig. 9a.

   b. In two other specimens the bolt failed outside of the concrete at the root of the thread where the loading nut was attached. One of these specimens is indicated by the arrow in Fig. 9b.

Although these four cases were the only ones in which the full ultimate tensile strength of the bolts were developed, the nominal yield points of the bolts were

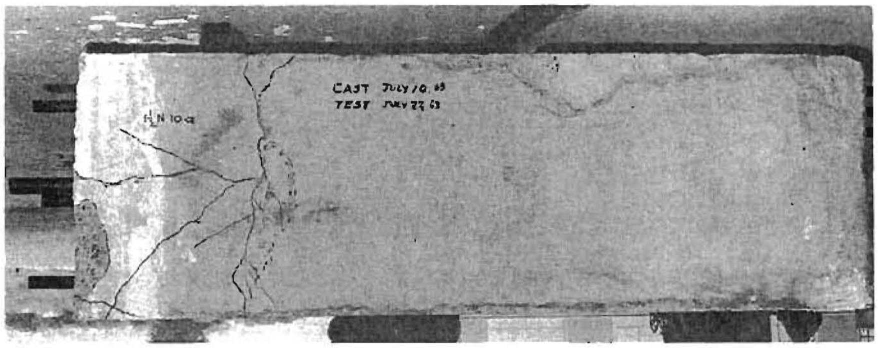


Fig. 7a. Splitting failure

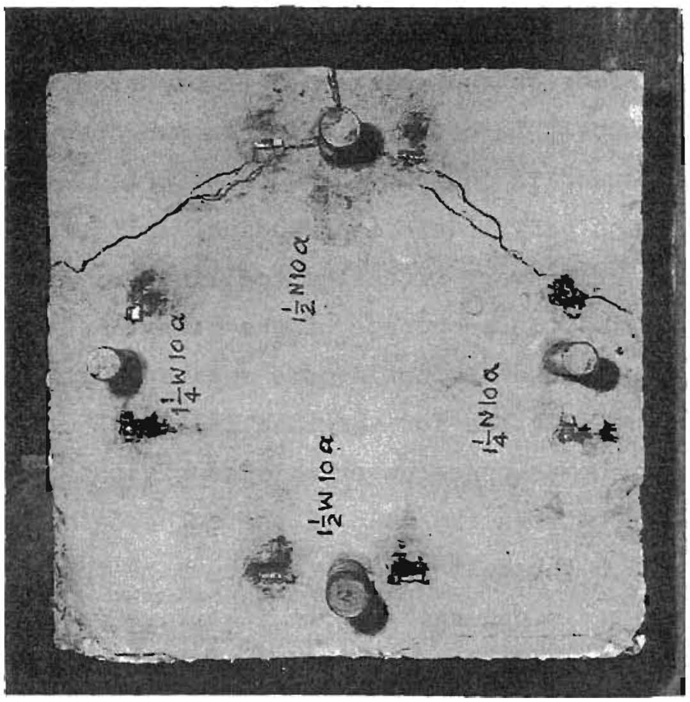


Fig. 7b. Splitting failure—end view

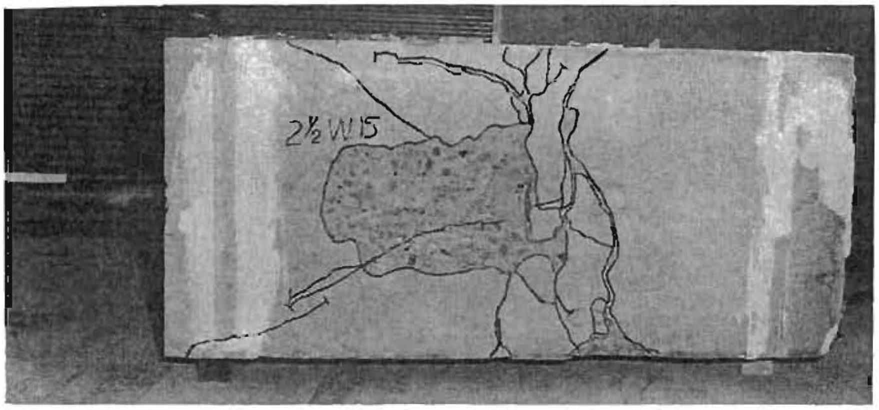


Fig. 8a. Crushing and spalling failure

Fig. 8b. Crushing failure—cover removed

Fig. 9a. Fracture of bolt inside concrete

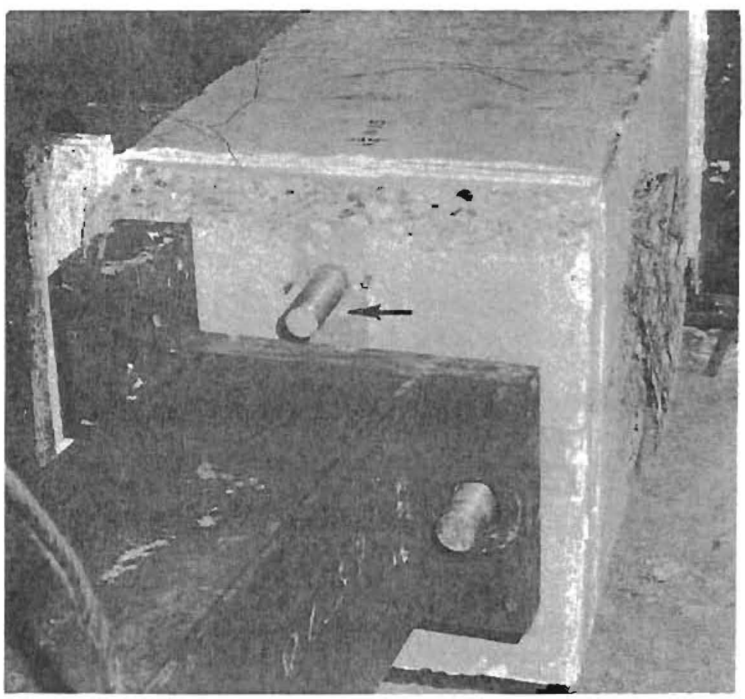


Fig. 9b. Fracture of bolt outside concrete

developed in over 90% of all the specimens tested.

In addition to these three general cases there is one other specialized type of failure. In the series to check special forms of end anchorage, one anchor bolt was used which had no anchorage. (That is, it was just a smooth round bar.) This specimen failed by simply pulling out of the concrete at a fairly low stress.

Basic Calculations

Since the loading system was statically determinate, the applied moment at the concrete bearing face can be calculated as a function of the known ram load. The calculated steel stress f_s can be found approximately as $f_s = M/(A_s jd)$. Because of some apparent discrepancies in preliminary calculations, it was decided that the values for calculated steel stress should be computed for both the gross bolt area ignoring the threads and for the mean thread area as recommended by ASTM standards.[5] Tests have shown that the strength computed on the area at the root of the thread gives a fictitiously high value. It has also been shown that if the strength is computed on the pitch area it gives a fictitiously low value. Slaughter[6] has observed that the mean diameter is closer to the diameter of an equivalent bar in tension than either of these other two diameters. The mean area can be calculated from Eq. 1.

$$A_{sm} = 0.7854 \left[D - \frac{0.9743}{n} \right]^2 \qquad (1)$$

where A_{sm} = mean stress area

D = the nominal diameter of the bolt

n = threads per inch

Because of the design of the loading assembly, the compression force was concentrated on a very small line area of the compression zone. Hence it was felt that the lever arm jd was known within an accuracy of 5%. Thus all steel stresses could be accurately calculated from $f_{sm} = M/(A_{sm} jd)$.

The unit bond stress, u, is that which would be obtained by dividing the calculated bolt tension by the entire surface of the bolt from the beginning of the embedment to the point of attachment of the washer and nut, as though the bond were uniformly distributed. In all cases when we speak of a number of diameters of bolt embedment we are speaking of this same distance, that is, from the point of embedment to the beginning of the head of the anchorage nut.

CHAPTER 3
ANALYSIS OF DATA

Variables

The primary variables in this investigation were three in number, namely, bolt size, bolt length, and type of end anchorage. Accompanying the variation in bolt size was the corresponding variation in the size of the standard nuts and washers used for the end anchorages. A complete tabulation of the bolt, nut, and washer dimensions is given in Table 1.

In addition three secondary variables were introduced due to the nature of the test specimens used and to the construction practices. The first of these is the ratio of the clear cover over a bolt to the bolt diameter. Following the standard practices of the Texas Highway Department, bolts in a footing specimen of a given depth were located with a fixed center to center dimension. Inherent in this method of detailing is a decreasing clear cover for larger bolt sizes. This can be represented both as an absolute decrease in cover and as a decreasing ratio of cover to bolt diameter. These values are also tabulated and are given in Table 1.

Secondly, the concrete strength varied somewhat during the test program. While it was desired that all concrete should have the same f_c', due to the inherent nature of scatter in the concrete mixes some deviation was noted. A histogram of the concrete strength f_c' is shown in Fig. 10. It can be seen that the average f_c' was 4650 psi. However, in two footing specimens a substantial deviation from this value was noted. Although in many bond investigations using deformed bars a correction for variation in concrete strength has been made assuming that the bond strength will vary as the square root of f_c', there is no evidence that this same relationship would apply for the steel or bond stress developed at a given slip in these specimens. Since very meager data is available for variation of concrete strength in this series, it was felt that the basic data could not be corrected for the variation in concrete strength. However, attention will be called to this point in later sections of this report.

Lastly, it has previously been mentioned that two of the anchor bolts delivered and tested were machined from a high strength steel with no well defined yield point. These specimens illustrate some of the effects of using higher yield point steels. In the original proposal it was indicated that an

TABLE 1

ANCHORAGE DETAILS

Bolt Diameter, in.	Nut Diameter,[1] in.	Washer Diameter, in.	Clear Cover, in.	Clear Cover / Diameter
1 1/4	1 7/8[2]	3 [4]	2 3/8	1.90
1 1/2	2 1/4[2]	3 1/2[4]	2 1/4	1.50
1 3/4	2 3/4[3]	4 [4]	3 1/8	1.79
2	3 1/8[3]	4 1/2[4]	3	1.50
2 1/2	3 3/4[2]	5 [5]	2 3/4	1.10
3	4 5/8[3]	5 1/2[6]	2 1/2	0.83

Notes: (1) Measured Across Flats

(2) Finished Hexagon Nut (Ref. 7)

(3) Heavy Semifinished Hexagon Nut (Ref. 7)

(4) Type B, Regular Series (Ref. 8)

(5) Type A (Ref. 8)

(6) Type B, Narrow Series (Ref. 8)

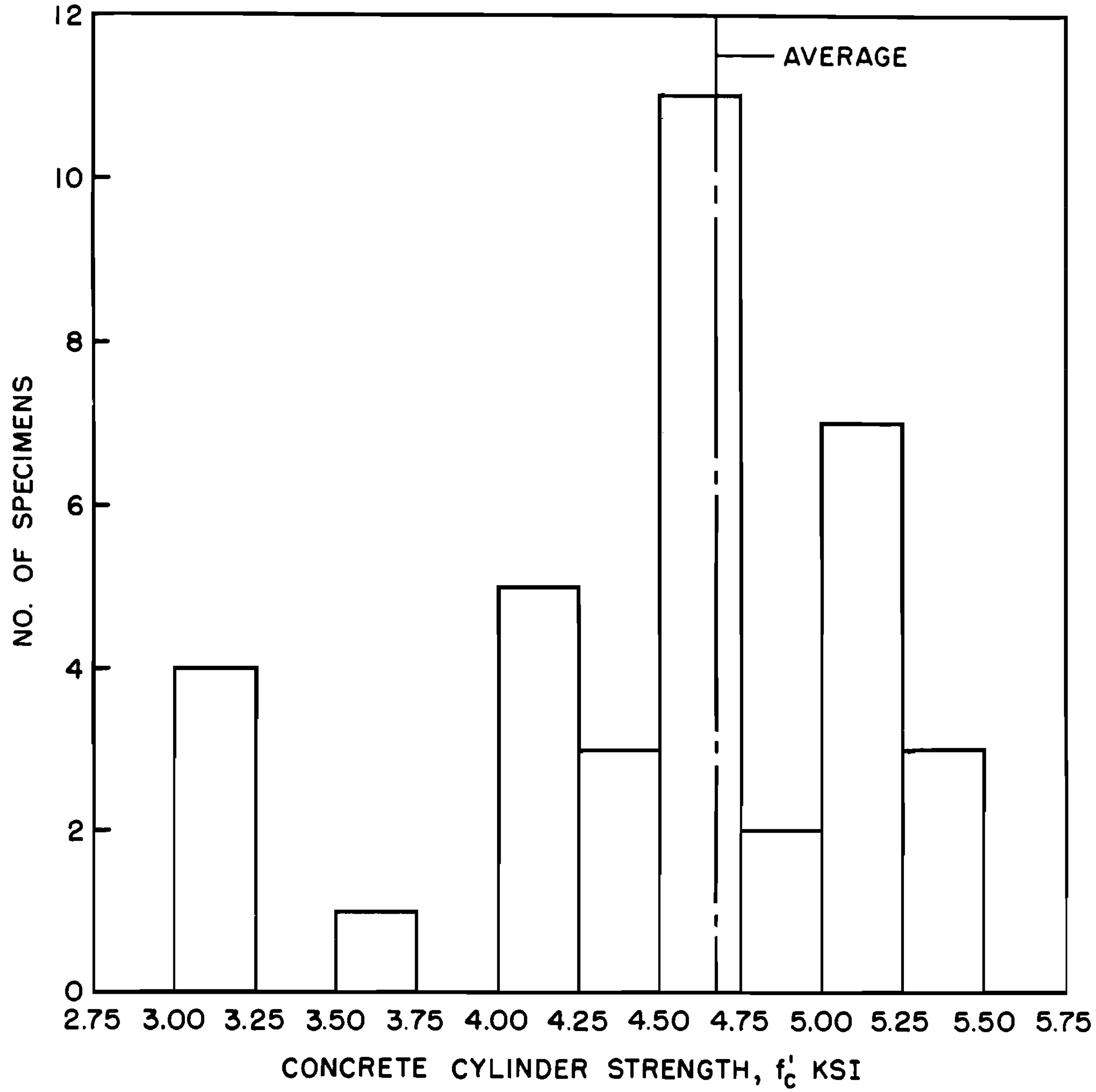


FIG. 10 HISTOGRAM OF CONCRETE STRENGTH, f'_c

exploratory specimen would be provided utilizing such high strength steel. The tests of these two bolts achieved this purpose.

Steel Stresses

As has been previously mentioned, a fundamental question which had to be determined in the interpretation of the data was the basis for computation of the steel stress on a partially threaded bolt. After a study of the available references it was decided to run a single companion test program to verify the accuracy of the mean thread area method. Accordingly, two tensile test specimens were ordered from the manufacturer of the bolts. The first was a piece of unthreaded 1-1/2" bar stock. The second was a piece of the same stock threaded over its full 18" length with standard 1-1/2" USS threads. The two specimens were then tested using an identical 5" gage length to measure the strains. The resulting stress-strain diagrams are shown in Fig. 11. The diagram for the unthreaded bar is indicated by the curve OA. The corresponding stress-strain diagram for the threaded bar based on the root area is indicated by the curve OB. It will be noted that for a given strain, the latter indicates a stress several percent greater than that of the unthreaded section. The stress-strain curve for the threaded bar as computed on the basis of the mean thread area is indicated by curve OC. Computations by this method indicate a stress for a given strain several percent less than that of the unthreaded bar. It was felt that, in the interpretation of the test results, steel stress computations should be based on the mean thread area so that the steel stresses indicated would be on the conservative side. Thus all steel stresses referred to in the remainder of the report pertain to the mean thread area.

