

# Development Length for Anchor Bolts

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This paper summarizes findings of a research program to study design criteria for the proper lengths for anchor bolt embedment into drilled shafts. The program included a series of tests on 36 anchor bolts ranging from  $1\frac{1}{4}$  to 3 in. in diameter which were embedded near the edge of a square shaft specimen and were under a combination of flexural tension, bond, and splitting conditions approximating those in the prototype. Loaded-end slip behavior was determined at nominal working loads, at first yielding, and in many cases at ultimate capacity. Important trends affecting design practices are indicated.

•THE GENERAL purpose of this investigation was to establish the development length or the embedment length required to develop the tensile yield capacity of ASTM A 7 anchor bolts. At the same time it was desired to explore the effect of various types of end anchorages and certain other variables. A substantial amount of recent test data have been compiled on the proper development length for ASTM A 305 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 Abrams (1) which involved plain bars anchored with nuts and without washers. As has been shown by Ferguson et al. (2), the results of such concentric pullout tests, in which the concrete around the bolt is under compression, do not reflect the bond-slip behavior of elements in the tension zone of beam or pier members, such as the prototype sign support drilled shaft foundations.

In order to simulate the behavior found in typical sign support structures, the test specimen shown in Figure 1 was developed. The bolt is embedded near the edge and tension is applied through a flexural loading. In this type of specimen, the concrete around the anchor bolt is under a combination of tension, bond, and splitting conditions approximating those in the prototype structure. However, the shear distribution is different because of the absence of a soil pressure distribution.

This investigation reports on tests of 12 shaft specimens containing a total of 36 bolts. The anchor bolt sizes ranged from  $1\frac{1}{4}$  in. to 3 in. 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. Each bolt in a given shaft specimen was loaded in turn until yielding occurred, and after all bolts in a specimen had yielded several bolts were reloaded until ultimate was attained.

## SPECIMENS

All of the specimens used in this program were of the same general type. When  $1\frac{1}{4}$ -in. through 2-in. diameter bolts were used, 4 anchor bolts were embedded in each specimen. However, only 2 bolts were used with the  $2\frac{1}{2}$ -in. and 3-in. sizes. The individual specimens are designated by a 4-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

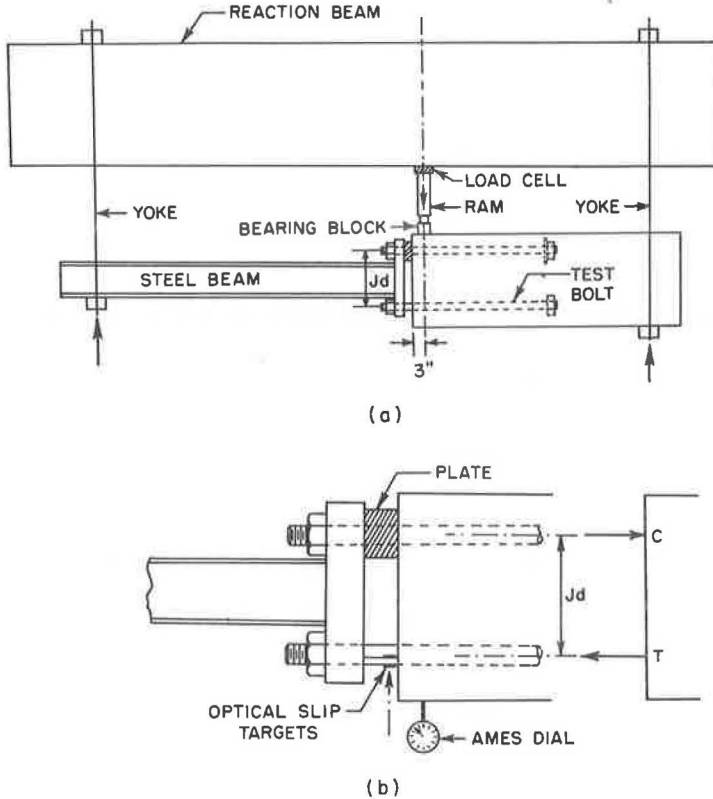


Figure 1. Test arrangement.

the designation of individual companion specimens. For example, the designation  $1\frac{1}{4}$  W 10a designates a specimen in which a  $1\frac{1}{4}$ -in. bolt with a nut and washer anchorage was embedded for 10 diameters. The a indicates that it was the first of this type tested. Figures 2, 3 and 4 show the entire set of test specimens with critical dimensions. A tabulation of the important variables and summary of the test results is provided in the Appendix.

## MATERIALS

### Concrete

A concrete conforming to Texas Highway Department "Class A" concrete was used (3). The  $1\frac{1}{4}$ -in. and  $1\frac{1}{2}$ -in. specimens were cast with a job-mixed concrete. The  $1\frac{3}{4}$ -in. through 3-in. specimens were cast with a ready-mixed concrete. High-early-strength cement (type III) was used with a cement factor of 5 sacks per cubic yard and a slump range of 3- to 4-in. Maximum aggregate size was 1 in. In general, the water-cement ratio was 7 gallons per sack.

### Anchor Bolts

The anchor bolts were specified to be manufactured from ASTM A 7 steel with a minimum yield strength of 33 ksi. The bolts were fabricated locally with USS threads from round bar stock and were not galvanized. Figure 5 shows typical stress-strain curves for the bolts used. In one set of  $1\frac{3}{4}$ -in. bolts, the shape of the stress-strain curve as shown in Figure 6 indicated that a high carbon or other alloy steel was substituted and the yield point was 91 ksi.

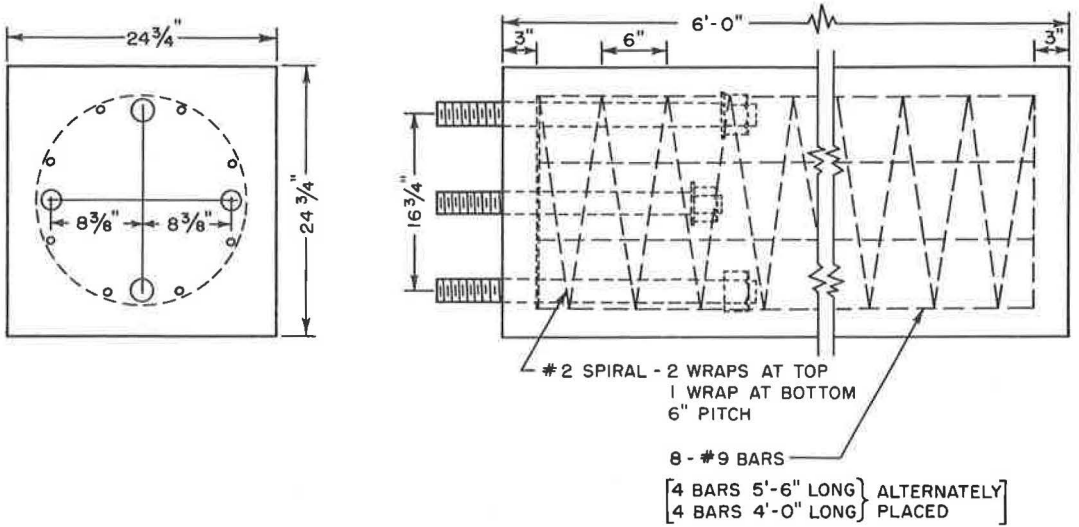
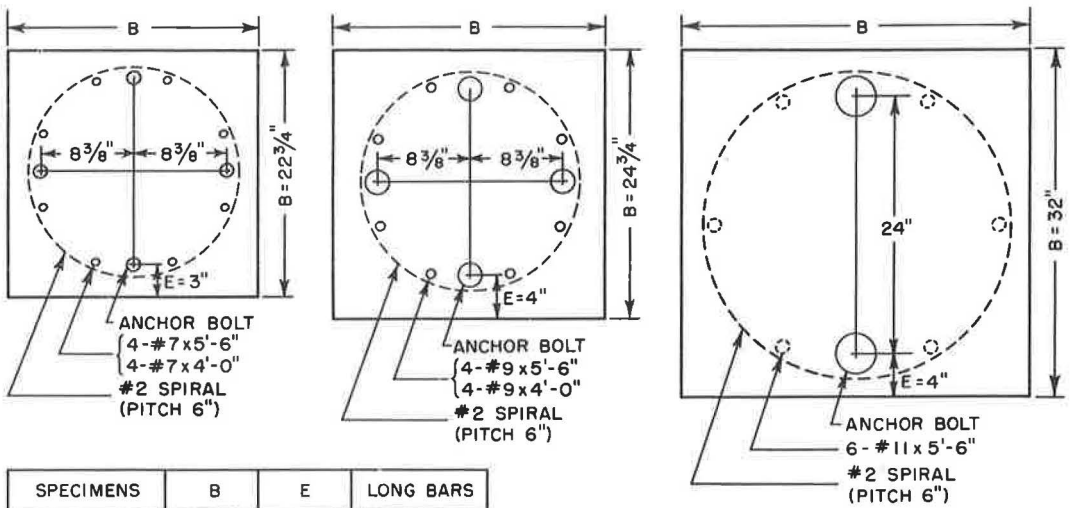


Figure 2. Typical specimen detail.



