To the Graduate Council:

I am submitting herewith a thesis written by Rex D. Rowell entitled "An Analysis of the Effects of Concrete Properties on the Loaded Behavior of Individual Wedge Bolt Expansion Anchors." I recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

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AN ANALYSIS OF THE EFFECTS OF CONCRETE PROPERTIES

ON THE LOADED BEHAVIOR OF INDIVIDUAL

WEDGE BOLT EXPANSION ANCHORS

A Thesis

Presented for the

Master of Science

Degree

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Rex D. Rowell

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ABSTRACT

The objective of this study was to evaluate the effects that certain properties of hardened concrete have on wedge bolt expansion anchors. Previous testing indicated that air content of hardened concrete could have a significant effect on these types of anchors. Thus, a program was set up which permitted an evaluation of anchor behavior with varying concrete air content, compressive strength and age. Additionally, tests to further define the loss of preload force in a wedge anchor were conducted.

The tests were performed on one brand of one-half inch wedge bolt expansion anchor. The same diameter of carbide-tipped drill bit was used for each anchor. Concrete slabs were cast with four different air contents ranging from essentially 0 to 10%. Anchors were set in groups of six and tested at predetermined compressive strengths and ages. The amount of slip each anchor underwent as well as its strength properties were analyzed. Three different methods were utilized to monitor the level of preload force remaining in an anchor prior to final loading. Statistical methods were used to evaluate the results.

The test results indicated that both during and after application of torque the amount of wedge anchor slip is affected by concrete air content. With increasing air content, above roughly 7%, the performance of an anchor begins to suffer. Variability also tends to increase for air content above this level. Depending on the air content, a wedge anchor lost roughly half of its applied preload force in the first few minutes after application. The results also indicated that further loss may occur with time.

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CHAPTER I

INTRODUCTION

A. BACKGROUND

The Tennessee Valley Authority (TVA) uses a variety of systems to attach various fixtures and equipment to hardened concrete. The two primary types are grouted anchors and expansion anchors. Although grouted anchors are acceptable and have a place in construction, they are labor intensive and require care and a certain level of craftmanship to properly install. Additionally, a curing period of at least 24 hours is normally required before being placed in service. Expansion anchors can be installed when and where they are required and placed in service immediately. Generally speaking, the majority of these latter types of anchors can be subdivided into two basic categories: expansion shell and wedge bolt anchors. Although both types are commonly used, wedge anchors have been gaining wider usage within TVA and throughout the nuclear industry in the last five to six years. One of the main advantages of a wedge bolt anchor as compared with other expansion anchors is that a desired level of preload can be placed in the anchor prior to the application of live loads. This enables a check on anchor strength before its use.

A wedge bolt anchor is defined by TVA (1) as: An expansion anchor which consists of an externally threaded bolt with a split ring or separate wedge pairs that expand laterally against the sides of a predrilled hole when the bolt is torqued, and which will expand further if the bolt is partially extracted from the hole by a tensile load. Wedge design and method of attachment to the stud vary somewhat between brands. Figure 1, Appendix A, shows some of the more common wedge designs and their method of attachment to the stud. The wedge anchor is installed by drilling a hole in the concrete with a carbide-tipped drill bit of proper diameter and a rotary hammer drill. Once a hole of minimum depth has been drilled, the anchor is placed in the hole and manually hammered to the proper embedment depth.

Since their introduction a number of factors have been identified that can critically effect the performance of wedge anchors:

1. Properties of the concrete in which the anchors are set.
2. Diameter of the hole drilled for the anchor.
3. Properties of the wedge and bolt themselves.
4. Method used to attach wedges to the stud.
5. Depth to which the anchor is set in the concrete.

Research has been conducted by TVA and other organizations towards investigating some of the factors listed above. However, routine anchor qualification testing at one of the TVA nuclear projects brought out a problem that had not been adequately addressed. Anchors were failing to meet the minimum ultimate strength requirements as set forth by TVA based on previous research. Although the test slab in which the anchors were set had what was felt to be adequate strength, further investigation revealed that the concrete had an air content of about 9%. Subsequent to these tests another TVA research program was conducted to evaluate the effect of drill bit diameter on the load capacity of wedge bolt expansion anchors. In the final report (2) it was felt that the high variability and low anchor strengths may have been caused by a higher than normal air content (7.5%). Normally TVA specifies an average air content of 5.5% ± 1.5%. Thus, not only was a need felt to investigate the effect of air content on the capacity of wedge anchors, but also to further evaluate the amount of variation that can be expected.

B. LITERATURE REVIEW

A study conducted by Adams (3) with the Bureau of Reclamation in 1954 evaluated some of the factors which influenced multiple expansion unit bolt anchors. Pullout tests were conducted to determine the difference in strength of various unit bolt anchors for different degrees of cleanliness and surface texture of the hole as well as depth of hole.

Three concrete test slabs were fabricated from the same materials with varying water-cement ratios. The anchors and methods of installation were significantly different from those under study in this program. Thus, although certain information concerning procedures and general theory were helpful, overall the tests were of minimal value.

Chabowski and Bryden-Smith (4) completed a more recent study in 1978 of in situ strength determination of concrete from expanding sleeve bolt pull-out tests. This report described the correlations found between compressive strength of prestressed beams and the torque or force required to produce the first cracking or ultimate load.