Loaded End Slip

The optical micrometer system did not measure slip alone but also included the elongation of the bolt between the position at which the target was attached to the bolt and whatever portion of the bolt was no longer bonded to the concrete. Since this length was highly indeterminate no attempt was made to reduce the apparent slip measurement to correct for the elongation of the bolt. Therefore in the remainder of this report when the term slip is used it will be taken to include the total extension of the bolt as measured at the face of the

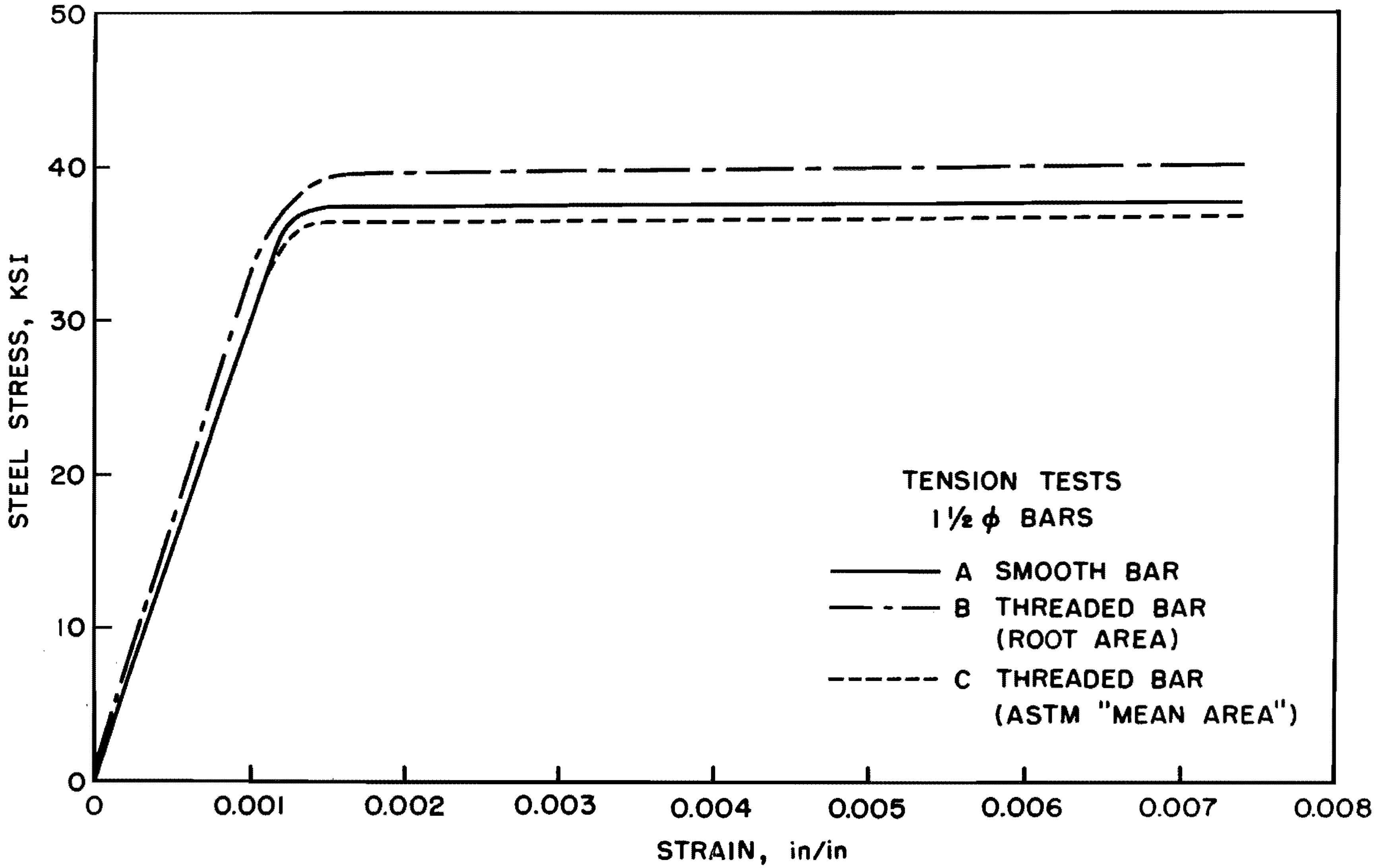


FIG. 11 EFFECT OF METHOD OF CALCULATION OF STRESS IN A THREADED SECTION

concrete.

The plot of this slip against either load, bolt stress, or bond stress gives the same curve shape for any given specimen since a combination of given bolt size and length establishes fixed ratios between load, tensile stress, and average bond stress.

It has been mentioned that in the test procedure many of the specimens were loaded initially only until apparent yielding. Then the specimens were unloaded and a corresponding set of zero load readings were taken. Upon completion of the testing of the other bolts in the footing specimen, many of the bolts were reloaded until failure became evident. After examination of the data observed in these tests it was found that all of the reloaded specimens seem to have a common behavior. As indicated in Fig. 12 the steel stress-slip curve OA indicates the results of the initial testing of specimen 1-1/4 W 10b. Testing was discontinued at A and the load released. The total permanent slip upon removal of the load was observed as indicated by the descending branch AB. After testing of several of the bolt specimens, test 1-1/4 W 10b-2 was run. In this test the steel stress-slip curve BCD was observed. On the same figure is illustrated the results of a generally similar test specimen in which the bolt was initially tested to a much higher slip level. This steel stress-slip curve (1-1/2 N 10b) is shown by curve OC. It should be noted that the last observation agrees with an observed value for the retesting of 1-1/4 W 10b. If one extends curve OA by the light dashed line shown until it intersects curve CD, it is reasonable to assume that a credible cumulative steel stress-slip curve for this specimen is described by the resulting total curve OACD. In the remainder of the report such a construction has been used to eliminate the unloading and reloading branches of the steel stress-slip curves, since this construction always takes place substantially above the nominal yield point value. It is felt that this hypothesized curve will serve as a reasonable indication of the measure of ductility observed. However, in all cases a dashed line will be used to indicate the portion of the curve which is not primary observed data.

The resulting mean area steel stress-slip curves are plotted as follows:

Figure 13. 1-1/4" bolts	Figure 16. 2" bolts
Figure 14. 1-1/2" bolts	Figure 17. 2-1/2" bolts
Figure 15. 1-3/4" bolts	Figure 18. 3" bolts