SPECIMENS	B	E	LONG BARS
ALL 1 1/4", 1 1/2"	22 3/4"	3"	8 - #7
ALL 1 3/4", 2"	24 3/4"	4"	8 - #9
ALL 2 1/2"	32"	4"	8 - #9
ALL 3"	32"	4"	6 - #11

Figure 3. Specimen details.

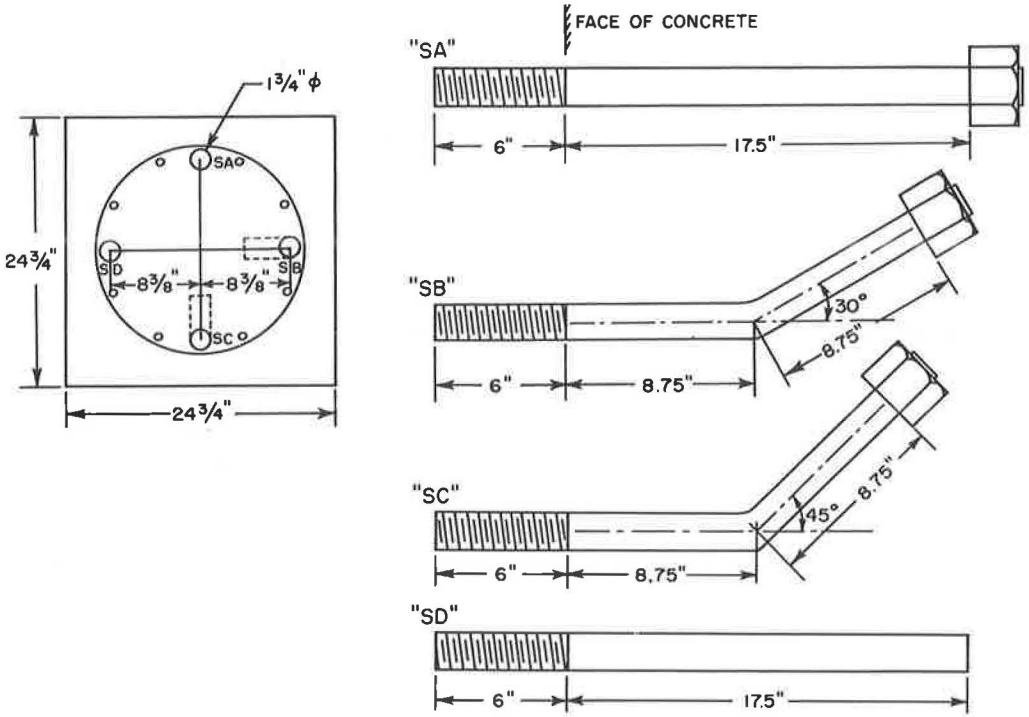


Figure 4. Details of special anchorage, series "S".

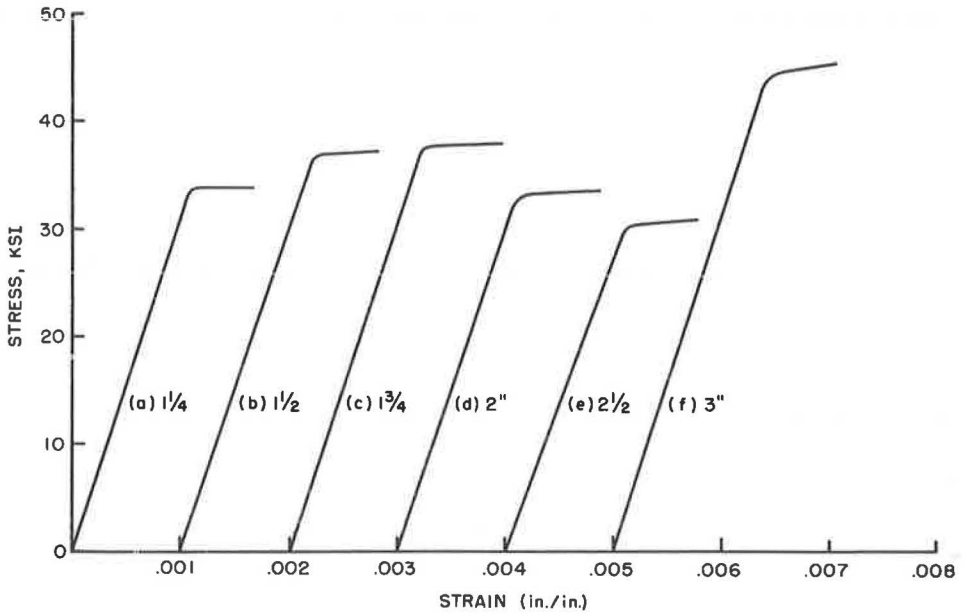


Figure 5. Mild steel anchor bolt stress strain diagrams.

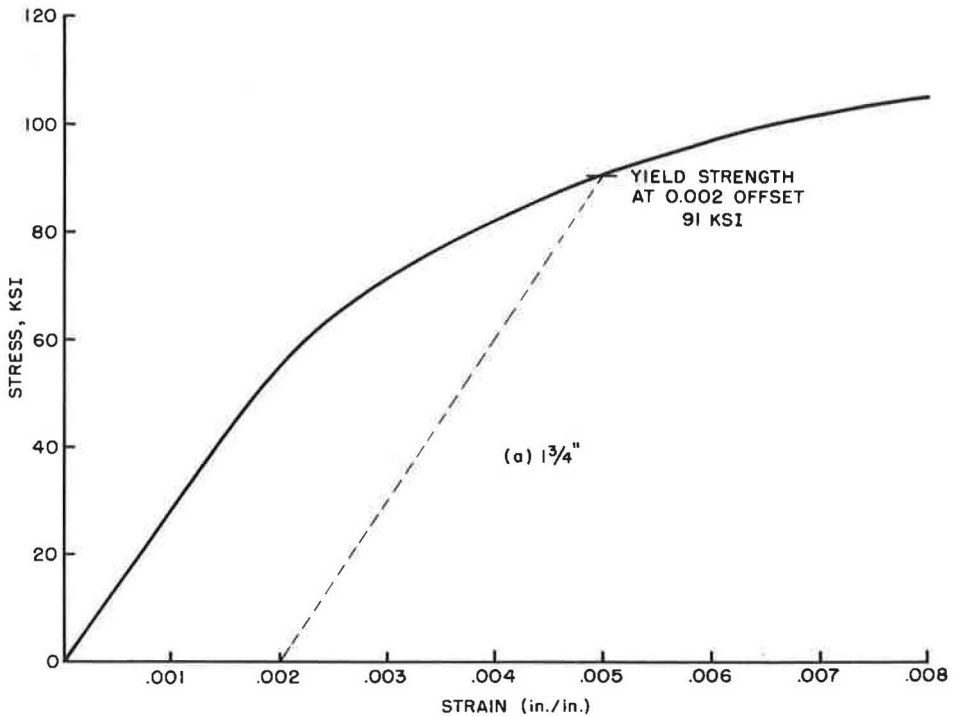


Figure 6. High strength anchor bolt stress strain diagram.

### Reinforcing Steel

The main reinforcing steel used met ASTM A 305 and A 15 intermediate grade specifications. The spirals were made from the same grade steel.

### Forms

All the specimens were cast in wooden forms as shown in Figure 7. The bolts were cast in a vertical position at the top of the forms. In all specimens the overall height was maintained at 6 ft to insure that the water gain effect would remain relatively constant in all tests and would closely approximate that found under field conditions.

### Casting Procedure

The concrete was vibrated into place with a large internal vibrator. Standard compression test cylinders were cast from the batch which was placed in the top half of the specimen. Because of the 3- to 4-in. slumps used, good compaction was obtained.

### Curing

The specimens were troweled and then left with a piece of moist burlap placed over the open end of the form. On the sixth day, the burlap and side forms were removed and the specimen was prepared for testing. At no time was the specimen lifted by the embedded anchor bolts.