Figure 19. The special anchorage group

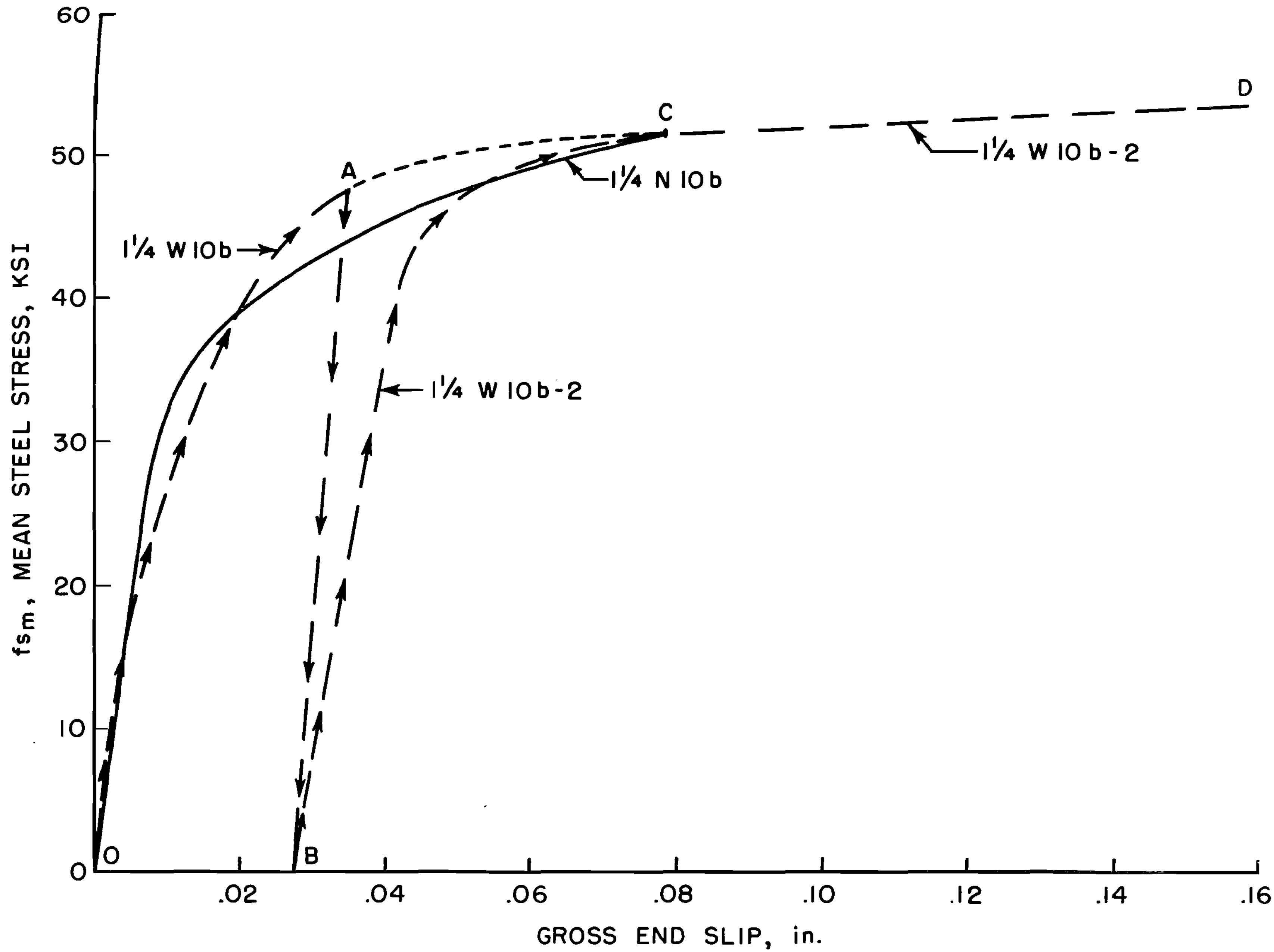


FIG. 12 EFFECT OF INTERRUPTED LOADING ON STEEL STRESS-SLIP RELATION

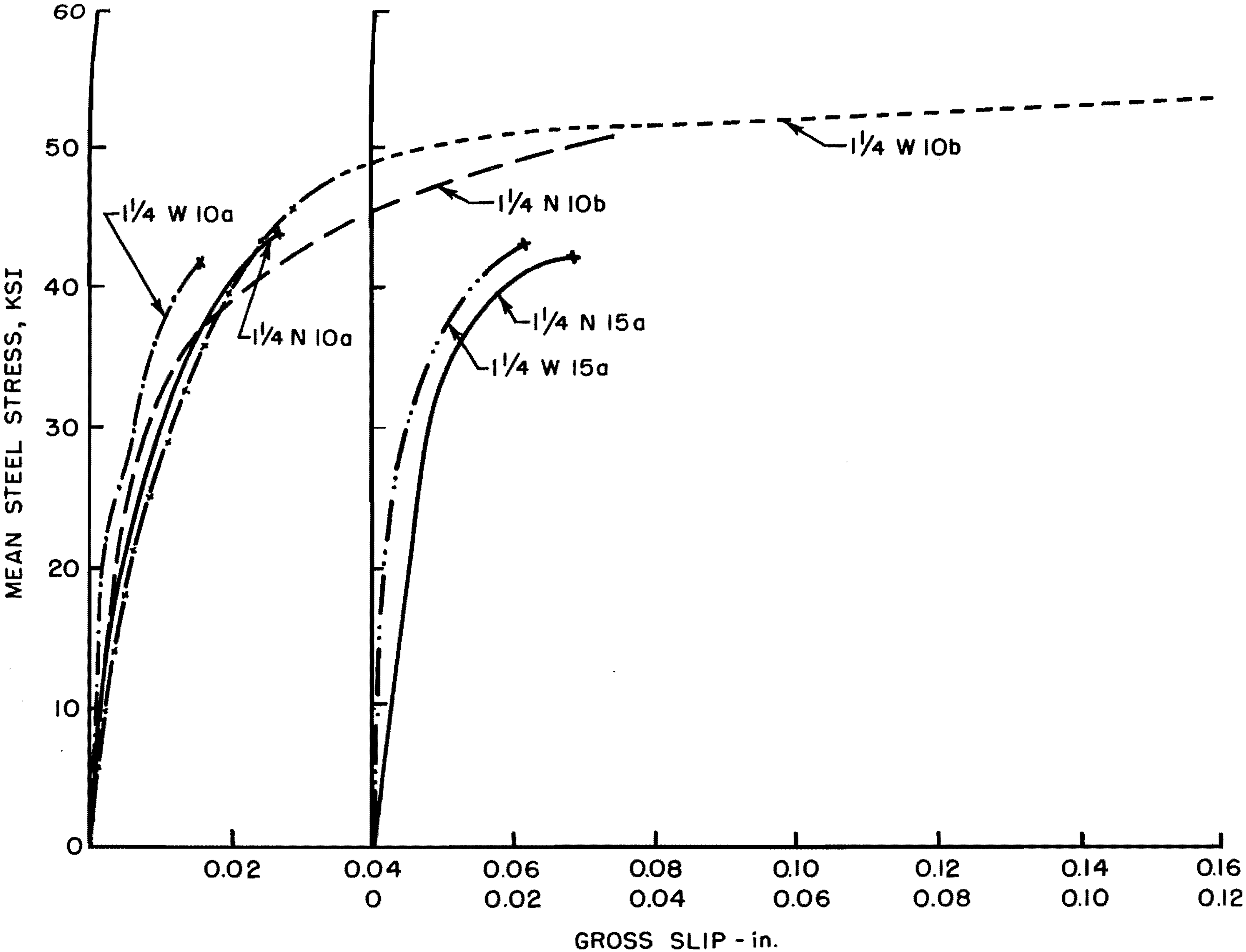


FIG. 13 STEEL STRESS-SLIP CURVES FOR 1¼" BOLTS

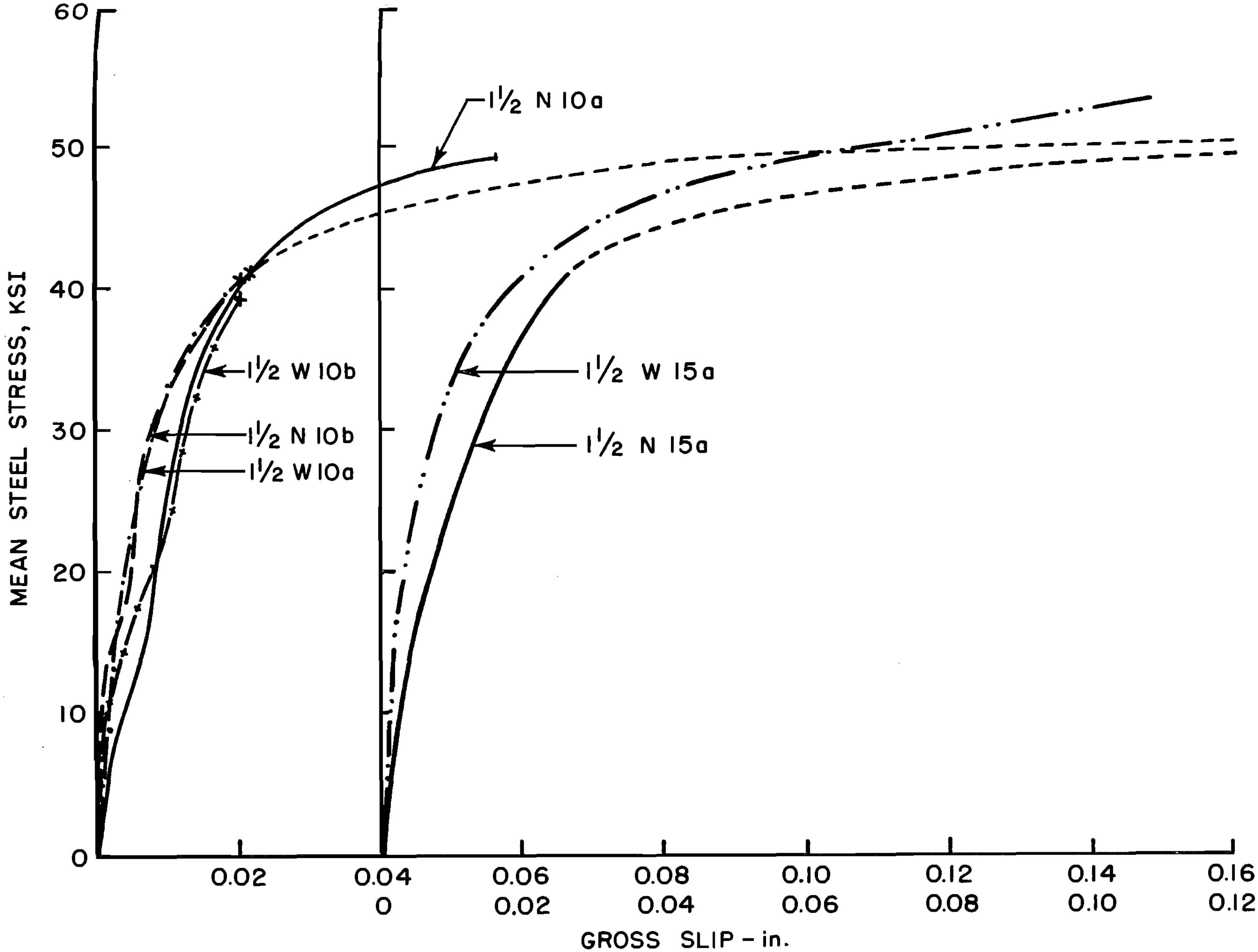


FIG. 14 STEEL STRESS-SLIP CURVES FOR 1½" BOLTS

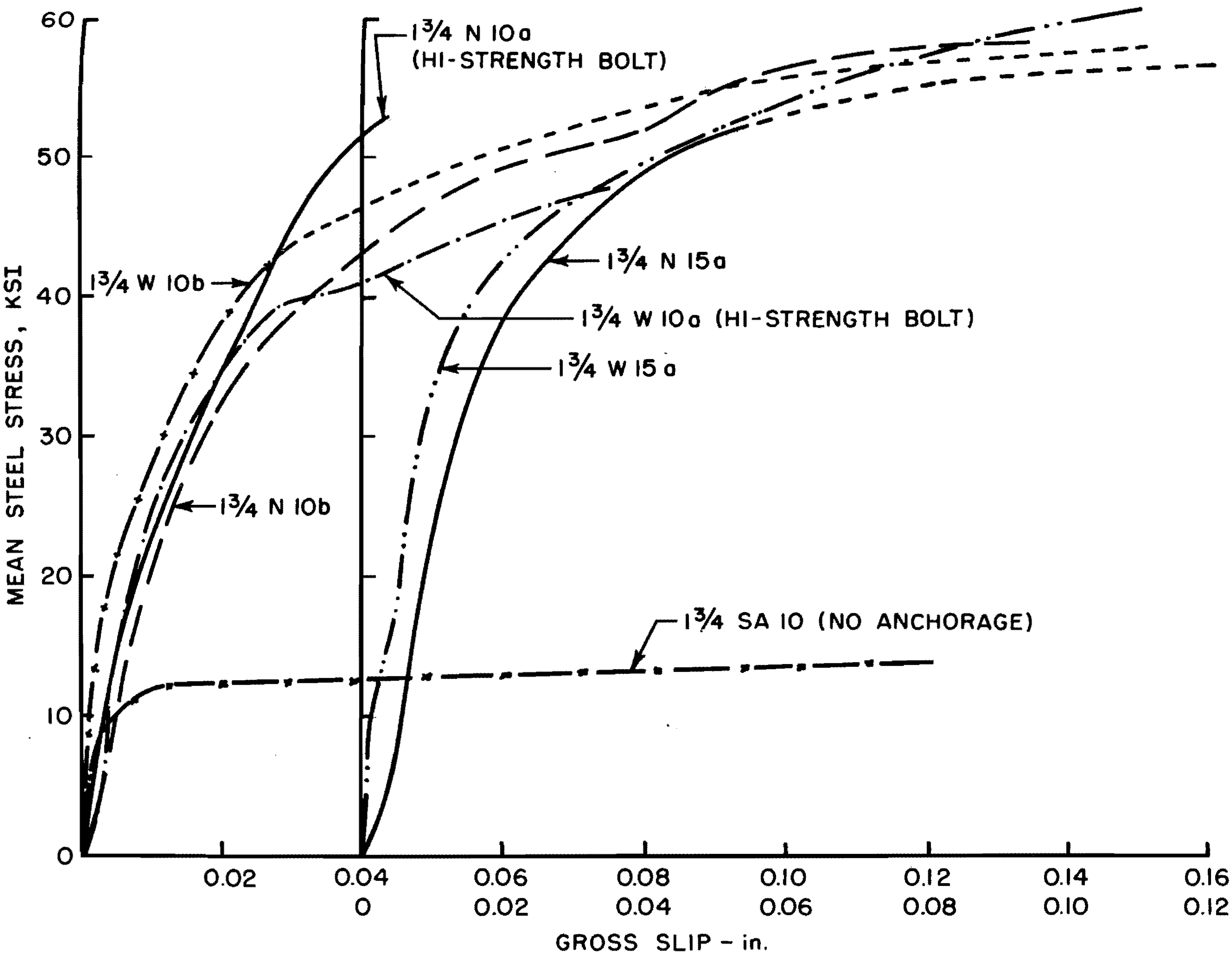


FIG. 15 STEEL STRESS-SLIP CURVES FOR 1¾" BOLTS

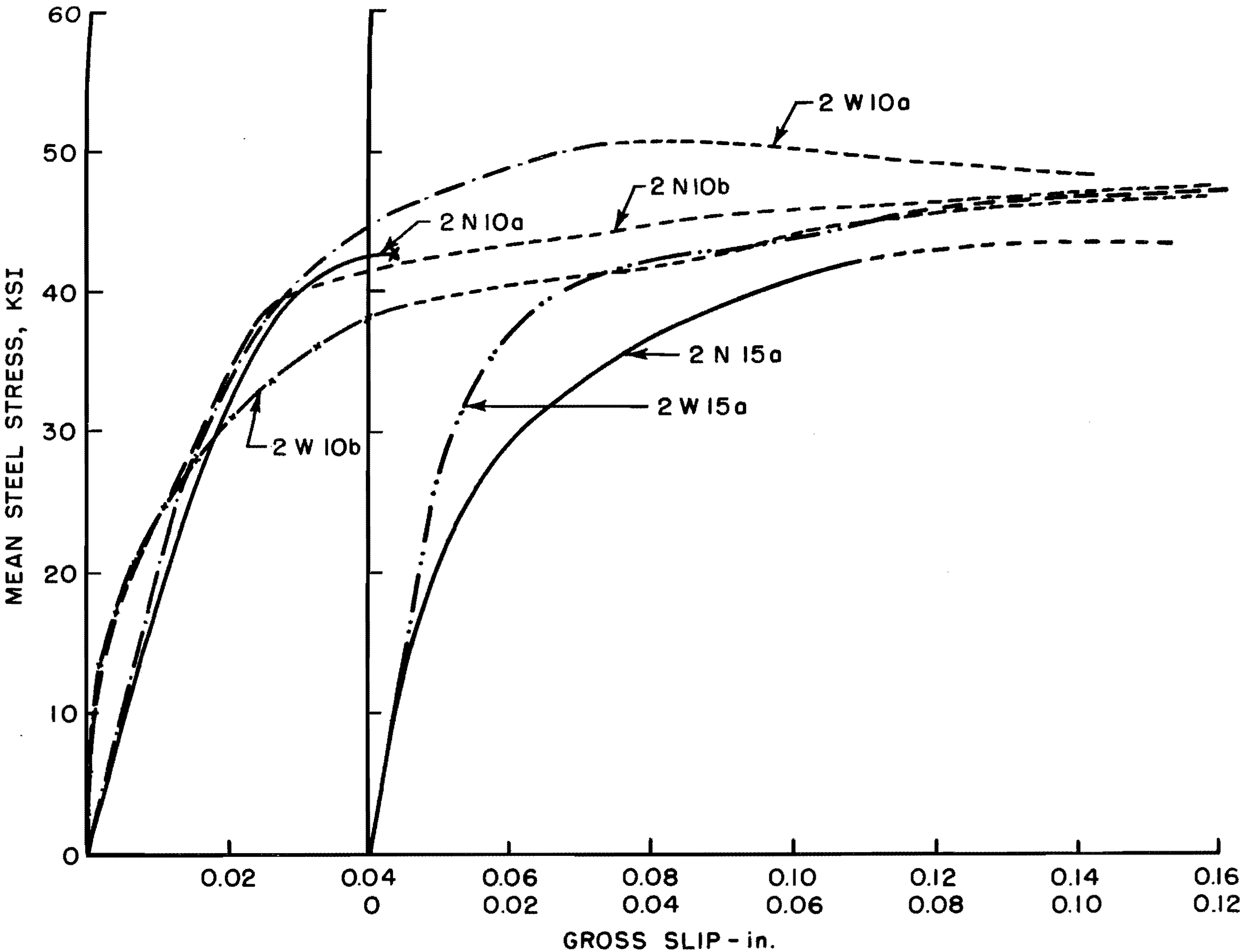


FIG. 16 STEEL STRESS-SLIP CURVES FOR 2" BOLTS

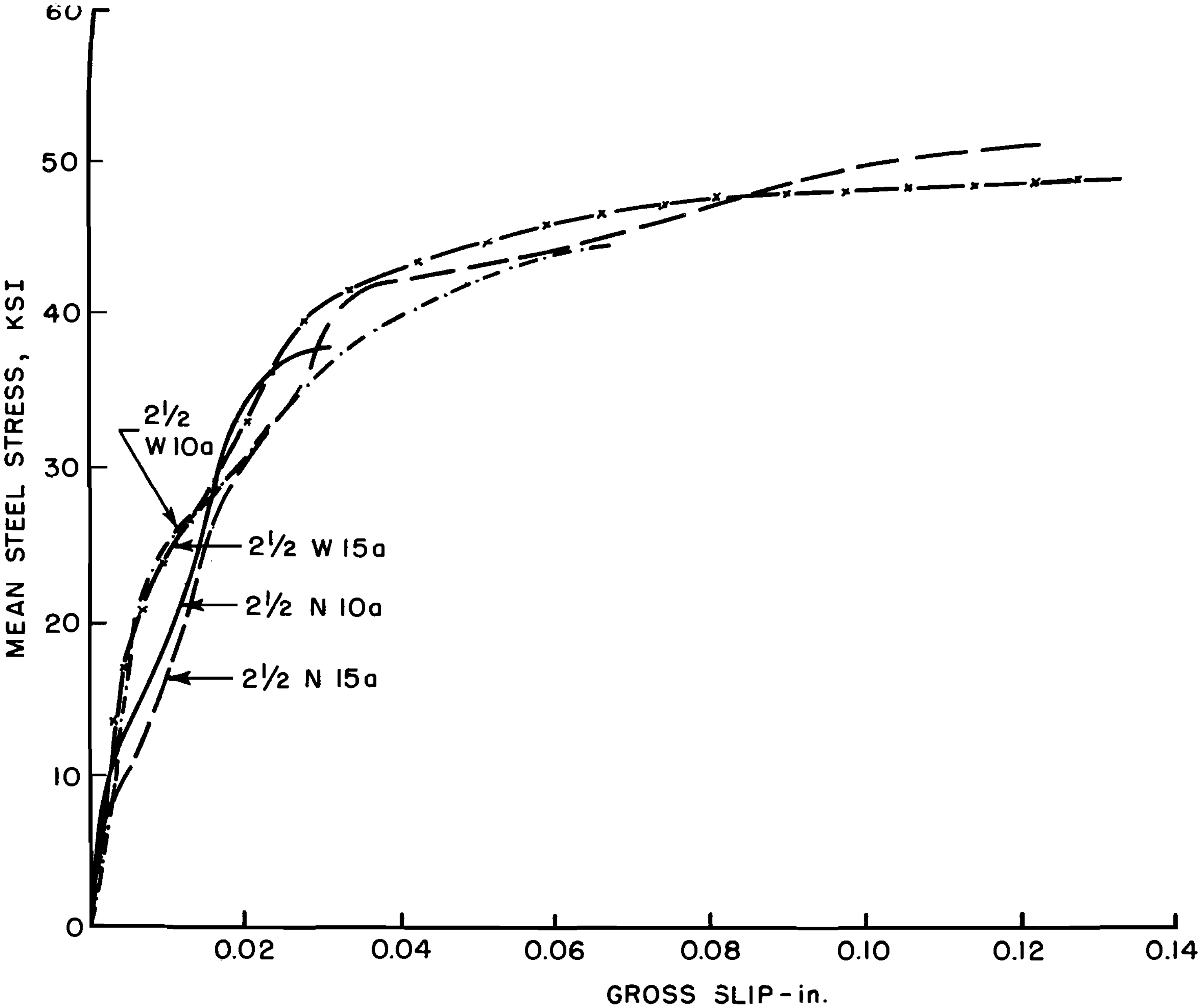


FIG. 17 STEEL STRESS-SLIP CURVES FOR 2½" BOLTS

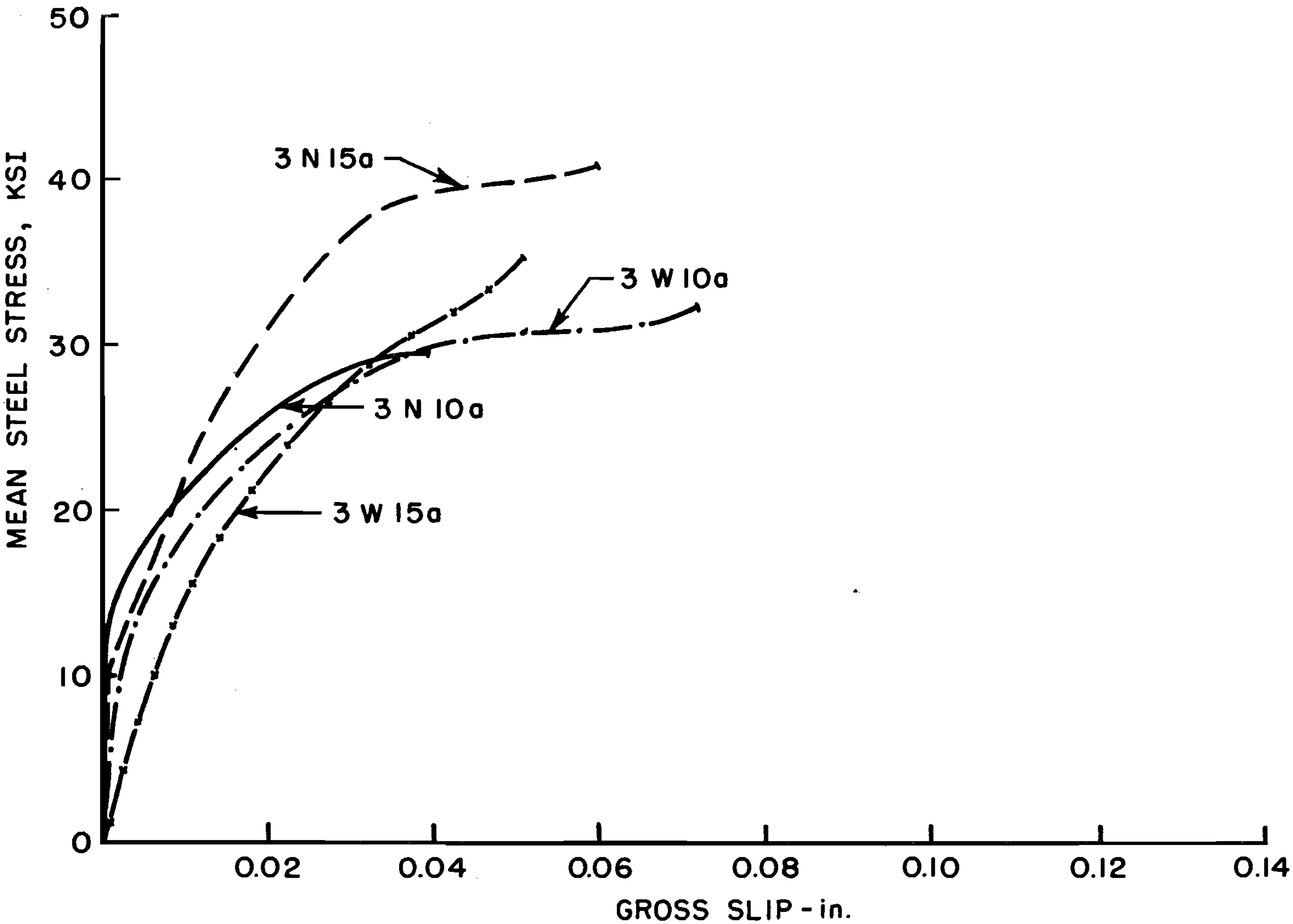


FIG. 18 STEEL STRESS-SLIP CURVES FOR 3" BOLTS

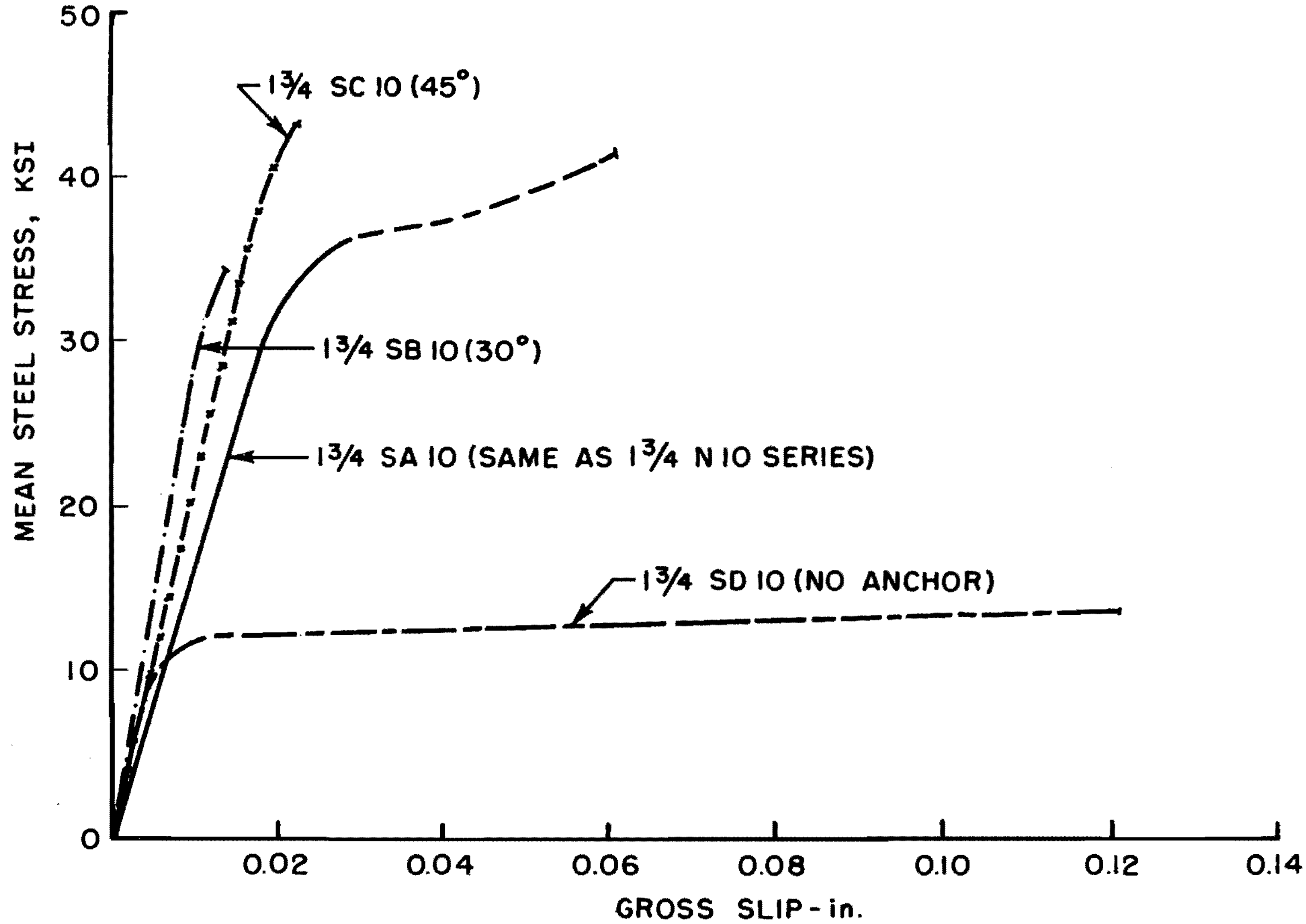


FIG. 19 STEEL STRESS-SLIP CURVES FOR 1 3/4" BOLTS WITH SPECIAL ANCHORS

These figures allow comparisons to be made between the main variables in each bolt size grouping. In all of these figures an x has been shown at the point of discontinuance of the testing because of apparent yielding if the bolts were not retested. Hence a curve which ends in an x should not imply lack of ductility in the particular test but rather incomplete data.