### Test Procedure

A calibrated hydraulic ram (A) and an electronic load cell (B) were inserted between the specimen and a reaction beam as shown in Figures 1 and 8. The ram load was distributed to the specimen through a thick steel bearing plate (C). Then a steel wide-

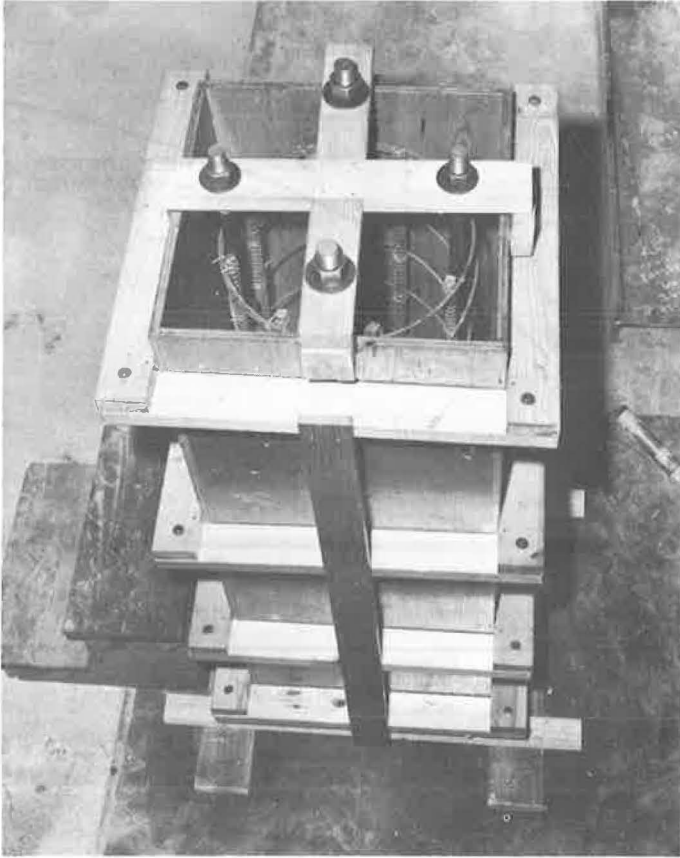


Figure 7. Specimen prior to concrete placement.

flange section (D) was attached. Reaction straps (E, F) were positioned to yoke the test assembly to the reaction beam (G). A heavy plate assembly was provided to concentrate the compression force over a relatively shallow area. This insured an accurate determination of the location of the compression centroid.

The instrumentation utilized was relatively simple. Dial gages were used at H, I, and J to measure the deflection at the reaction straps and directly under the applied load. An optical micrometer mounted on the theodolite shown at K in Figure 9 was used to measure the relative slip between the anchor bolt and the concrete face. Targets were mounted above and below the bolt on steel rods embedded in the concrete. A third target was mounted on the bolt next to the face of the concrete (L). The optical micrometer was used to read the relative displacement between the targets with a sensitivity of 0.001 in.

The loads were applied in increments and held until the instrumentation was read and any cracking recorded. Loading for a particular test required approximately 1 to 2 hours. Because each shaft contained more than one anchor bolt, the loading pattern had to be varied to preserve the specimen until all bolts were tested. This was accomplished by initially loading a bolt until yielding occurred. Then the specimen was unloaded, rotated, and the next bolt loaded to yielding. This continued until the last anchor bolt in a specimen, which would be loaded until complete failure occurred. If the condition of the specimen still permitted, it would be rotated and other bolts would be retested until complete failure occurred. However, since the specimens with  $2\frac{1}{2}$ -in. and 3-in. sizes contained only 2 bolts in each footing and since the overall specimen was not

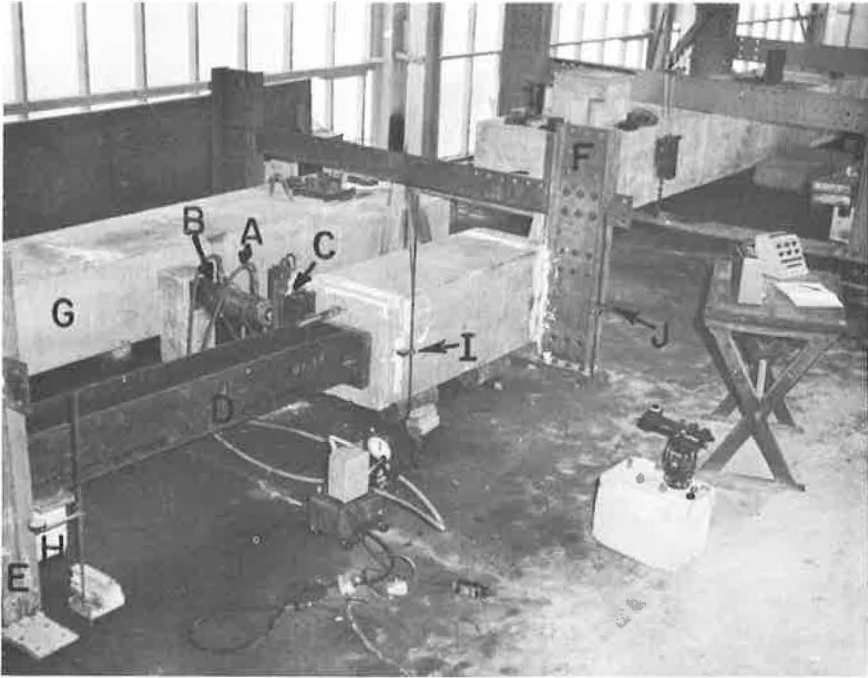


Figure 8. Loading arrangement.

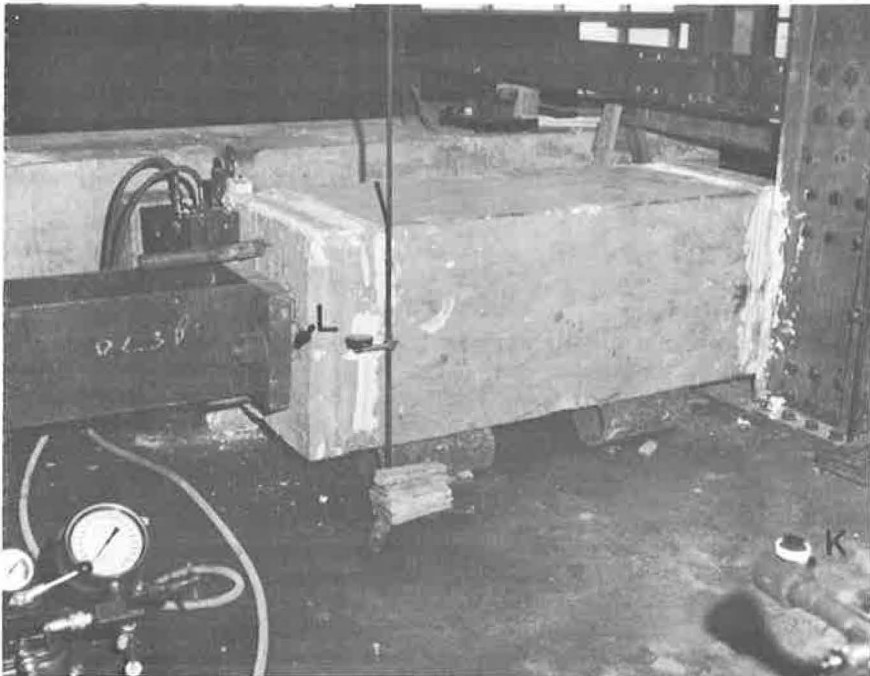


Figure 9. Slip measurement system.

too badly damaged after failure of one bolt occurred, these were taken to failure without retesting.

### SPECIMEN BEHAVIOR AND FAILURE

Loaded-end slip started almost immediately upon initial loading and progressed with gradually increasing slip increments for equal increases in load. 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 tension face. Usually the largest flexural crack occurred in the vicinity of the anchorage of the bolt. The final failure of those specimens which were taken to ultimate fell into 3 different classes:



Figure 10. Splitting failure.

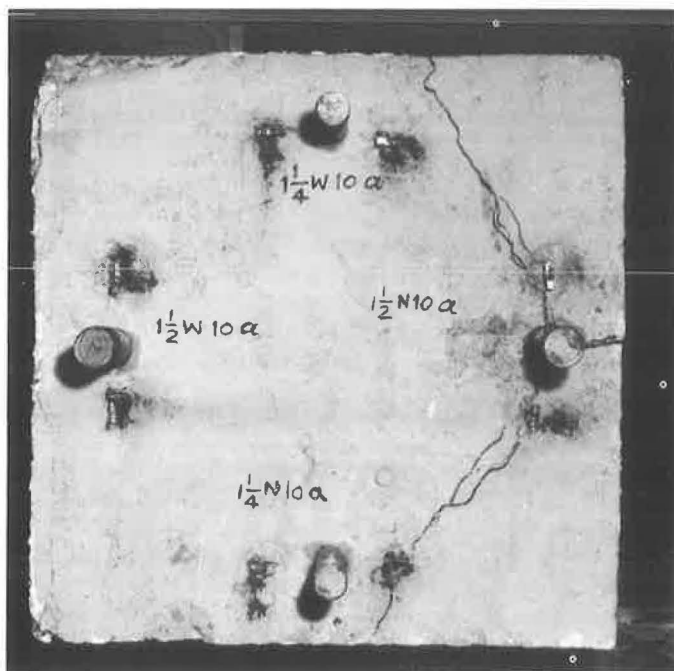


Figure 11. Splitting failure—end view.