Since there is an extremely large amount of data presented in these figures, selected portions of the significant data will be rearranged in the succeeding sections to facilitate analysis.

End Slip at Critical Steel Stresses

In order to illustrate service load behavior the measured end slip corresponding to a steel stress of 20 ksi is shown in Fig. 20 as a function of the bolt diameter. This steel stress corresponds closely to the present service load design stress used by the Texas State Highway Department Bridge Division for sign structures. The bolts with 10 D embedment lengths tend to indicate slightly greater slip than do those with 15 D embedment lengths. However, there is not a clear cut definite pattern and in several cases the longer bolts actually show a greater slip than the shorter bolts. Except with the 3" bolts the combination nut and washer anchorage (W) shows somewhat less slip than the plain nut anchorage (N). The 3" bolts with nut and washer anchorages show more slip than the corresponding specimens with nut anchorages alone. This is discussed later with regard to the reduced cover over the washers.

In the analysis of pullout tests of deformed bars it has normally been assumed that the slip at the loaded end might be considered as half of the crack width that would be developed in a beam at that bar stress. Under these conditions a crack 0.02" wide would correspond to an end slip of 0.01". The 0.02" crack is about an upper limit on suggestions that have been made as to maximum permissible crack widths in beams at working loads. It is realized that this crack width limitation is not necessarily applicable to the slip occurring at the end of an anchor bolt. Indeed a good justification may be given for selecting the limiting service load value of slip as 0.02" in this type application.

If the more stringent slip limit of 0.01" is applied, all of the anchor bolts of less than 2" diameter meet this requirement regardless of the embedment length or type of anchorage. (Note the 1-3/4" special anchorage series is omitted.) For all practical purposes this statement could be extended to

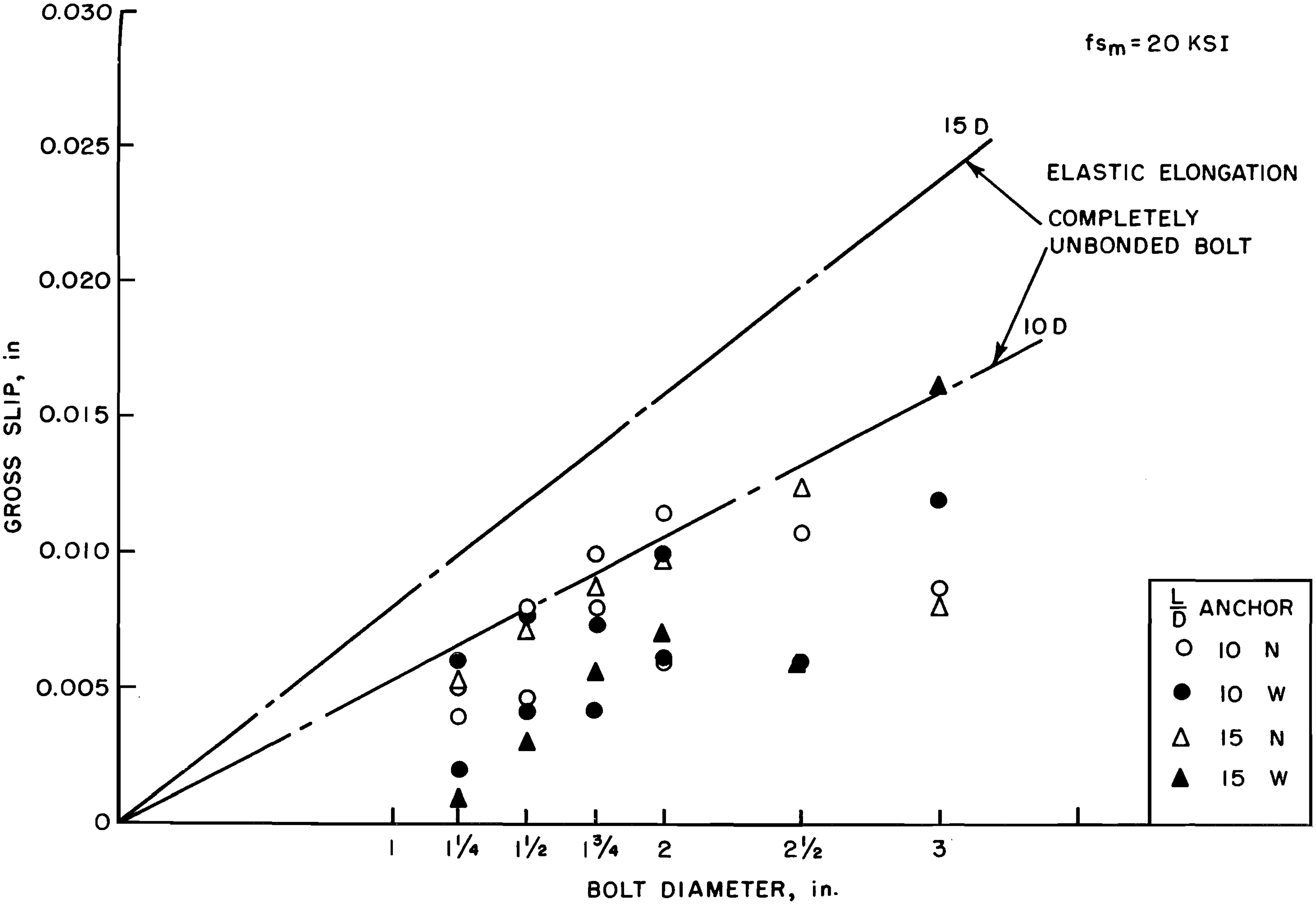


FIG. 20 SLIP AT A STEEL STRESS OF 20 KSI

include the 2" diameter bolts since only one of these shows a slip in excess of this limit. Both 2-1/2" bolts with nut and washer anchorages meet the criteria although those with nut anchorages do not. However, the 3" bolts have an exactly opposite behavior. Bolts with nuts developed much less slip than those with nuts and washers. This is apparently due to the extremely shallow cover (1-1/4" clear cover) over the washer in this series which led to early cracking. If the criteria of 0.02" is accepted, all of the bolts would meet the criteria.

Figure 21 illustrates the observed slip at the nominal yield point of the bolts, that is a mean area steel stress of 33 ksi. While the allowable slip criteria for this case cannot be easily formulated, it is interesting to note that all specimens except three of the 3" bolts indicate less than 0.03" end slip at the nominal yield. In the case of the 3" bolts, the specimens with 10 D embedment length had failed (as indicated) at less than 33 ksi, while the 15 D specimen with nut and washer anchorage had a slip of approximately 0.046" at this stress level. It can again be noted that generally the 15 D specimens with washers show the least amount of slip at this stress level. In the 10 D specimens, slip does not seem to vary consistently with the type of end anchorage. In some cases, specimens with nuts show less slip than specimens with nuts and washers. The opposite is true in other cases.

In order to provide a qualitative indication of the relative relation between slip and elastic elongation, two additional curves are given in Figs. 20 and 21. Assuming that the anchor bolt is unbonded between the face of the concrete and the head of the anchor and using the stress as computed on the unthreaded area in this region, one can determine the theoretical elastic elongation of the bolt. This approximate elongation is shown for both 10 D and 15 D embedment lengths. In Fig. 20, the 10 D bolts tend to indicate an end movement fairly close to that which might be expected if no bond were present, while the 15 D bolts indicate substantially higher bonding characteristics. In Fig. 21 the 10 D bolts generally indicate an even greater end movement than that calculated. This indicates two important factors. The first is that the effect of bond at this level must be very small. Since the 1-3/4" bolt with no end anchorage and a 10 D embedment length pulled out of the specimen at a stress of about 13.6 ksi, the major portion of the tensile development must be by bearing on the end anchorage device. Only a minor portion can be credited to friction bond. Second, since the measured end movement is greater than the total elastic elongation computed assuming zero bonding, the end anchorage

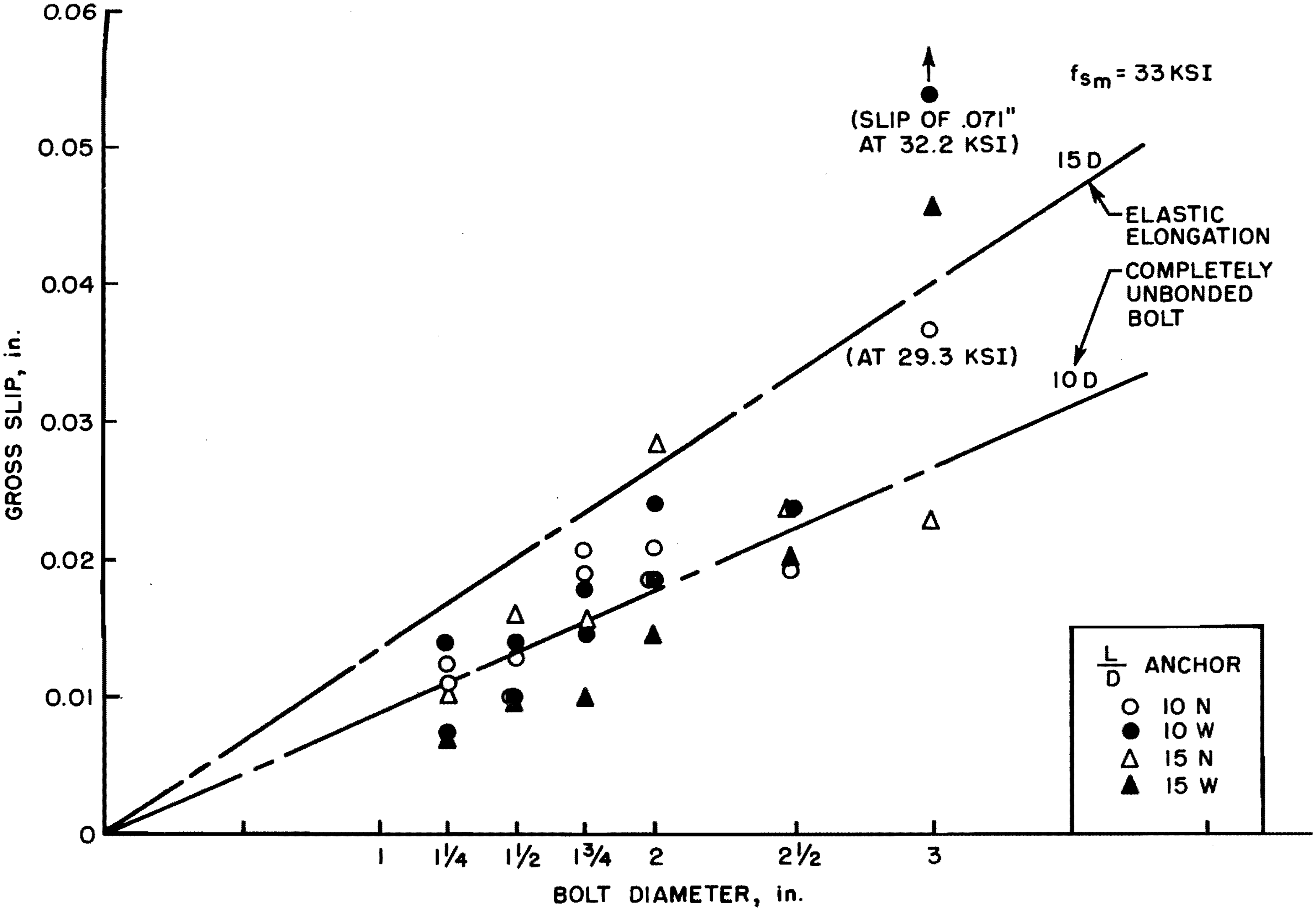


FIG. 21 SLIP AT A STEEL STRESS OF 33 KSI

device must have moved. After completion of the testing, the cover over the bolts on many specimens was carefully removed. As can be seen in Fig. 22 a definite separation was visible between the rear end of the nut and the concrete immediately adjacent to it. Similarly, as shown in Fig. 23, in the cases of specimens with nut and washer anchorages both a gap behind the nut and a definite bending of the washer around the nut were observed. This simply reinforces the conclusions arrived at by Abrams[1] that a certain amount of slip is essential to bring the end anchorage into action, and that the amount of movement required to bring this end anchorage into bearing will almost completely destroy the bars adhesion and sliding resistance. However, the 15 D bolts generally have less end movement than if completely unbonded. Therefore, it can be concluded that the longer embedment length does not rely so completely on end anchorage to develop the nominal yield point of the bolt.

Steel Stress at Limiting End Slip

Another way of illustrating the concept of a limiting slip criteria is to rearrange the basic data to show the steel stress developed at a particular end slip. Using the criteria of a limiting service load end slip of 0.01" as discussed in the previous section, the relation between steel stress, bolt size, embedment length, and type of anchorage is illustrated in Fig. 24. As shown previously, all 1-1/4" through 1-3/4" bolts (with the exception of the unanchored 1-3/4" bolt) developed greater than 20 ksi at this end slip. All of the 2" bolts except one developed greater than 20 ksi. Only the 2-1/2" and 3" bolts pose a serious problem with the development lengths used. Two factors must be considered for these bolts. First, should the allowable slip criteria be relative to bolt size? Is it realistic to expect that a 3" bolt should meet the same deformation criteria as a 1-1/4" bolt? Secondly, in a subsequent section the cover used with these bolts will be shown as possibly inadequate. With revised cover and a more realistic criteria for the large bolts, all sizes should fulfill service load demands.