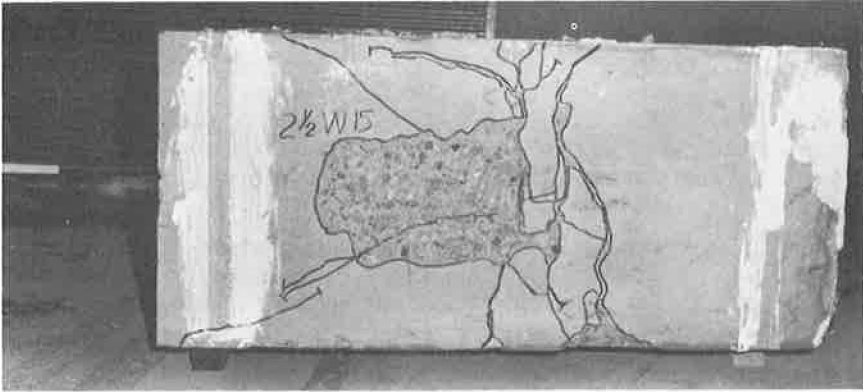


Figure 12. Crushing and spalling failure.

1. Specimens in which very definite and pronounced splitting appeared along the full length of the bolt at ultimate load, as shown in Figures 10 and 11. A very wide splitting crack extends from the bolt to the face of the specimen. The concrete cover over the bolt tends to break off into 2 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 spalling and crushing did not become apparent until failure occurred. A typical case is shown in Figure 12.

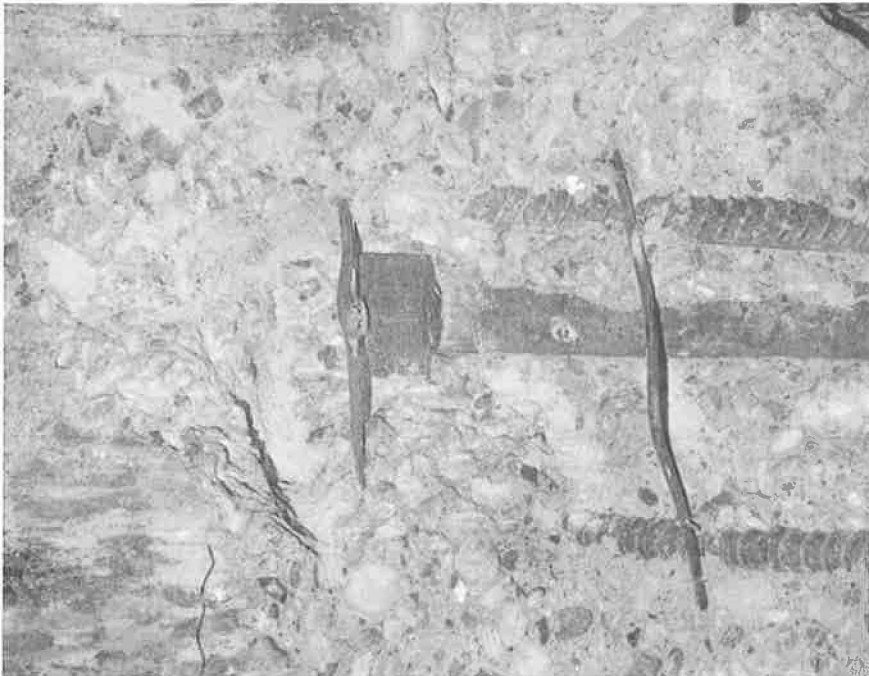


Figure 13. Fracture of bolt inside concrete.

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: (a) in 2 specimens the bolt failed inside the concrete in the root of the thread immediately in front of the anchor nut (Fig. 13); and (b) in 2 other specimens the bolt failed outside of the concrete at the root of the thread where the loading nut was attached. Although these 4 cases were the only ones in which the full ultimate tensile strength of the bolts was developed, the nominal yield points of the bolts were developed in all but 2 of the specimens tested.

In addition to these 3 general cases there was one other specialized type of failure. One anchor bolt was used which had no anchorage (i. e. , 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 was known. The calculated steel stress,  $f_s$ , can be found approximately as  $f_s = M/(A_s j d)$ . It was decided that the values for calculated steel stress should be computed for the mean thread area as recommended by ASTM standards (4). Tests have shown that the strength computed on the area at the root of the thread gives a fictitiously high value, whereas strength computed on the pitch area gives a fictitiously low value. Slaughter (5) has observed that the mean diameter is closer to the diameter of an equivalent bar in tension than either of these other 2 diameters. This was confirmed by comparative tests on threaded and unthreaded samples of the bolt stock used in this series (6). The mean area can be calculated from

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

where

- $A_{sm}$  = mean stress area,
- D = the nominal diameter of the bolt, and
- n = threads per inch.

Because the compression force was concentrated on a very small line area of the compression zone, the lever arm  $jd$  was known within an accuracy of 5 percent. Thus all steel stresses could be fairly accurately calculated.

TABLE 1  
ANCHORAGE DETAILS

Bolt Diameter, in.	Nut Diameter, in. <sup>a</sup>	Washer Diameter, in.	Clear Cover, in.	Clear Cover Diameter
1 <sup>1</sup> / <sub>4</sub>	1 <sup>7</sup> / <sub>8</sub> <sup>b</sup>	3d	2 <sup>3</sup> / <sub>8</sub>	1.90
1 <sup>1</sup> / <sub>2</sub>	2 <sup>1</sup> / <sub>4</sub> <sup>b</sup>	3 <sup>1</sup> / <sub>2</sub> d	2 <sup>1</sup> / <sub>4</sub>	1.50
1 <sup>3</sup> / <sub>4</sub>	2 <sup>3</sup> / <sub>4</sub> <sup>c</sup>	4d	3 <sup>1</sup> / <sub>8</sub>	1.79
2	3 <sup>1</sup> / <sub>8</sub> <sup>c</sup>	4 <sup>1</sup> / <sub>2</sub> d	3	1.50
2 <sup>1</sup> / <sub>2</sub>	3 <sup>3</sup> / <sub>4</sub> <sup>b</sup>	5e	2 <sup>3</sup> / <sub>4</sub>	1.10
3	4 <sup>5</sup> / <sub>8</sub> <sup>c</sup>	5 <sup>1</sup> / <sub>2</sub> <sup>f</sup>	2 <sup>1</sup> / <sub>2</sub>	0.83

<sup>a</sup>Measured across flats.

<sup>b</sup>Finished hexagon nut (7).

<sup>c</sup>Heavy semifinished hexagon nut (7).

<sup>d</sup>Type B, regular series (8).

<sup>e</sup>Type A (8).

<sup>f</sup>Type B, narrow series (8).

## VARIABLES

The primary variables were bolt diameter, bolt length, and type of end anchorage. Accompanying the variation in bolt diameter was the corresponding variation in the size of the standard nuts and washers used for the end anchorages. A complete tabulation of these dimensions is given in Table 1.

In addition, 3 secondary variables were introduced owing to the nature of the test specimens used and to the construction practices. The first is the ratio of the clear cover over a bolt to the bolt diameter. Following standard practice of the Texas Highway Department, bolts in a shaft 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 given in Table 1.

The concrete strength varied somewhat during the test program. The average  $f_c'$  was 4650 psi. However, in 2 shafts 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. The basic test data were not corrected for the variation in concrete strength.

Finally, 2 of the anchor bolts tested were made from a high strength steel with no well-defined yield point.

## LOADED END SLIP

The plot of total 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.

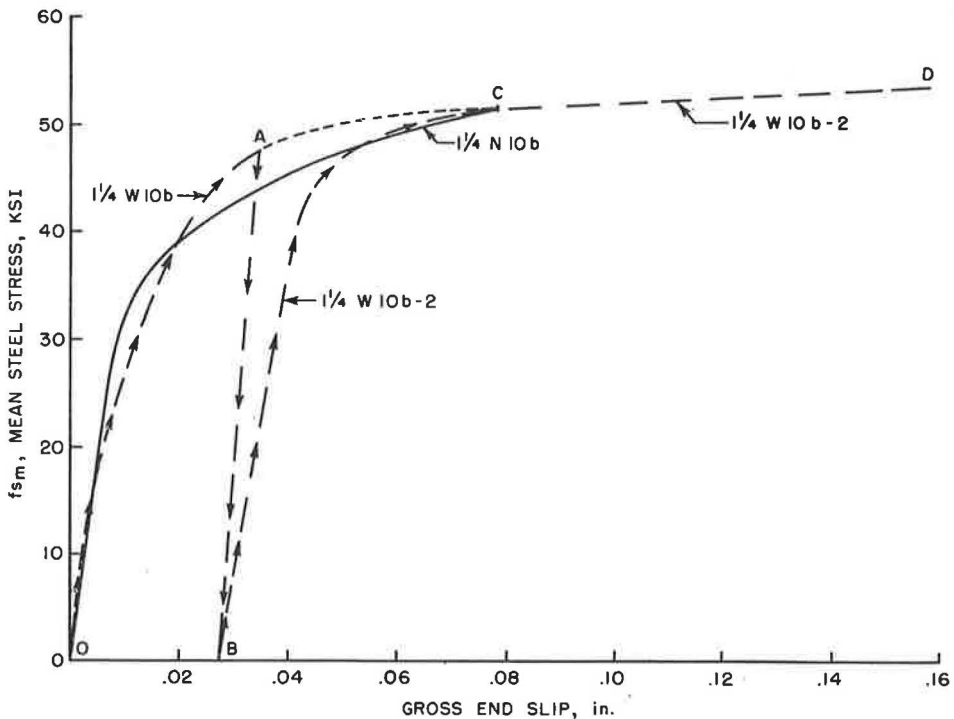


Figure 14. Effect of interrupted loading on steel stress-slip relation.

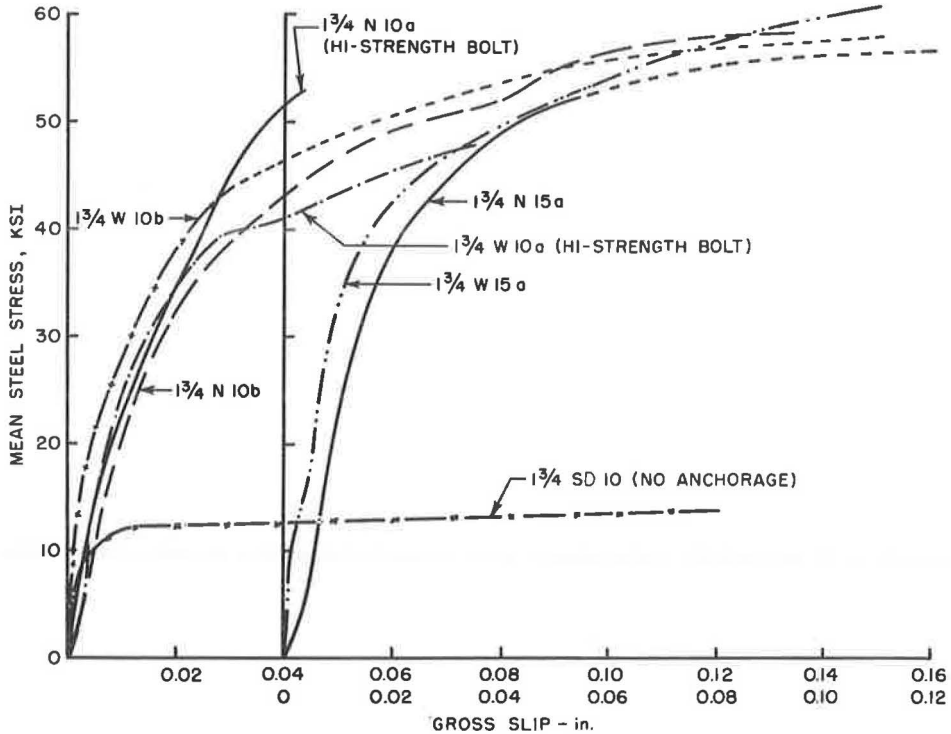


Figure 15. Steel stress-slip curves for  $1\frac{3}{4}$ -in. bolts.

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 specimen, many of the bolts were reloaded until failure occurred. From the data in these tests it was found that all of the reloaded specimens seemed to have a common behavior. The steel stress-slip curve OA in Figure 14 shows the results of the initial testing of specimen  $1\frac{3}{4}$  W 10b. Testing was discontinued at A and the load released. The total permanent slip OB upon removal of the load was observed as indicated by the descending branch AB. After testing the other bolts, bolt  $1\frac{3}{4}$  W 10b-2 was reloaded. In this test the steel stress-slip curve BCD was observed. The results of a generally similar test specimen in which the bolt was initially tested to a much higher slip level is shown by curve OC. 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. Such a construction was used to eliminate the unloading and reloading branches of the steel stress-slip curves, since this construction always took place substantially above the nominal yield point value. This hypothesized curve serves as a reasonable indication of the measure of ductility observed.

Steel stress-slip curves for all the specimens tested are given elsewhere (6). A typical pattern is indicated in Figure 15 where the results of the  $1\frac{3}{4}$ -in. diameter bolt tests are shown. It is especially interesting to note the performance of specimen  $1\frac{3}{4}$  SD 10 in which no end anchorage at all was provided. The bolt barely developed 13 ksi in bond through friction and adherence before pulling out. The inclusion of end anchorage completely changes the load capacity, although at the expense of much increased slips.

### END SLIP AT CRITICAL STEEL STRESSES

To illustrate service load behavior, the measured end slip corresponding to a steel stress of 20 ksi is shown in Figure 16 as a function of the bolt diameter. The 10 D bolts exhibit slightly greater slip than the 15 D lengths. However, in several cases the longer bolts actually show a greater slip. Except with the 3-in. bolts, the combination nut and washer anchorage (W) shows somewhat less slip than the plain nut anchorage (N). The reverse is true for the 3-in. bolts and is probably due to the reduced cover over the washers. Based on crack width limitations used with deformed bars, a good justification may be given for selecting the limiting service load value of slip as 0.02 in. for this type application. All of the bolts would meet this criterion.

Figure 17 shows the observed slip at the 33 ksi nominal yield point of the bolts. All specimens except 3 of the 3-in. bolts exhibit less than 0.03-in. end slip. In the case of the 3-in. 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 in. 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.

To provide a qualitative indication of the relation between slip and elastic elongation, 2 additional curves are given in Figures 16 and 17. Assuming that the anchor bolt is completely unbonded, the theoretical elastic elongation is shown for both embedment lengths. In Figure 16, some of the 10 D bolts indicate an end movement fairly close to that which might be expected if no bond were present, while the 15 D bolts indicate substantially lower movements. In Figure 17, the 10 D bolts generally show 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 $\frac{3}{4}$ -in. bolt with no end

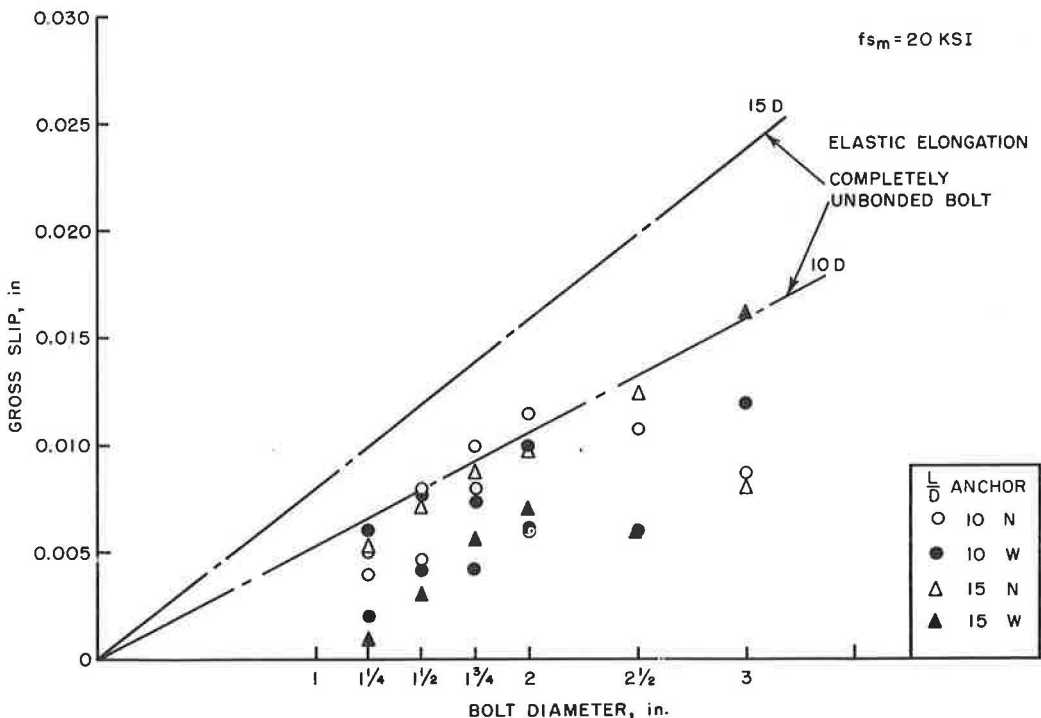


Figure 16. Slip at a steel stress of 20 ksi.



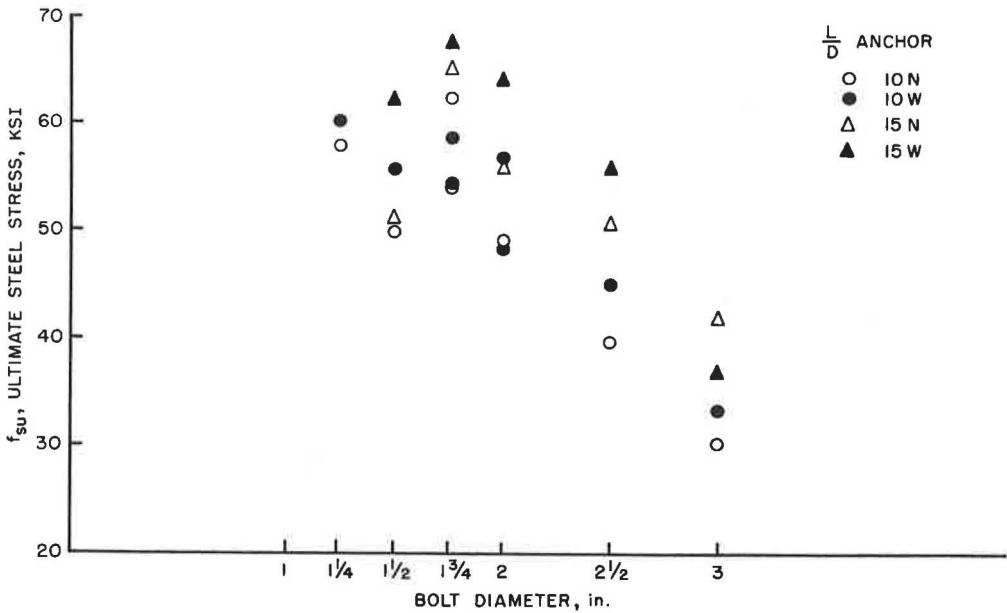


Figure 18. Ultimate steel stress.