Ultimate Strength Behavior

In addition to meeting service load requirements, it is important to ensure that the anchor bolts will not fail due to loss of end anchorage or

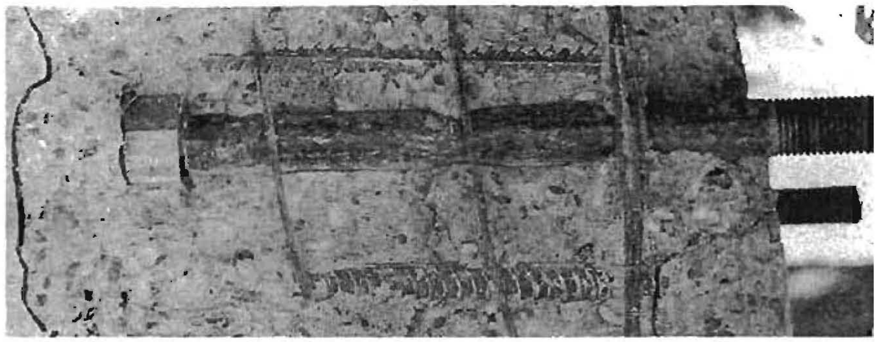


Fig. 22. End movement of nut

Fig. 23. Movement and distortion of nut and washer

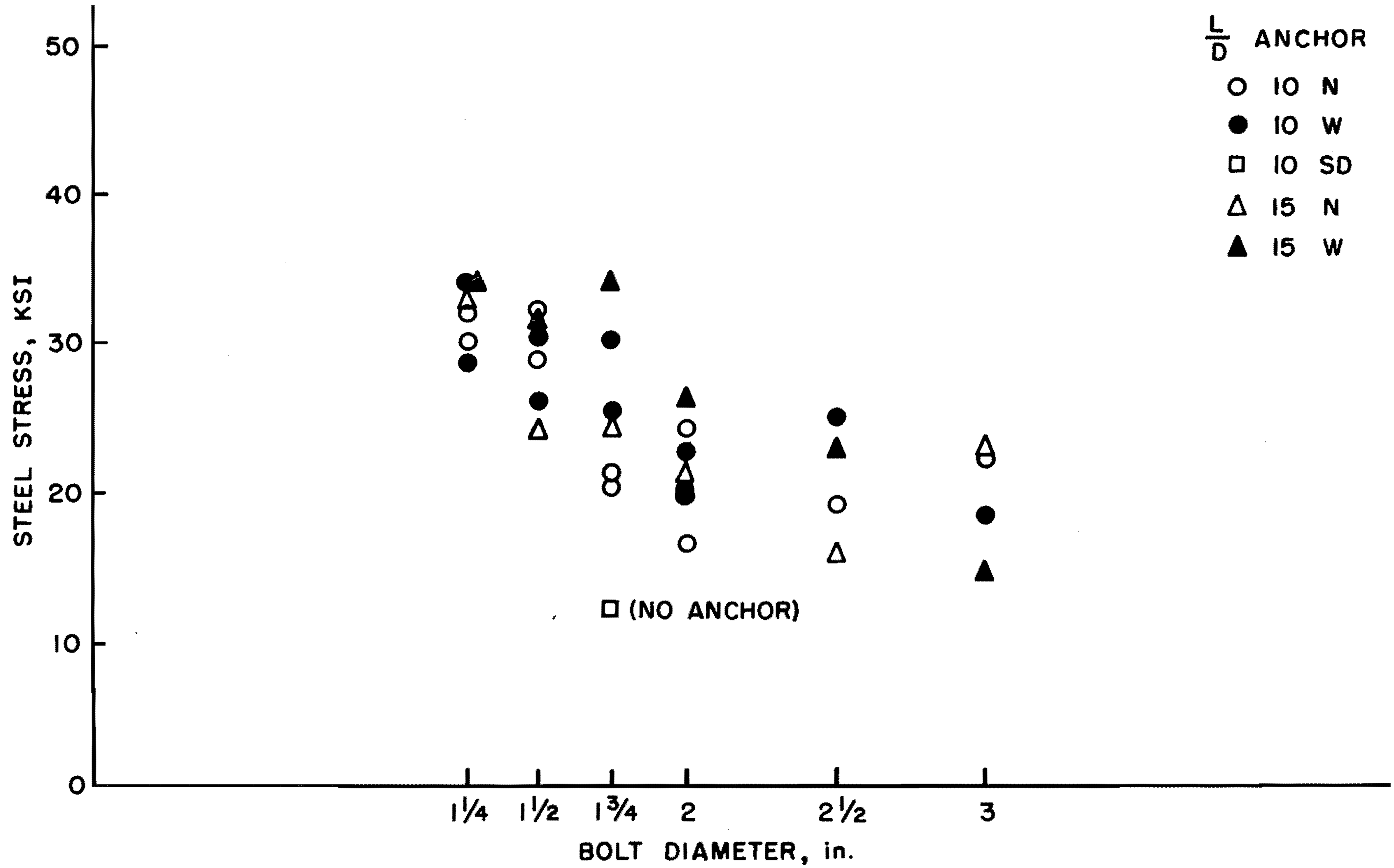


FIG. 24 STEEL STRESS AT A SLIP OF 0.01 in.

bearing capacity prior to the development of their full yield point stress. Unless this yield point can be developed, the structural design will not have the desired factor of safety. Accordingly, even though it is realized that the extreme amounts of slip occurring at high loads would be undesirable, a study has been made of the ultimate strength characteristics of the specimens.

Fig. 25 shows the ultimate steel stress (f_{su}) as a function of the bolt diameter, D. All specimens tested to failure except the 10 D 3" bolts developed the nominal yield point of 33 ksi. In general the 15 D embedment was somewhat more effective than the 10 D embedment. However, this effectiveness was definitely much less than the 50% increase in embedment length. Upon study of the data it was felt that one variable in the test was not reflected in this figure. This was the amount of clear cover over the bars. As has been discussed previously in the section on specimen behavior and failure, the larger bolt specimens with less cover failed due to crushing of the concrete over the anchor. After an attempt to relate ultimate steel stress to clear cover proved unsatisfactory it was decided to investigate the relationship between ultimate steel stress and the ratio clear cover divided by bolt diameter. This relationship is illustrated in Fig. 26. Comparison with Fig. 25 indicates clear cover as a function of bolt size seems to be a significant parameter. Two distinct curves are visible, one for the 10 D and the other for the 15 D embedment length. In addition, the nature of the failures have been shown. It can be seen that usually with low cover to diameter ratios, failure occurs due to crushing and spalling in the vicinity of the anchor. With higher cover to diameter ratios, the failure occurs due to splitting above the bolt. Unless sufficient cover is provided, the concrete above the anchor will crush prior to splitting of the cover or the development of the tension force in the bolt.

Bearing Stresses

In Abram's study of the concentric pullout specimen he had reported bearing stresses of an much as 13,000 psi in the specimens with nut anchorages. His concrete had a compressive cube strength of 2240 psi. In terms of cylinder strength, this would be equivalent to f_c' of about 1850 psi. Thus, the calculated bearing stresses would be as high as 7.0 f_c'. Of course, it must be realized that this is a case of a concrete element under a great deal of confinement and it should be possible to develop very high compressive

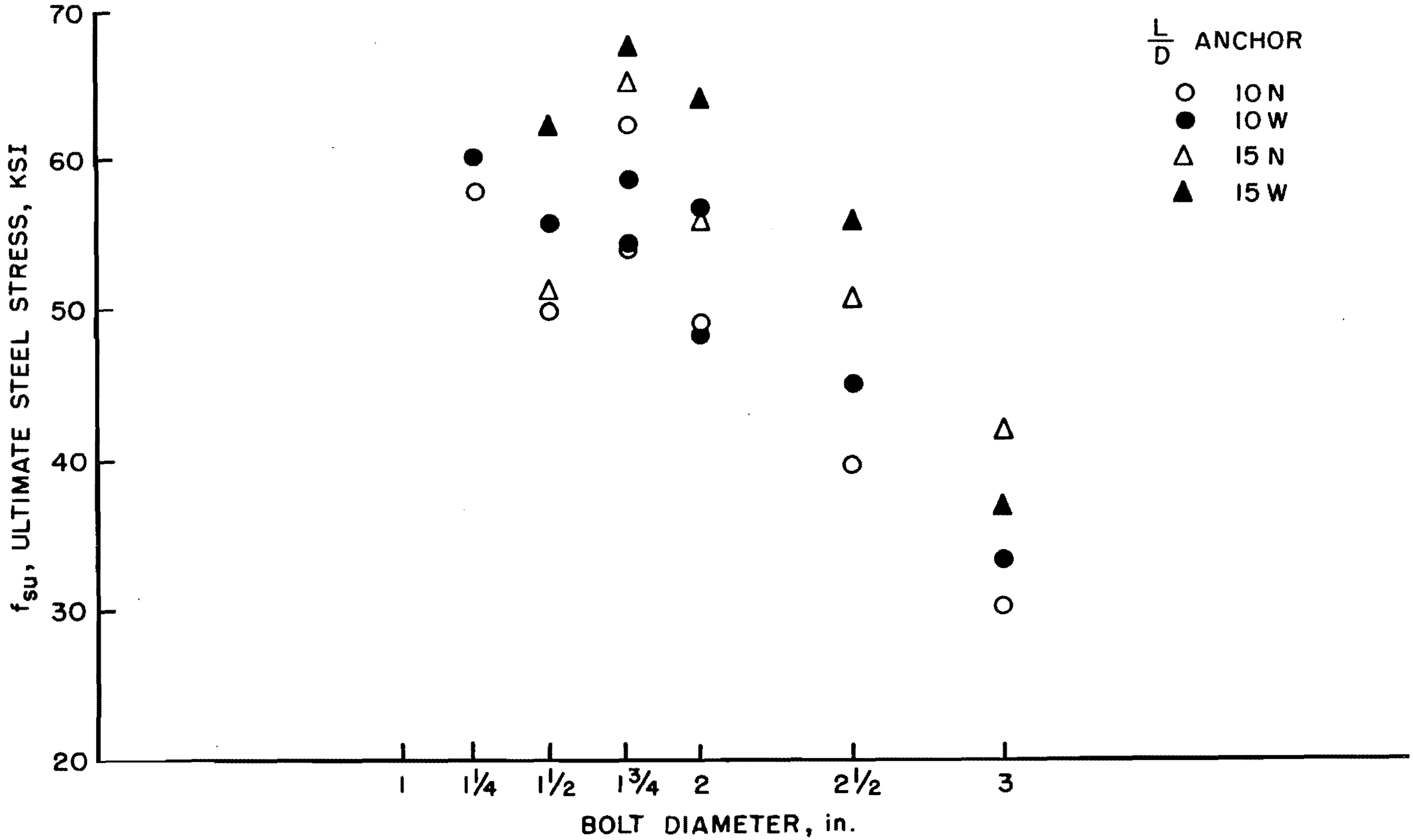


FIG. 25 ULTIMATE STEEL STRESS

stresses under such a triaxial stress condition. However, even considering this, such magnitudes of stress are surprising.

In order to check the general magnitude of the bearing stresses developed by the anchorage devices, a simplified set of computations were made. Since the ultimate bond characteristics of the 1-3/4" bolt without an anchorage device were comparatively small, it was assumed that at failure the total tension force in the bolt was taken by the anchor device. This certainly was true in the cases of the two ultimate tension failures occurring at the anchors inside the specimen. It was probably true in the majority of other specimens. Since only the general magnitude of the bearing stress is sought, no further precision is justified. Equating the tensile force in the bolt to the bearing stresses on the nut or anchor we see that

$$A_s f_{su} = A_c f_c$$

where A_s is the mean area of the bolt, f_{su} is the steel stress on the mean area as tabulated in Appendix A, f_c is the concrete bearing stress, and A_c is the area of the concrete in direct bearing. Therefore,

$$f_c = \frac{A_s}{A_c} f_{su}$$

The ratio A_s/A_c is given in Table 2. It can be seen that the average value of this ratio for all the specimens with nut anchorage is about 0.5. The value for the specimens with washers varied from about 0.18 to 0.36. Using the values for the tests with nuts, we see that the bearing stress developed at ultimate would be approximately one-half of the ultimate steel stress.

From Fig. 26 illustrating ultimate steel stresses it can be seen that computed bearing stresses would range from 15,000 to 32,000 psi. This is for a concrete with an average f_c' of about 4650 psi and hence ranges from 3 to 6 f_c'. This seems to be in reasonable agreement with Abrams' test. It should be noted that in both Abrams' tests and in these specimens, spiral reinforcement was provided. This will be extremely helpful in containing internal splitting and increasing lateral stresses. For the specimens with nut and washer anchorages, the bearing stresses developed are approximately 2 to 3 f_c' if the full area of the washer is used. However, since so many of the specimens displayed only a slight increase in ultimate steel stress when the

TABLE 2

BEARING AREAS

D Nominal Bolt Diameter in.	A_s Mean Stress Area, Sq. in.	A_{cn} Net Nut Area, Sq. in.	A_{cw} Net Washer Area, Sq. in.	$\frac{A_s}{A_{cn}}$	$\frac{A_s}{A_{cw}}$	A_{cr} Critical Stress Area Sq. in.	$\frac{A_s}{A_{cr}}$
1 1/4	0.969	1.82	5.84	0.533	0.166	27.0	0.036
1 1/2	1.405	2.61	7.84	0.538	0.180	26.5	0.053
1 3/4	1.899	4.13	10.14	0.460	0.187	47.9	0.040
2	2.498	6.00	14.59	0.417	0.172	47.0	0.053
2 1/2	3.999	7.29	14.73	0.548	0.272	45.3	0.088
3	5.967	10.48	16.66	0.570	0.358	43.2	0.138

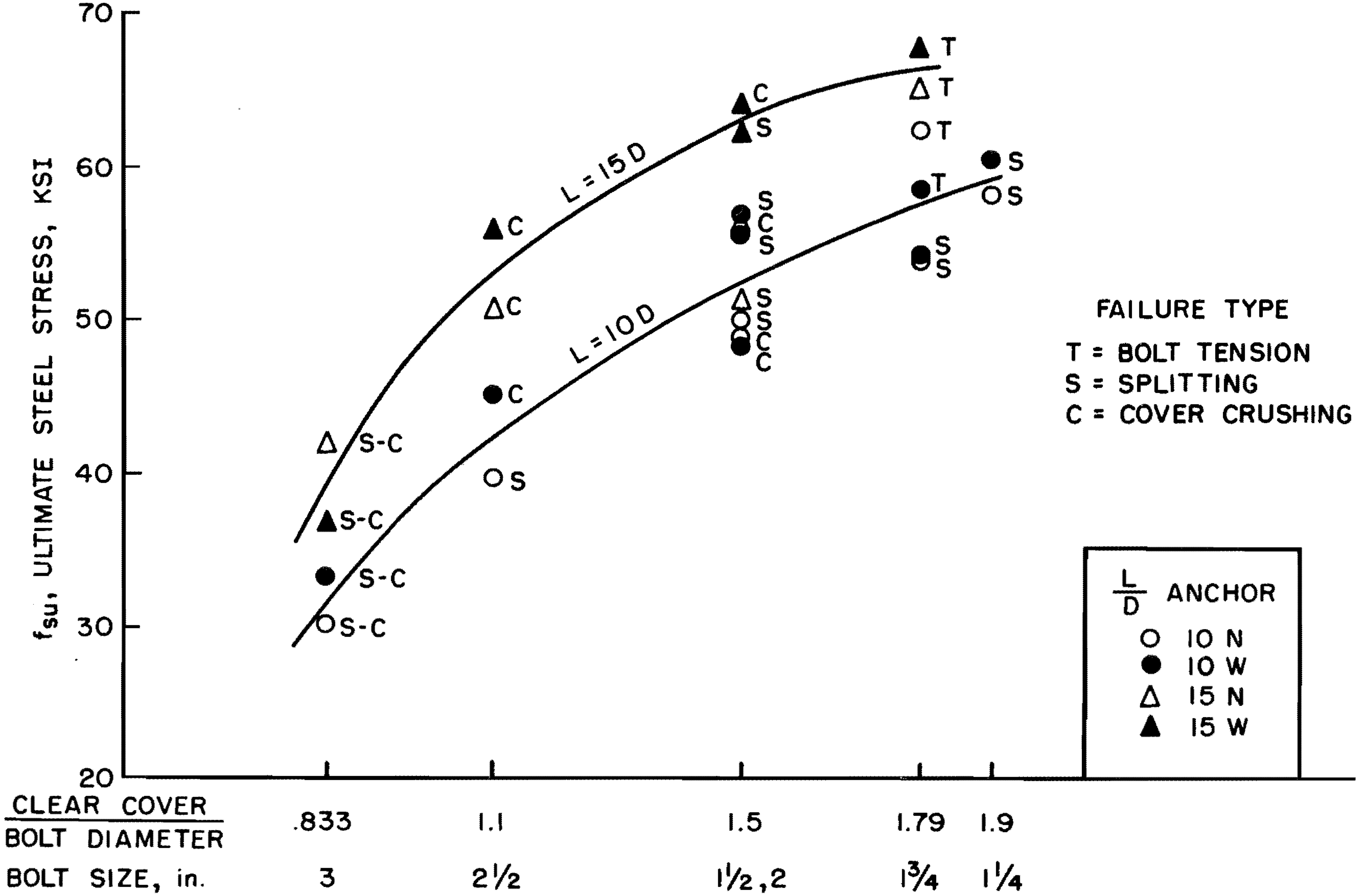


FIG. 26 ULTIMATE STEEL STRESS AS A FUNCTION OF CLEAR COVER

washers were used, it can be concluded that the total area of the washer was not fully effective in developing bearing. Examination of the specimens after failure indicated that the washers were generally bent back over the nuts. This would indicate a diminished effectiveness in bearing.

Bond Stresses

Any single curve for steel stress vs. slip, by a change in vertical scale, can be turned into a bond stress vs. slip curve. However, the change in scale is different for each bolt size and for each embedment length. Because of the way in which the average bond stress has been defined, specimens of identical bolt size but different embedment lengths and with almost identical steel stress vs. slip curves would yield quite different bond stress vs. slip curves. For instance, in Fig. 17 specimens 2-1/2 W 10 and 2-1/2 W 15 have practically identical steel stress-slip curves up to about 30 ksi. Yet, if the bond stress was computed at any given slip, the bond stress for specimen 2-1/2 W 15 would be only 2/3 that for 2-1/2 W 10. This is probably not reflected physically since it is possible that the local bond stresses at a particular point might be almost identical. As a matter of fact, near ultimate, local bond stresses for the two cases are probably close to zero since it is the end anchorage which seems important. Therefore, it was felt unnecessary to present the basic data in terms of bond stresses.

Effect of Clear Cover

It was indicated in connection with the discussion of ultimate strength that one of the main variables in developing the ultimate strength of a given anchor bolt was the effect of the clear cover over the bolt. More precisely, it seems to be the effect of the clear cover over the bolt as related to the diameter of the bolt used. In order to investigate this in more detail, several studies were undertaken to attempt to determine the effect of the clear cover. Unfortunately, in this particular program there was not a great variation of cover to diameter ratios. However, there does seem to be enough information to indicate certain trends.

In the cases in which failures occurred prematurely (that is, before the desired tensile strengths of the bolts were developed) the failure seemed to be one of compression crushing and spalling of the cover immediately over the anchor device. A view of such spalling is indicated in Figs. 27a and 27b.

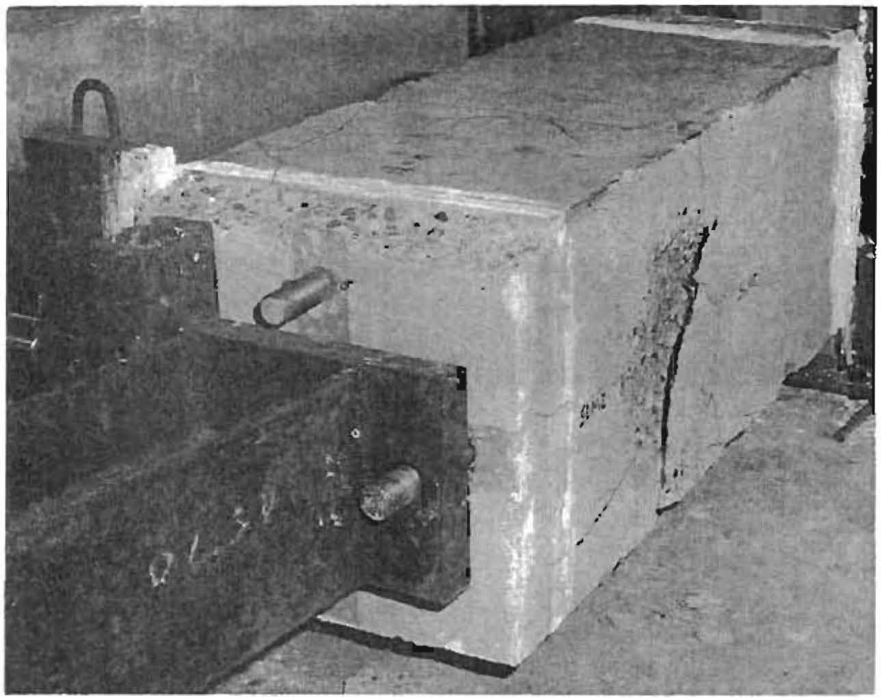


Fig. 27a. Cover spalling

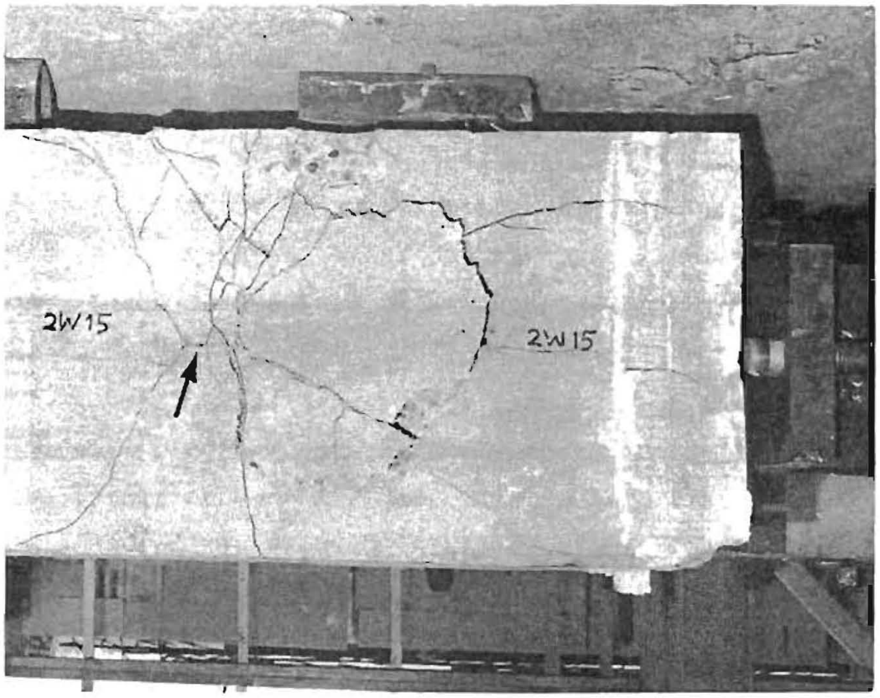


Fig. 27b. Spalling over anchor

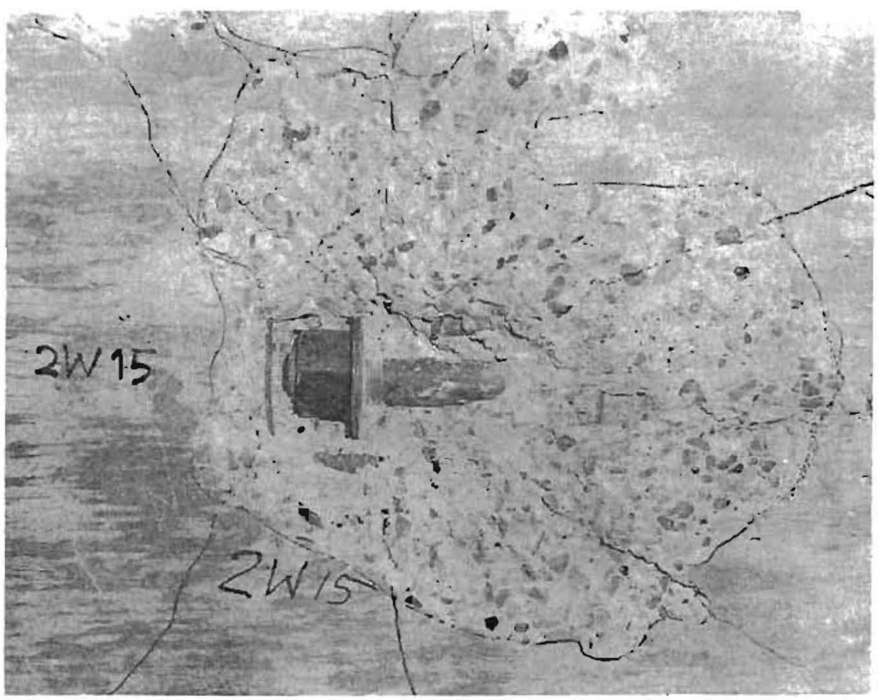


Fig. 27c. Anchor after removal of cover

By matching the actual location of the anchor and the cracking patterns shown in Fig. 27c with those of Fig. 27b, the location of the nut and washer have been indicated. It can be seen that the crushing seems to radiate from the anchor in a pyramidal pattern at angles somewhere between 30 and 45 degrees. Postulating that this same type stress distribution might be taking place in three dimensions, leads to the idea of a truncated cone of stress radiating out from the head.

The following hypothesis is advanced: Assume that the critical stress area for bearing will occur not at the end anchor, but on the base of the cone defined where this cone of stresses intersects the surface of the specimen. This is illustrated in Fig. 28. This area which we will call the critical stress area can be computed as

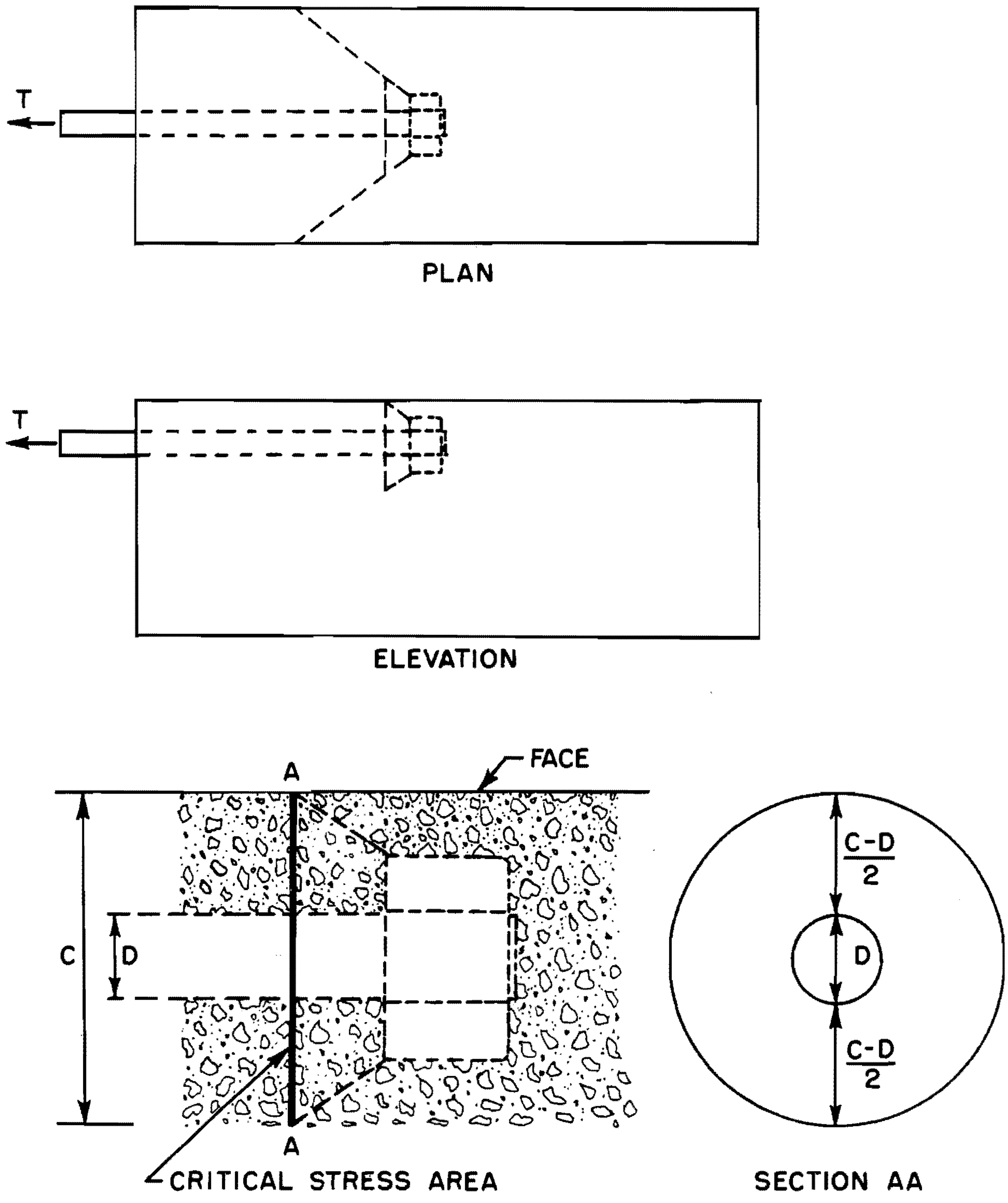
$$A_{cr} = \frac{\Pi}{4}(C^2 - D^2)$$

where $\frac{C}{2}$ is the total cover measured from the center line of the bolt and D is the diameter of the bolt. The values of this critical stress area are given for the various bolt size and cover combinations in Table 2. By recomputing the bearing stresses on this fictious stress area, a new index can be obtained which will include cover. Since

$$f_{su}A_{sm} = f_{cr}A_{cr}$$

$$f_{cr} = \frac{A_{sm}}{A_{cr}} f_{su}$$

The fictious ultimate bearing stresses f_{cr} have been computed and are given in Table A. In Fig. 29 these computed stresses are plotted as a function of the parameter clear cover/diameter. The results tend to group quite well. The data points indicated with the letter D represent specimens in which the tests were discontinued at yielding and represent proof loads. It can be noted that the values of f_{cr} for the low cover/diameter ratios are in the general vicinity of cylinder strength. They represent the larger bolts which failed principally in crushing and spalling. In order to recognize the variation of the concrete strength in the specimens, it was felt that this data should be corrected for variation in cylinder strength. Since much of the failure phenomena involved splitting and since this is affected to a great extent by the tensile strength, any correction for concrete strength