Figure 18 indicates that 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. With low cover-to-diameter ratios, failure usually 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

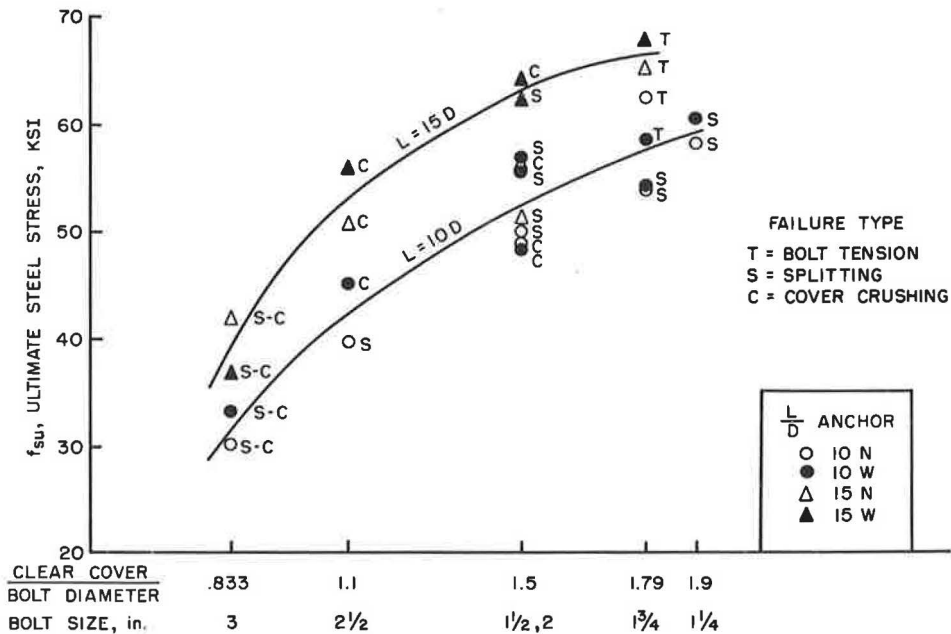


Figure 19. Ultimate steel stress as a function of clear cover.

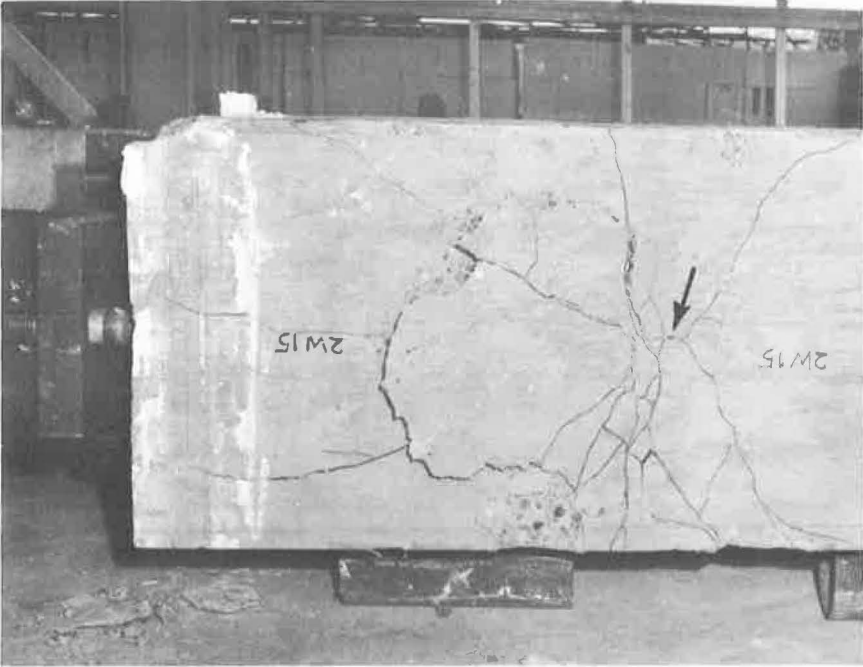


Figure 20. Spalling over anchor.

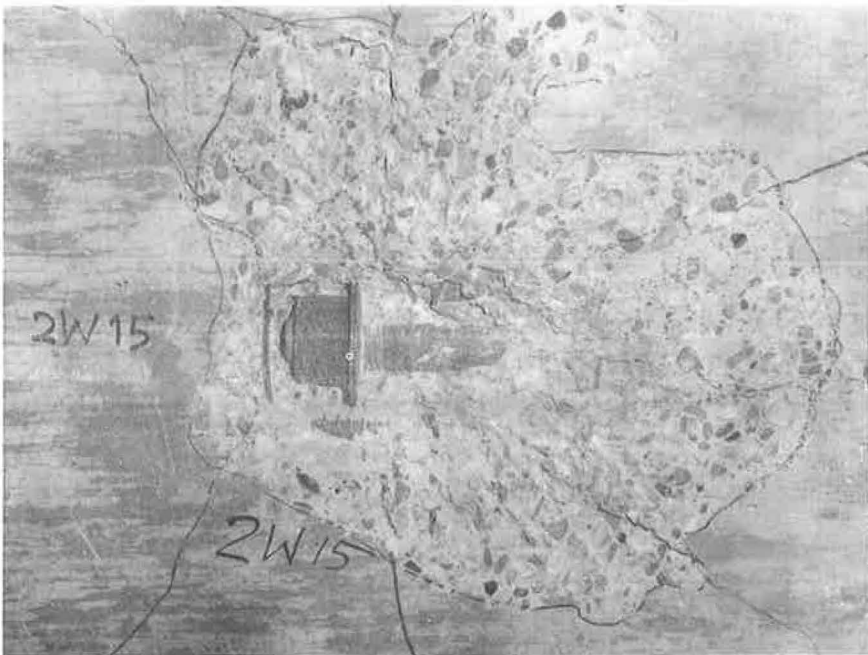


Figure 21. Anchor after removal of cover.



provided, the concrete above the anchor will crush prior to splitting of the cover or development of the tension capacity of the bolt.

### BEARING STRESSES

In Abrams' study of the concentric pullout specimen, he had reported bearing stresses of as much as 13,000 psi in the specimens with nut anchorages. With  $f_c'$  of about 1850 psi, the calculated bearing stresses would be as high as  $7.0f_c'$ . This is a case of a concrete element under a great deal of confinement and it should be possible to develop very high compressive stresses. Still, such magnitudes of stress are surprising.

In order to check the general magnitude of the ultimate bearing stresses developed by the anchorage devices, a simplified set of computations was made. Since the ultimate bond characteristics of the  $1\frac{3}{4}$ -in. 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. With nut anchorages, the computed bearing stresses range from 15,000 to 32,000 psi. Since average  $f_c'$  is about 4650 psi, this ranges from 3 to  $6f_c'$  and is in reasonable agreement with Abrams' tests. 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. For the specimens with nut and washer anchorages, the bearing stresses developed are approximately 2 to  $3f_c'$  if the full area of the washer is used. However, since so many of the specimens displayed

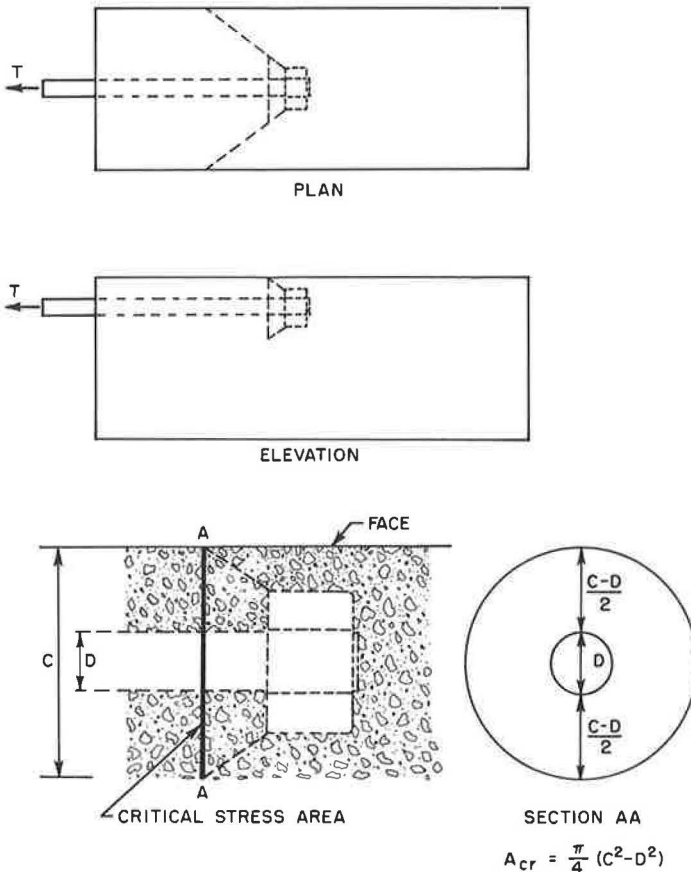


Figure 22. Critical bearing area hypothesis.

only a slight increase in ultimate steel stress when the 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 showed that the washers were generally bent back over the nuts.