$$A_{cr} = \frac{\pi}{4}(C^2 - D^2)$$

FIG. 28 CRITICAL BEARING AREA HYPOTHESIS

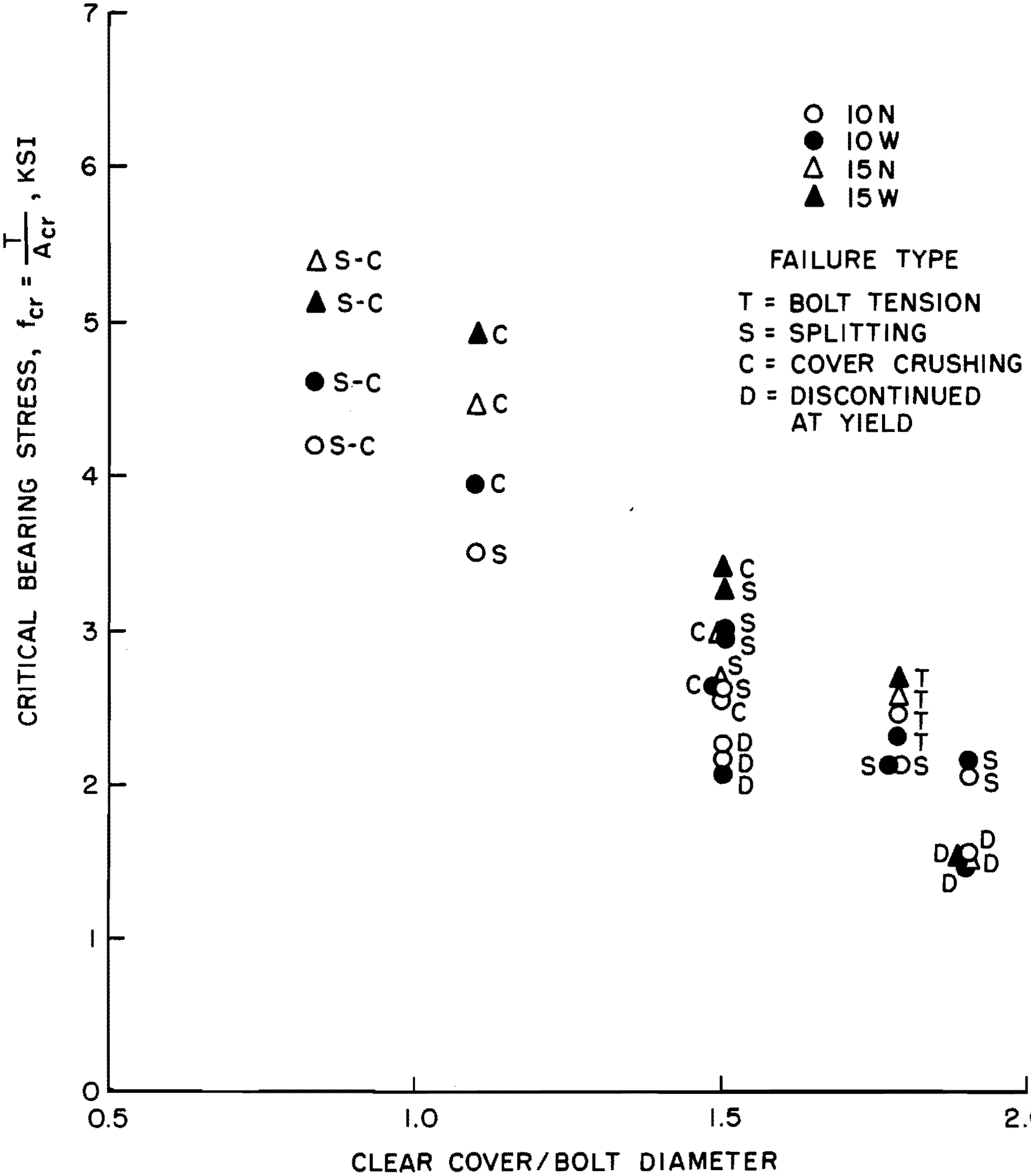


FIG. 29 CRITICAL BEARING STRESS

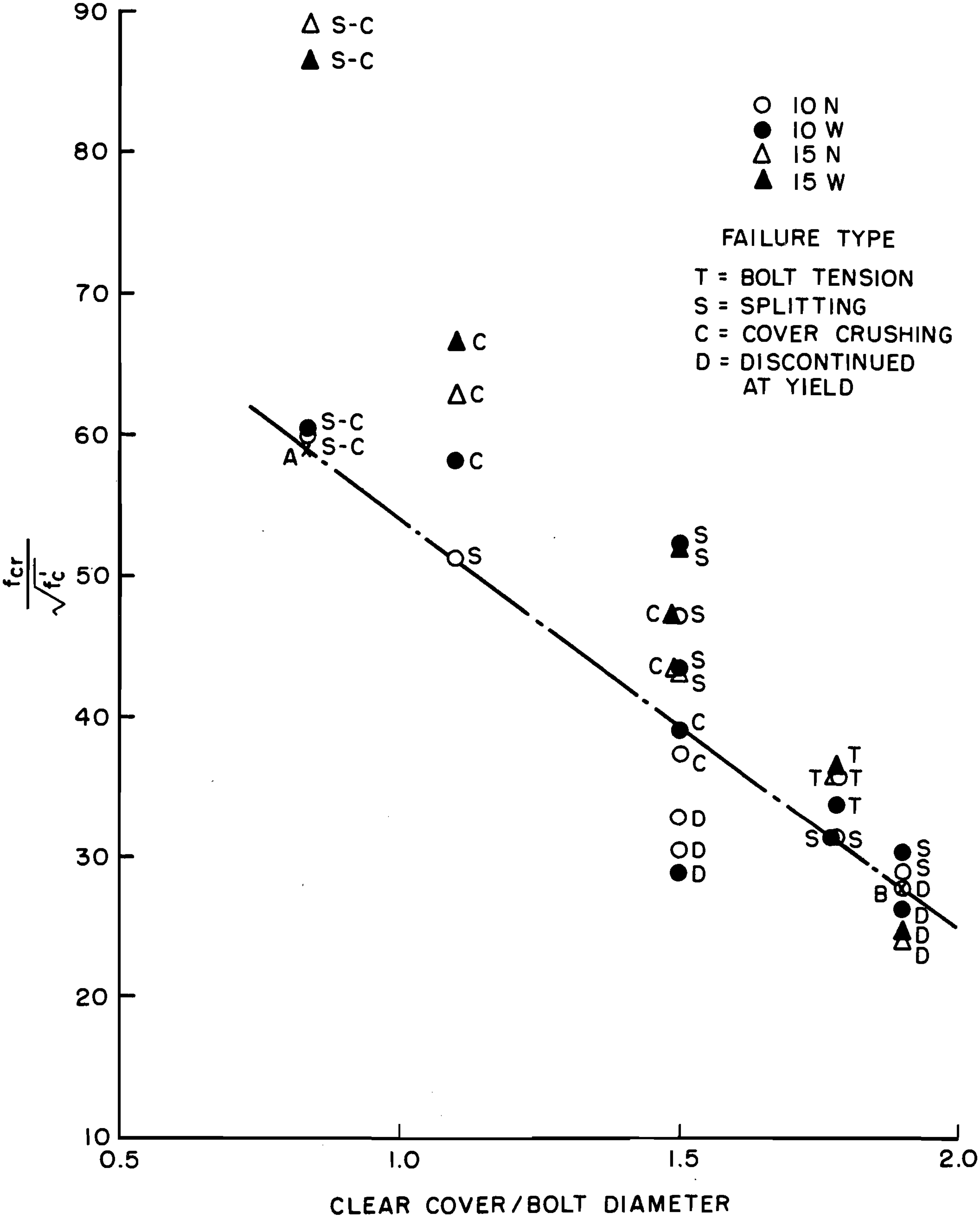


FIG. 30 CRITICAL BEARING STRESS RELATED TO CONCRETE STRENGTH

should possibly reflect the general variation of tensile strength with compressive strength. Since many investigations in bond, shear, diagonal tension, and tensile capacity of concrete have shown that these values tend to vary with the square root of f_c' rather than as a linear function of f_c', this correction was used as an index of the variation of concrete strength. In Fig. 30, the same data are re-expressed as a function of the ratio $f_{cr}/\sqrt{f_c'}$ and the parameter clear cover/bolt diameter. Comparison of Figs. 29 and 30 will indicate that much of the scatter has diminished. This is especially true in the low cover-diameter ratios. The 3" bolt specimens for 10 D and 15 D now plot almost together reducing a great deal of the scatter of the previous figure.

It was felt that the relation shown in Fig. 30 might prove helpful in formulating some rough guides for design criteria. Since it is desirable that any such criteria be on the conservative side, a lower envelope was drawn in the form of the straight line AB indicated on Fig. 30.* This line can be expressed by the equation

$$\frac{f_{cr}}{\sqrt{f_c'}} = 94 - 14\frac{C}{D}$$

In order to have a clearer idea about the relationship proposed, it is necessary to investigate what happens at the limits and what the overall shape of the curve might be. As $\frac{C}{2}$ approaches D, that is, the clear cover approaches zero, if we have a bolt with a finite value of tensile force T, $f_{cr}/\sqrt{f_c'}$ must approach infinity. Similarly as $\frac{C}{2}$ becomes much greater than D, then A_{cr} approaches infinity. If the critical stress area approaches infinity, the stress must approach zero. Hence we see that the over-all form of the relationship expressed should be a hyperbola as shown in Fig. 31. The straight line portion AB previously predicated may only be valid for the very small range of variables for which we have test data. However, a few other points are available. In a pilot series of tests run prior to this investigation, a footing specimen was tested with a 1-3/4" anchor bolt and approximately 10 D embedment. The anchorage was a standard nut and a cut washer. This bolt failed in tension with a nominal steel stress of 76 ksi.

*Note that the points marked as "D" do not represent failure loads and were ignored in selection of the criteria.

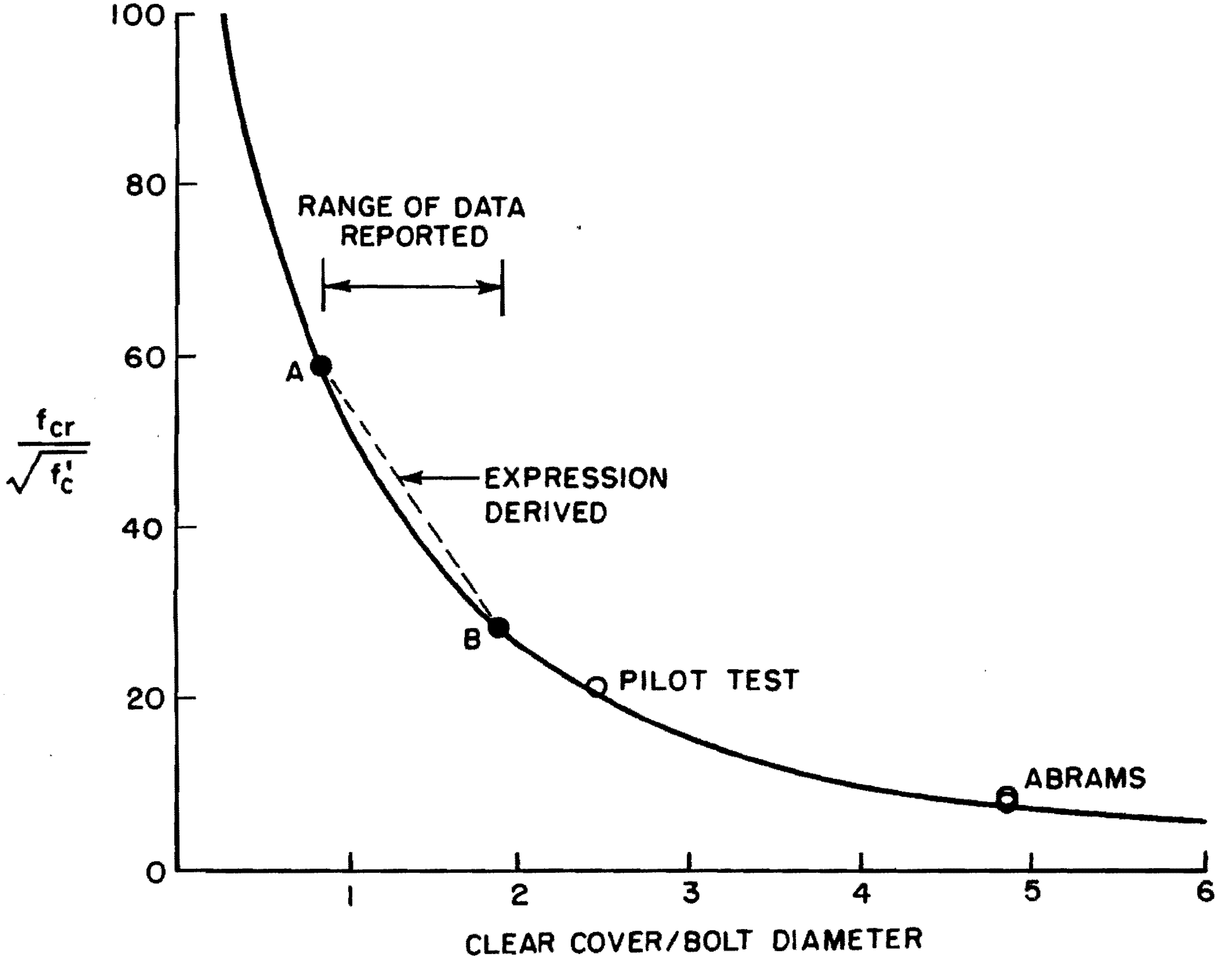


FIG. 31 PROBABLE GENERAL FORM OF CRITICAL BEARING STRESS-COVER RELATIONSHIP

However, in this particular specimen a greater amount of cover was used than in the present series. The clear cover over the bolt was 4-1/4". This makes the ratio clear cover/diameter equal to 2.43. The computed f_{cr} = 1640 psi. The concrete strength was 5734 psi and hence the ratio $f_{cr}/\sqrt{f_c'}$ = 21.5. This point is plotted on Fig. 31. In addition, in Abrams' original set of pullout specimens he tested some 3/4" bars with a nut and washer anchorage embedded for 8" in a 8" diameter concrete specimen. The length of embedment is again approximately 10 diameters. For these specimens the ratio clear cover/diameter equals 4.84. The specimens failed by splitting of the concrete. He reports only average bond stresses but from these it can be calculated that f_{cr} = 349 psi for the specimens with nuts and 385 psi for the specimens with nuts and washers. The cylinder strength would be approximately 1860 psi, hence the ratios $f_{cr}/\sqrt{f_c'}$ = 8.1 and 8.95 respectively. These values are also shown in Fig. 31 and tend to indicate the general trend of the results. However, it must be emphasized that any conclusions as to the desirable amount of cover deduced from these results are extremely approximate and should be considered within the limits of test data available in developing these expressions. As in all empirical expressions a great danger exists in extrapolating limited test results. However, in this case it is felt that an attempt should be made to give some guidance as to the magnitude of adjustment of cover in order to increase anchor bolt performance. Further testing should be done to validate such changes.

If we accept the validity of this expression over a limited range we can compute some desirable cover dimensions. Since

$$f_{cr} = \frac{A_s f_s}{A_{cr}} = (94 - 14\frac{C}{D})\ \sqrt{f_c'}$$

and, since for design f_s should equal f_y,

$$A_{cr}\ (94 - 14\frac{C}{D}) = \frac{A_{sm} f_y}{\sqrt{f_c'}}$$

If we work with design values of f_y = 33,000 and f_c' = 3,000 then $f_y/\sqrt{f_c'}$ = 600. Substituting these values along with the value for the critical area, we can obtain the expression

$$(7.35 - 1.1\frac{C}{D})\ (C^2 - D^2) = 60A_{sm}$$

For each standard bolt there is a fixed relation between D and A_{sm}. Substituting these in the previous equation we can get an expression which is a cubic function of C. Solving this numerically and obtaining C to the nearest quarter inch we obtain the following results.

D	C Calculated	C Actual
1.25	4.25	6*
1.5	5.00	6*
1.75	6.00	8
2.00	6.75	8
2.50	8.50	8
3.00	10.75	8

A comparison of the minimum desired cover and the cover utilized indicates that the present cover is satisfactory except for the 2-1/2" and 3" bolt sizes. This would be expected from the nature of the test results. The specimens which suffered from apparent lack of cover were the largest sizes. The results would indicate that the 2-1/2" bolt cover should be increased slightly. (The cover from the center of the bolt to the outer edge is equal to C/2.) This indicates that the cover should be increased from 4 to 4-1/4". However, for the 3" bolt a more substantial change is indicated. The present 4" cover from the center of the bolt should be increased to approximately 5-1/2". Accepting the general limitations of the method, it is felt that this will give a general idea of how to compute the required cover over the bolt. While a different set of relationships could be developed for the 15 D embedment lengths, it is not felt that the precision of the method warrants it. In preliminary computations such an expression was developed and it was found that virtually no changes in cover were indicated other than those found by the preceding expression.

Pilot Tests-Bent Bolts

In order to get a general idea of what the effect might be of bending the bolt so as to get more cover over the anchorage zone, a limited series of tests were run. In this series a single footing specimen with 4 bolts was

*Present design standards actually use C = 8 but in the test specimen C = 6.

used. SA was a standard 1-3/4" bolt 10 D embedment and a standard nut. Bolt SB was the same as SA except for the 30^{o} bend at midpoint. Bolt SC was similar to bolt SB except that a 45^{o} bend was put in the bolt. Bolt SD simply had 17-1/2" of plain unthreaded bar with no anchorage. The bent part of the bolts were placed along a radius towards the center as illustrated in Fig. 4. The results of this series is shown in Fig. 19. It has been previously mentioned that the specimen with no end anchorage simply slipped out at a very low stress level. Specimen SA behaved the same as the other specimens with similar embedment. Specimens SB and SC indicated a slightly stiffer load-slip response. However, when the values are compared to those shown in Fig. 15 for the similar 1-3/4" bolts, it will be noted that they did not develop as high an ultimate strength. Possibly this was due to very severe internal splitting. The higher bearing stresses developed in the other specimens could not be obtained. Another possibility is that the bend in the bar reduces the effective embedment length and causes a stress raiser with an adverse effect. Unfortunately the results of this series must be regarded as somewhat inconclusive in that while the bending of the bolt seems to give somewhat better service load characteristics, the ultimate strengths found were lower than in the unbent anchor bolts.

Pilot Tests-High Strength Bolts

A very limited series of tests of high strength bolts was included in the over-all program. The bolts used for specimens 1-3/4 N 10a and 1 3/4 W 10a had no well defined yield point as shown from the stress-strain curve in Fig. A 2. Their yield strength based on a 0.002 offset was 91 ksi. As can be seen in Fig. 15 the specimens were loaded until end slips of greater than 0.02 inches developed. Shortly after this the specimens failed by splitting. Both bolts developed an ultimate steel stress of 54.1 ksi. Their companion specimens with A7 bolts (1-3/4 N 10b and 1-3/4 W 10b) were the pair of specimens in which the bolt failed in the thread area at the face of the anchor nut. The latter pair developed ultimate steel stresses of 62.7 and 58.9 ksi. Since the high strength bolts did not develop as high an ultimate stress as the A7 bolts due to failure of the concrete, it might be initially felt that there is no advantage to their use. However, design for A7 bolts would be based on a minimum yield point of 33 ksi, which is less than 60% of the ultimate steel stress developed. Design for higher strength bolts would be based on a higher stress level and could be based on the more realistic limit of concrete control. However, it is important to note the observed reduction in ductility when high strength bolts are used. This might be adverse in some structures.

CHAPTER 4
DISCUSSION OF RESULTS AND CONCLUSIONS

General Behavior

The testing of a group of anchor bolts in footing specimens similar to those in use with sign structures has indicated certain important trends in behavior. Observations of gross end slip developed at yield levels of computed steel stress indicate that with an embedment length of 10 diameters the load transfer is primarily through bearing of the end anchor against the concrete. With an embedment length of 15 diameters an increased portion of the load is transmitted through bond. However, the increase in load developed is much less than the increase in embedment length.

The failure patterns noted indicated that the specimens could fail either in tension by exceeding the ultimate tensile capacity of the bolts, by splitting of the concrete block along the bolt axis or by crushing and spalling of the concrete immediately over the end anchorage of the bolt. In some cases a combination of these failures occurred. In almost all cases yielding of the bolt occurred before final failure.

Comparison of the test results with a pilot test of a bolt with no end anchorage indicated that the end anchorage is extremely important. It appears that a standard nut will be sufficient anchorage on most bolts. Results of tests with a combination nut and washer indicate only minor increase in load capacity at ultimate. However, it is possible that the effect of heavier washers would have been more significant. In the specimens with a low ratio of cover/bolt diameter, the addition of the washer seems to have had a detrimental effect. A limited series of tests with bent bolts were inconclusive.

Conclusions and Recommendations

The test series included anchor bolts ranging from 1-1/4" to 3" in diameter with embedment lengths of 10 and 15 diameters. The bolts were anchored with a standard nut or a combination of a standard nut and washer. All specimens had a #2 intermediate grade spiral at 6" pitch. Only a limited range of clear cover was investigated. All conclusions must be restricted to this range of physical dimensions.