EFFECT OF CLEAR COVER

In the cases in which failures occurred prematurely (i. e., before the 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 as seen in Figures 20 and 21. The location of the nut and washer is indicated by the arrow in Figure 20. 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 of stress distribution might be taking place in 3 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 the cone of stresses intersects the surface of the specimen as illustrated in Figure 22. This critical stress area can be computed as

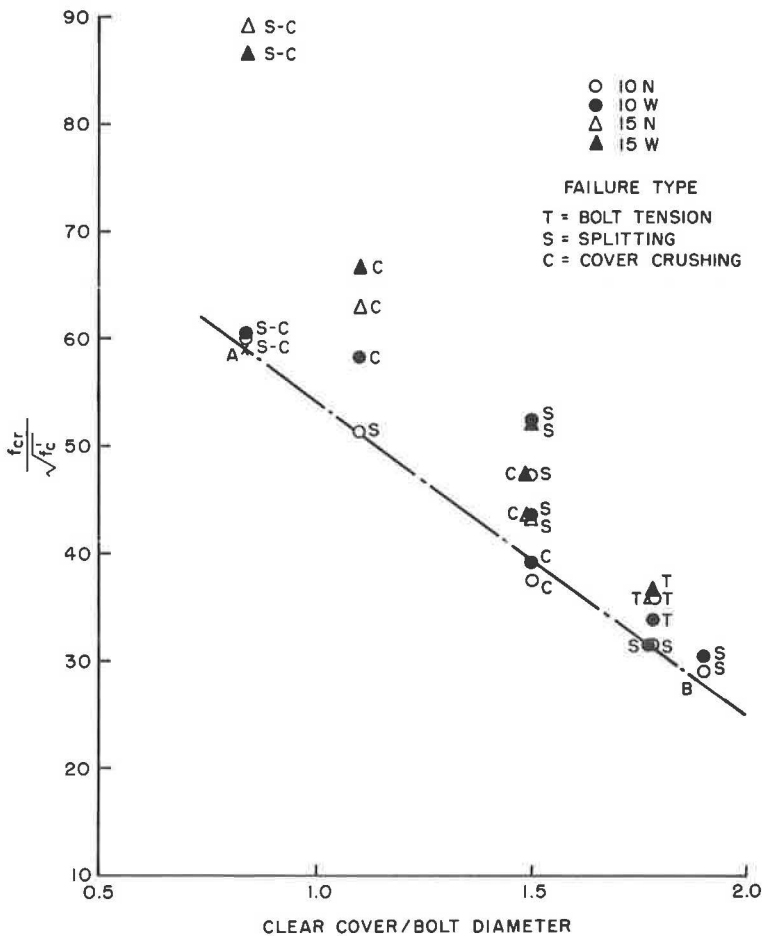


Figure 23. Critical bearing stress related to concrete strength.

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

where  $C/2$  is the total cover measured from the center line of the bolt and  $D$  is the diameter of the bolt. The fictitious ultimate bearing stresses on this section,  $f_{cr}$ , have been computed and are given in the Appendix.

In order to recognize the variation of the concrete strength in the specimens, the data were corrected for variation in cylinder strength. Since many of the failure phenomena involved splitting and since this is affected to a great extent by the tensile strength, the square root of  $f_c'$  was used. In Figure 23 the data for all specimens tested to ultimate are shown as a function of the ratio  $f_{cr}/\sqrt{f_c'}$  and the parameter clear cover/bolt diameter,  $\alpha$ .

The relation shown can be used 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. This line can be expressed by the equation

$$\frac{f_{cr}}{\sqrt{f_c'}} = 80 - 28\alpha \quad (3)$$

The limits and the overall shape of the curve were also investigated. As  $C$  approaches  $D$ , the clear cover approaches zero. For a bolt with a finite value of tensile force,  $f_{cr}/\sqrt{f_c'}$  must approach infinity. Similarly, as  $C$  becomes much greater than  $D$ , then  $A_{cr}$  approaches infinity, and the stress must approach zero. Hence the overall form of the relationship should be a hyperbola, as shown in Figure 24. The straight line portion AB may only be valid for the very small range of variables in the present series. However, a few other points are available. In a pilot series of tests run prior to this investigation, a specimen was tested with a  $1\frac{3}{4}$ -in. anchor bolt, approximately  $10D$  embedment, a standard nut and washer anchor, and increased clear cover. This bolt failed in tension and the results are plotted on Figure 24. In addition, in Abrams' original set of pullout specimens he tested some  $\frac{3}{4}$ -in. bars with a nut and washer anchorage embedded for 8 in. in an 8-in. diameter concrete specimen. For these

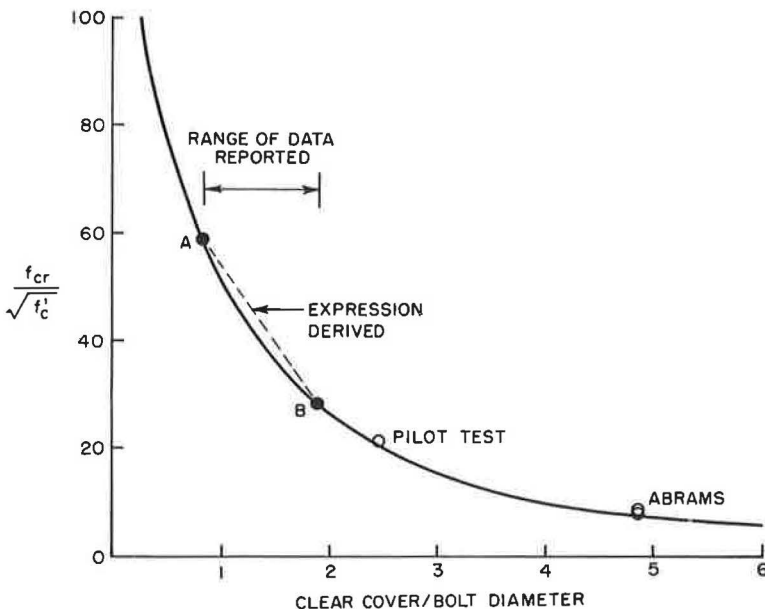


Figure 24. Probable general form of critical bearing stress-cover relationship.

specimens  $\alpha$  is 4.84. The specimens failed by splitting of the concrete. These results are also shown and tend to indicate the general trend. However, it must be emphasized that any conclusions as to the desirable amount of cover deduced from these results are only approximate and should be considered within the limits of test data available. 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 to validate such changes is presently underway.

#### PILOT TESTS—BENT BOLTS

In order to get an idea of the effect of bending the bolt to get more cover over the anchorage zone, a limited series of tests was run. A single specimen was used (Fig. 4). Two of the 4 bolts were bent, a third had no anchor and the other was a standard bolt. The specimen with no end anchorage (SD) simply slipped out at a very low stress level. The standard specimen (SA) behaved the same as similar specimens. The bent specimens (SB and SC) showed a slightly stiffer load-slip response but 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 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

The bolts used for specimens  $1\frac{3}{4}$  N 10a and  $1\frac{3}{4}$  W 10a had no well-defined yield point, as shown in Figure 6. Their yield strength, based on 0.002 offset, was 91 ksi. As can be seen in Figure 15, the specimens were loaded until end slips greater than 0.02 in. developed. Shortly after this the specimens failed by splitting. Both bolts developed an ultimate steel stress of 54.1 ksi. Their companion specimens with A 7 bolts failed in the thread area at the face of the anchor nut at 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 A 7 bolts owing to failure of the concrete, it might initially appear that there is no advantage in their use. However, design for A 7 bolts would be based on a minimum yield point of 33 ksi, which is less than 60 percent 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.

#### CONCLUSIONS AND RECOMMENDATIONS

The test series included anchor bolts ranging from  $1\frac{1}{4}$  in. to 3 in. 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 No. 2 intermediate grade spiral at 6-in. 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 reached:

1. The pullout tests indicated that A 7 (33 ksi yield point) steel anchor bolts can be fully developed with a 15-diameter embedment length and a standard nut anchor in all bolt sizes. (Steel stress computations are based on the mean diameter stress area.)
2. Anchor bolts of A 7 (33 ksi yield point) steel with diameters of  $2\frac{1}{2}$  in. or less can be fully developed with a 10-diameter embedment length and a standard nut anchor.
3. An important variable affecting the developable tensile strength is the amount of clear cover. Results of calculations based on a limited empirical expression indicate that the clear cover should be increased for the  $2\frac{1}{2}$ -in. and 3-in. diameter bolts over the  $2\frac{3}{4}$ -in. and  $2\frac{1}{2}$ -in. used in these tests.

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

1. The clear cover over the large bolts (2 $\frac{1}{2}$ -in. and 3-in. diameter) should be increased if possible. The clear cover should be at least as great as that used with the smaller (1 $\frac{3}{4}$ -in. and 2-in. diameter) bolts. An indication of the cover required can be found using Eq. 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 are 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 corresponding 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.

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## Appendix

### SUMMARY OF DATA

Specimen	$f_c'$ (ksi)	$f_y$ (ksi)	$f_s$ at		Slip at		$f_{su}$ (ksi)	Failure <sup>a</sup>	$f_{cr}$ (psi)	$\frac{f_{cr}}{\sqrt{f_c'}}$	Clear Cover Diameter
			Slip = 0.01 (ksi)	Slip = 0.02 (ksi)	$f_s = 20$ (in.)	$f_s = 33$ (in.)					
1 $\frac{1}{4}$ N10a	3.16	33.7	30	40	0.005	0.012	43.5	D	1560	27.9	1.9
1 $\frac{1}{4}$ N10b	5.04	33.7	32	39	0.004	0.011	58.0	S	2085	29.3	1.9
1 $\frac{1}{4}$ W10a	3.16	33.7	37	—	0.002	0.007	41.2	D	1480	26.5	1.9
1 $\frac{1}{4}$ W10b	5.04	33.7	27	39	0.006	0.014	47.3	D	1700	24.0	1.9
1 $\frac{1}{4}$ W10b-2	5.04	33.7	—	—	—	—	60.3	S	2170	30.6	1.9
1 $\frac{1}{4}$ N15a	4.04	33.7	33	40	0.005	0.010	41.9	D	1520	24.0	1.9
1 $\frac{1}{4}$ W15a	4.04	33.7	37	42	0.002	0.007	42.7	D	1540	24.5	1.9
1 $\frac{1}{2}$ N10a	3.16	37.0	26	40	0.008	0.013	50.0	S	2640	47.2	1.5
1 $\frac{1}{2}$ N10b	5.04	37.0	33	40	0.005	0.010	41.1	D	2170	30.6	1.5
1 $\frac{1}{2}$ W10a	3.16	37.0	32	41	0.004	0.010	41.6	D	2200	39.2	1.5
1 $\frac{1}{2}$ W10a-2	3.16	37.0	—	—	—	—	55.8	S	2950	52.8	1.5
1 $\frac{1}{2}$ W10b	5.04	37.0	23	39	0.008	0.014	38.9	D	2060	29.0	1.5
1 $\frac{1}{2}$ N15a	4.04	37.0	25	37	0.007	0.016	41.1	D	2170	34.5	1.5
1 $\frac{1}{2}$ N15a-2	4.04	37.0	—	—	—	—	51.1	S	2700	43.2	1.5
1 $\frac{1}{2}$ W15a	4.04	37.0	33	41	0.003	0.010	62.2	S	3280	52.2	1.5
1 $\frac{1}{2}$ N10a	4.66	91.0	22	34	0.008	0.019	54.1	S	2140	31.6	1.785
1 $\frac{1}{2}$ N10b	4.74	38.0	20	32	0.010	0.021	62.7**	T	2480	36.0	1.785
1 $\frac{1}{4}$ W10a	4.66	91.0	24	34	0.008	0.018	54.1	S	2140	31.5	1.785
1 $\frac{1}{4}$ W10b	4.74	38.0	28	38	0.004	0.014	43.8	D	1740	25.3	1.785
1 $\frac{1}{4}$ W10b-2	4.74	38.0	—	—	—	—	58.9**	T	2340	34.0	1.785
1 $\frac{1}{2}$ N15a	5.30	40.5	24	38	0.008	0.015	51.6	D	2050	28.2	1.785
1 $\frac{1}{2}$ N15a-2	5.30	40.5	—	—	—	—	65.3**	T	2590	35.6	1.785
1 $\frac{1}{2}$ W15a	5.30	40.5	33	42	0.006	0.010	67.9**	T	2690	36.7	1.785
2N10a	4.66	36.7	18	32	0.011	0.021	42.5	D	2260	33.2	1.5
2N10b	4.74	30.6	24	34	0.006	0.018	38.5	D	2045	29.7	1.5
2N10b-2	4.74	30.6	—	—	—	—	49.0	C	2600	37.8	1.5
2W10a	4.66	36.7	20	34	0.010	0.018	50.3	D	2670	39.3	1.5
2W10a-2	4.66	36.7	—	—	—	—	48.3	C	2570	37.8	1.5
2W10b	4.74	30.6	24	31	0.006	0.024	38.5	D	2045	29.8	1.5
2W10b-2	4.74	30.6	—	—	—	—	56.8	S	3020	43.8	1.5
2N15a	5.17	33.5	21	29	0.009	0.027	41.8	D	2220	43.2	1.5
2N15a-2	5.17	33.5	—	—	—	—	56.2	C	2990	41.5	1.5
2W15a	5.17	33.5	28	37	0.007	0.016	45.7	D	2430	33.6	1.5
2W15a-2	5.17	33.5	—	—	—	—	64.0	C	3400	47.3	1.5
2 $\frac{1}{2}$ N10a	4.63	30.7	19	34	0.011	0.019	39.7	S	3510	51.5	1.1
2 $\frac{1}{2}$ W10a	4.68	30.7	25	30	0.006	0.024	45.0	C	3980	58.4	1.1
2 $\frac{1}{2}$ N15a	5.06	30.7	16	30	0.011	0.024	30.7	C	4470	63.0	1.1
2 $\frac{1}{2}$ W15a	5.46	30.7	24	32	0.006	0.021	55.9	C	4940	66.8	1.1
3N10a	4.88	45.0	21	26	0.008	—	30.4	S-C	4200	60.1	0.833
3W10a	4.75	45.0	19	24	0.012	—	33.3	S-C	4600	60.8	0.833
3N15a	4.18	45.0	23	31	0.008	0.023	42.0	S-C	5800	89.2	0.833
3W15a	3.55	45.0	15	23	0.016	0.046	36.9	S-C	5100	86.5	0.833
1 $\frac{3}{8}$ SD10	4.90	35.5	12	12	—	—	16.1	SL	640	9.1	1.785
1 $\frac{3}{8}$ SA10	4.44	35.5	16	32	0.012	0.022	36.5	D	1450	21.8	1.785
1 $\frac{3}{8}$ SA10-2	4.90	35.5	—	—	—	—	43.3	S	1720	24.6	1.785
1 $\frac{3}{8}$ SB10	4.44	35.5	29	—	0.007	0.012	34.0	S	1350	20.2	1.785
1 $\frac{3}{8}$ SC10	4.44	35.5	21	41	0.010	0.015	44.1	S	1750	26.2	1.785

<sup>a</sup>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.

## Discussion

W. C. ANDERSON, Chief Research & Development Engineer, the Union Metal Manufacturing Co., Canton, Ohio—The subject covered in Prof. Breen's paper is very timely and gives data regarding means for obtaining more economical anchorages. Particularly in the Sign Support program the general specifications are those of AASHO, and I don't believe there is any reference to allowable stress in bonding or bearing in concrete of anchor bolts. American Concrete Institute Building Code (June 1963) gives some bond stresses and bearing values which are not really applicable but can be used as guides.

The study is particularly pertinent to shallow steel reinforced foundations and low strength bolts (A 7). It further establishes that the real bearing value of concrete in compression is many times the generally accepted allowable compression stress, if the concrete is sufficiently contained. The data support "clear cover" distances which are rational to diagonal shear stress analyses. The study also shows that washers as bearing plates on top of the nuts must be adequate in thickness to support the loads imposed or they have little value.

Other related information needed includes further investigation of shear bond stresses with higher strength steels with specifically noted surface conditions. The value of nuts embedded in deep foundations should be studied.

An examination of different styles of foundations helps show where the greatest interest should lie. Spread or slab type footings are classic with bridge engineers. These are ideal to support vertical loads. Here short anchor bolts have a real advantage. Cylindrical foundations will generally cost one-tenth or less as much as a slab foundation to support a vertical cantilever. Here the anchor bolts have been long, nearly to the bottom of the cylinders and have eliminated the need for any other reinforcing. In this case the greatest economy points to a yield of at least 55,000 psi, which is now readily available at no premium.

In the steel pole industry there have been many thousands of installations of deep cylindrical foundations over the last 40 years with loads up to 1,000,000 ft-lb that we know of, and I don't know of a single structural failure.

Getting back to the main subject, I would like to see some adoption of engineering specifications which would permit the designer to use the most economical anchor bolts in the most economical overall design. The anchoring value of a nut on the bottom of a rod might make a real contribution.

JOHN E. BREEN, Closure—Mr. Anderson's discussion is timely in pointing out the need which exists for extending the study to include the effects of higher strength steel bolts and deep cylindrical foundations. In an extension of the project reported on in this paper, both variables are under investigation as well as a study of the effect of repeated loads at service load levels.