The following conclusions were made:

1. The pullout tests indicated that A7 (33 ksi yield point) steel anchor bolts can be fully developed with a 15 diameter embedment length in all bolt sizes. (Steel stress computations are based on the mean diameter stress area.)
2. Anchor bolts of A7 (33 ksi yield point) steel with diameters of 2-1/2" or less can be fully developed with a 10 diameter embedment length.
3. An important variable affecting the developable tensile strength is the amount of cover over the anchor. Results of calculations based on a limited empirical expression indicate that the clear cover should be increased for the 2-1/2" and 3" diameter bolts over the 2-3/4" and 2-1/2" used in these tests.
4. Considering a 0.01" gross end slip at service loads as a limit, the following mean area steel stresses were developed in 4650 psi concrete: (see Fig. 24)

D in.	10 D Embedment	15 D Embedment
1 1/4	28 ksi	32 ksi
1 1/2	26 "	24 "
1 3/4	20 "	24 "
2	16 "	21 "
2 1/2	19 "	16 "
3	18 "	15 "

It should be noted that because the development of these anchor bolts depended primarily on end anchorage, the longer embedment lengths often show larger end slips due to the elastic elongation of the partially unbonded steel bolt. The slip limit of 0.01" may not be realistic for service conditions anticipated. Possibly any such limit should vary with the size of the bolt. A similar table can be prepared for any other slip criteria.

As a result of the evaluation of this test series, it is recommended:

1. The clear cover over the large bolts (2-1/2" and 3" diameter) should be increased if possible. The clear cover should be at least as great as that used with the smaller (1-3/4" and 2" D) bolts. A more detailed recommendation for determining the cover required is given in Chapter 3.
2. Additional specimens should be tested to investigate the effect of clear cover, particularly with respect to the development of large bolts. The data provided by this program is quite limited in this respect.
3. Consideration should be given to the effect of repeated loadings on the anchorage characteristics. Literature searches have indicated that very little information is available in this area.
4. Further study should be given to the use of higher strength steel bolts. The limited pilot tests indicate these may be more efficient at service load levels than the larger bolts. It is possible that some of the cover modifications associated with the very large diameter bolts could be partially avoided by providing reduced diameter, high strength bolts.

REFERENCES

1. Abrams, D. A., "Tests of Bond Between Concrete and Steel," University of Illinois Engineering Experiment Station Bulletin No. 71, December, 1913, pp. 97-99.

2. Ferguson, P. M., Turpin, R. D., and Thompson, J. N., "Minimum Bar Spacing as a Function of Bond and Shear Strength," Journal of the American Concrete Institute, Vol. 50, June 1954, pp. 869-888.

3. Ferguson, P. M., Breen, J. E., and Thompson, J. N., "Comparative Bond Efficiency of Large High-Strength Deformed Bars," report to the Committee of Concrete Reinforcing Bar Producers, American Iron and Steel Institute, February 1962, pp. 66.

4. Texas Highway Department, "Standard Specifications for Road and Bridge Construction," January 2, 1962.

5. American Society for Testing Materials, Tentative Specification for "Low Carbon Steel Externally and Internally Threaded Standard Fasteners," ASTM Designation: A 307-58T, 1958 Standards, Part I, p. 749.

6. Slaughter, E. M., "Tests on Threaded Sections Show Exact Strengthening Effect of Threads," Metal Progress, Vol. 23, No. 3, March 1933, pp. 18-20.

7. American Standard ASA B18.2-1960 "Square and Hexagon Bolts and Nuts," published by American Society of Mechanical Engineers, p. 17, p. 26.

8. American Standard ASA B27.2-1958 "Plain Washers," published by American Society of Mechanical Engineers, p. 7, pp. 9-10.

APPENDIX

TABLE A

SUMMARY OF DATA

Specimen	f_c' ksi	f_y ksi	f_s at Slip= 0.01 ksi	f_s at Slip= 0.02 ksi	Slip at $f_s = 20$ in.	Slip at $f_s = 33$ in.	f_{su} ksi	Failure*	f_{cr} psi	$\frac{f_{cr}}{\sqrt{f_c'}}$	Clear Cover / Diameter
1-1/4N10a	3.16	33.7	30	40	.005	.012	43.5	D	1560	27.9	1.9
1-1/4N10b	5.04	"	32	39	.004	.011	58.0	S	2085	29.3	"
1-1/4W10a	3.16	"	37	—	.002	.007	41.2	D	1480	26.5	"
1-1/4W10b	5.04	"	27	39	.006	.014	47.3	D	1700	24.0	"
1-1/4W10b-2	5.04	"					60.3	S	2170	30.6	"
1-1/4N15a	4.04	"	33	40	.005	.010	41.9	D	1520	24.0	"
1-1/4W15a	4.04	"	37	42	.002	.007	42.7	D	1540	24.5	"
1-1/2N10a	3.16	37.0	26	40	.008	.013	50.0	S	2640	47.2	1.5
1-1/2N10b	5.04	"	33	40	.005	.010	41.1	D	2170	30.6	"
1-1/2W10a	3.16	"	32	41	.004	.010	41.6	D	2200	39.2	"
1-1/2W10a-2	3.16	"					55.8	S	2950	52.8	"
1-1/2W10b	5.04	"	23	39	.008	.014	38.9	D	2060	29.0	"
1-1/2N15a	4.04	"	25	37	.007	.016	41.1	D	2170	34.5	"
1-1/2N15a-2	4.04	"					51.1	S	2700	43.2	"
1-1/2W15a	4.04	"	33	41	.003	.010	62.2	S	3280	52.2	"
1-3/4N10a	4.66	91.0	22	34	.008	.019	54.1	S	2140	31.6	1.785
1-3/4N10b	4.74	38.0	20	32	.010	.021	62.7**	T	2480	36.0	"
1-3/4W10a	4.66	91.0	24	34	.008	.018	54.1	S	2140	31.5	"
1-3/4W10b	4.74	38.0	28	38	.004	.014	43.8	D	1740	25.3	"
1-3/4W10b-2	4.74	38.0					58.9**	T	2340	34.0	"
1-3/4N15a	5.30	40.5	24	38	.008	.015	51.6	D	2050	28.2	"
1-3/4N15a-2	5.30	40.5					65.3**	T	2590	35.6	"
1-3/4W15a	5.30	40.5	33	42	.006	.010	67.9**	T	2690	36.7	"

*T = Bolt tension; S = Splitting; C = Crushing over anchorage; D = Discontinued after apparent yield; SL = Bolt sliding out.

**Average tensile ultimate of 3 companion specimens—63.8 ksi.

TABLE A (con't)

SUMMARY OF DATA

Specimen	f_c' ksi	f_y ksi	f_s at Slip= 0.01 ksi	f_s at Slip= 0.02 ksi	Slip at $f_s = 20$ in.	Slip at $f_s = 33$ in.	f_{su} ksi	* Failure	f_{cr} psi	$\frac{f_{cr}}{\sqrt{f_c'}}$	Clear Cover / Diameter
2N10a	4.66	36.7	18	32	.011	.021	42.5	D	2260	33.2	1.5
2N10b	4.74	30.6	24	34	.006	.018	38.5	D	2045	29.7	"
2N10b-2	4.74	30.6					49.0	C	2600	37.8	"
2W10a	4.66	36.7	20	34	.010	.018	50.3	D	2670	39.3	"
2W10a-2	4.66	36.7					48.3	C	2570	37.8	"
2W10b	4.74	30.6	24	31	.006	.024	38.5	D	2045	29.8	"
2W10b-2	4.74	30.6					56.8	S	3020	43.8	"
2N15a	5.17	33.5	21	29	.009	.027	41.8	D	2220	43.2	"
2N15a-2	5.17	33.5					56.2	C	2990	41.5	"
2W15a	5.17	33.5	28	37	.007	.016	45.7	D	2430	33.6	"
2W15a-2	5.17	33.5					64.0	C	3400	47.3	"
2-1/2N10a	4.63	30.7	19	34	.011	.019	39.7	S	3510	51.5	1.1
2-1/2W10a	4.68	"	25	30	.006	.024	45.0	C	3980	58.4	"
2-1/2N15a	5.06	"	16	30	.011	.024	30.7	C	4470	63.0	"
2-1/2W15a	5.46	"	24	32	.006	.021	55.9	C	4940	66.8	"
3N10a	4.88	45.0	21	26	.008	—	30.4	S-C	4200	60.1	0.833
3W10a	4.75	"	19	24	.012	—	33.3	S-C	4600	60.8	"
3N15a	4.18	"	23	31	.008	.023	42.0	S-C	5800	89.2	"
3W15a	3.55	"	15	23	.016	.046	36.9	S-C	5100	86.5	"
1-3/4SD10	4.90	35.5	12	12	—	—	16.1	SL	640	9.1	1.785
1-3/4SA10	4.44	"	16	32	.012	.022	36.5	D	1450	21.8	"
1-3/4SA10-2	4.90	"					43.3	S	1720	24.6	"
1-3/4SB10	4.44	"	29	—	.007	.012	34.0	S	1350	20.2	"
1-3/4SC10	4.44	"	21	41	.010	.015	44.1	S	1750	26.2	"

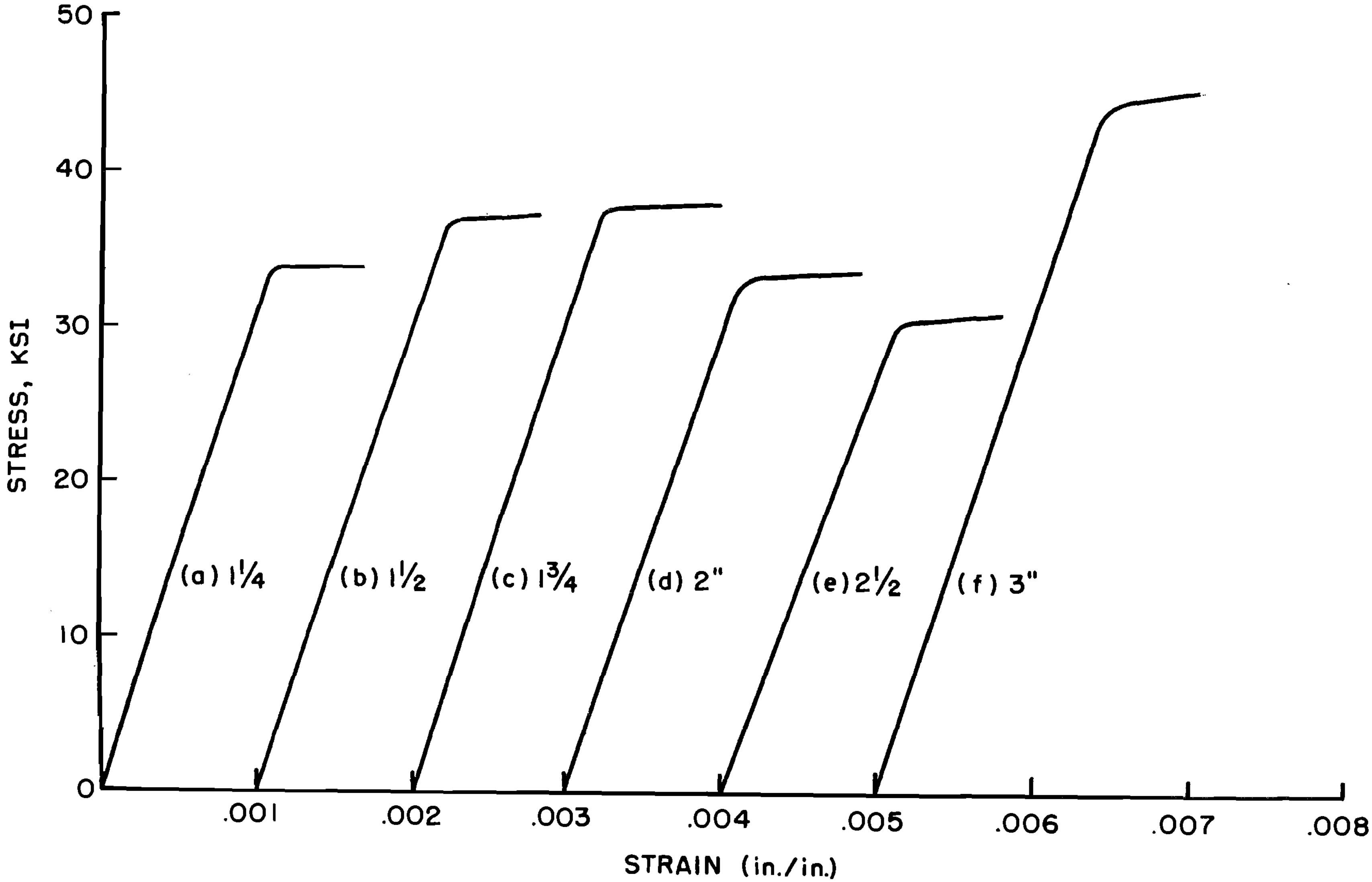


FIG. A1 MILD STEEL ANCHOR BOLT STRESS STRAIN DIAGRAMS

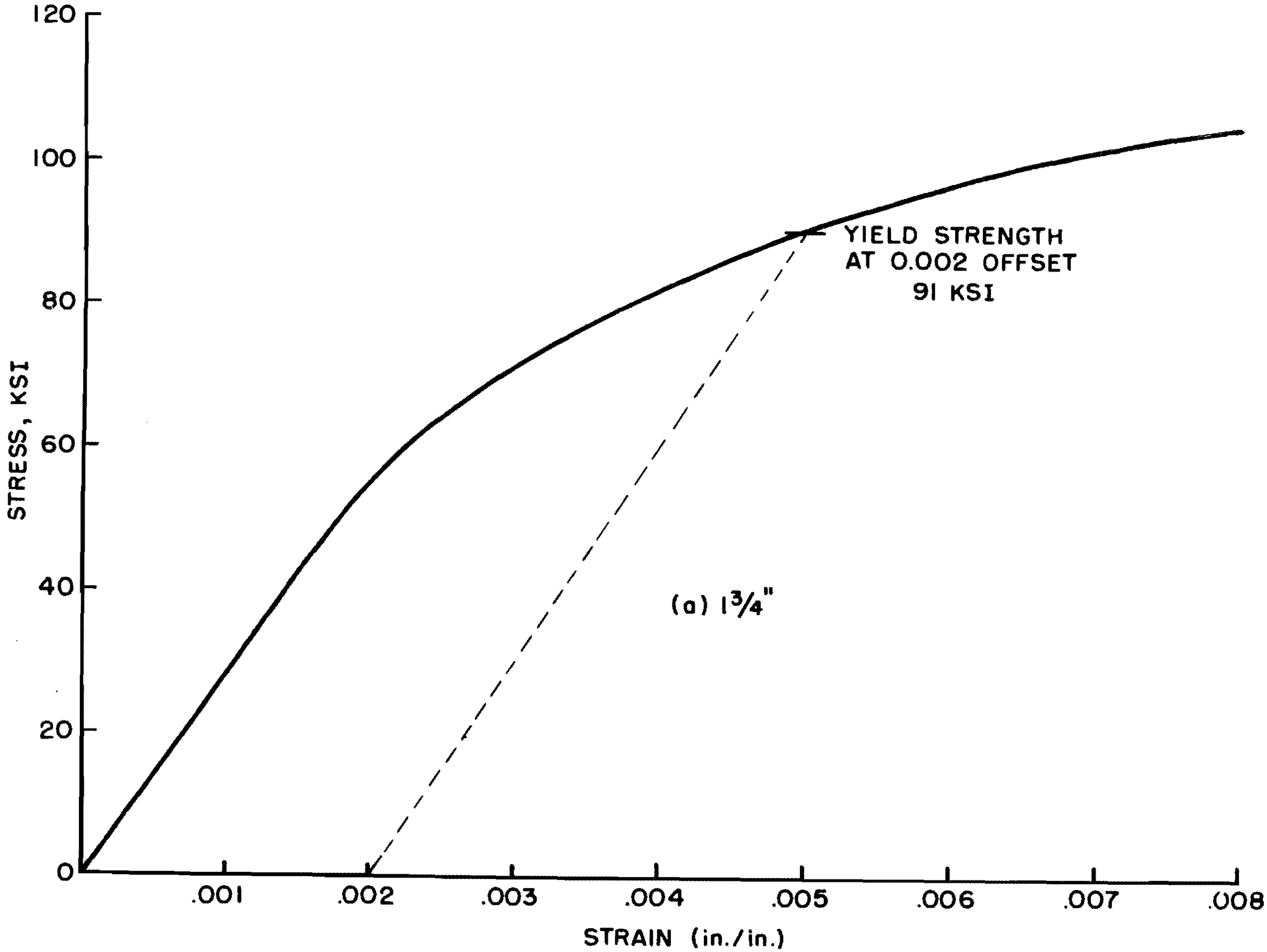


FIG. A2 HIGH STRENGTH ANCHOR BOLT STRESS STRAIN DIAGRAM