A load cell, strain indicator and torque meter were the primary tools in these tests. Although a load cell was used initially in the program to investigate the variability of results and lack of perpendicularity of bolts, the authors subsequently omitted the load cell and utilized a torque meter. Six tests at each strength level were conducted. One common drill bit size was utilized.

The results were analyzed statistically and the distribution of anchor pull-out values and cube crushing strengths was found to have been a log-normal distribution. The authors felt that size and straightness of hole should be checked frequently to ensure a good test, and enough washers used under the nut to prevent bottoming of the threads and erroneous readings. The bolt and nut threads were both greased for this study. Rate of testing or loading and straightness of hole were felt to be essential to greater reproducibility.

A report by Cones (2), with TVA, on the effects of drill bit diameter on load capacity of wedge anchors was completed in 1979. The tests were to determine if the limitations on maximum drill bit diameter specified in TVA were appropriate. Tests were performed on one brand of one-half and three-fourths inch anchors, and on two other brands of three-fourth inch diameter. Six different diameter drill bits were used for each anchor size and brand. One class of concrete was utilized for all anchors.

The results indicated that an inversely proportional relationship exists between wedge anchor capacity and hole diameter. The results also indicated that air content of concrete may have a significant effect on wedge anchor capacity. It was speculated that high air content concrete would not confine the wedges as well as equal strength concrete of lower air content.

The design of the wedging device was found to be a significant factor of ultimate capacity; thus certain brands are more likely to meet required loads than others. It was recommended, based on these tests, that new anchor drill bits conform to ANSI B94.12 (5). Drill bits used for qualification tests on wedge anchors should be near the midrange of ANSI tolerances. Air contents of concrete used to qualify wedge bolts should be kept near the mean for TVA concrete.

An anchorage study was conducted by TVA in 1976 to determine the effects of preload torque on anchor lift-off load deflection. Three different diameter of anchors were set at various depths in the concrete. Then each size of anchor and embedment depth were tested at three different torque levels. A loading rig was attached to the anchors and as load was applied, the deflection was monitored with dial micrometers.

Each anchor was loaded to lift-off and then the load released, a higher torque applied and the procedure repeated.

The results of these tests were used to help establish specifications within TVA covering the use of wedge bolts and led to a better understanding of the behavior of these anchors. Although these tests were not concerned with concrete properties, the test equipment and anchor lift-off procedures were closely paralleled in this study.

A report by Goel, Khatua and Longlais (6) documented the effect of initial torque on an anchor as well as load deflection characteristics. Although dealing primarily with anchored base plates, the discussions of preload were relevant to this research.

Stress relaxation tests to evaluate the feasibility of maintaining anchor preloads were conducted as a part of a study by Burdette and Tinsley (7) at The University of Tennessee in 1979. Two types of concrete were utilized. Series one concrete had a 6000 psi compressive strength at the start of testing and 1.5% air content. Series two had 3000 psi strength and 10% air content. The two series of tests were run with three-fourth inch diameter wedge bolts. Series one anchors were torqued to 150 ft-lb and series two to 200 ft-lb. Strain gauges were utilized to monitor the anchor load, similar to the method employed in this program. It was estimated that a total loss of 50 to 60% of initial stress could be assumed for an anchor. The percentage of losses were felt to have been independent of the initial stress. Higher air content concrete was shown to have decreased the effectiveness of wedge anchors.

Other sources of information utilized included manufacturers' literature (8) and field test results.

CHAPTER II

METHODS AND PROCEDURES

To obtain relevant, statistically accurate data which covered the pertinent factors, an analysis of variance approach was utilized not only to design the experiments but also subsequently to analyze the results. The two-way classification analysis of variance allows a determination of the effects of two controllable sources of variation on the desired factor.

Tests for significance were made for each factor which influenced anchor response as well as whether interaction was present. The test for significance (F test) is made to determine if the null hypothesis (H_0) for each response can be rejected. The null hypothesis states that the treatments, or main effects such as air content and compressive strength, had no effect on the responses and that there was no interaction between each set of treatments that influenced the response. Thus when F_0 (calculated from the data) was greater than F_α (level of test significance selected) the null hypothesis was rejected and the respective treatment considered to have significantly effected the response. An interaction means that the treatment effect does not remain the same over different levels of the other treatment. That is to say, the effect of one level of treatment A on the response depends upon the level of treatment B. Where interaction was not found to have been significant then the outcome of each treatment's "F" test could be interpreted as determined. If interaction was significant then the main effects which calculate as significant can still be interpreted as such. However, nonsignificant main effects cannot be interpreted as indicated.

It was determined that due to the wide range of variables it would be necessary to select or set certain arbitrary limits. The major factors and the limits set were:

1. Air content of concrete. Since the air content normally specified by TVA is 5.5 ± 1.5%, a range that would completely bracket this was desired. Four separate air contents were chosen, 10%, 7%, 5% as well as a concrete without any air entraining agent added, which ran about 1%.

2. Compressive strength of concrete at time of test. Groupings of anchors were tested at three different concrete strengths, 2000, 3000 and 4000 pounds per square inch (psi). These particular strengths were chosen to provide a reasonable spread of data within an acceptable time frame based on the basic 3000 psi mix.

3. Age of concrete at time of anchor testing. Three different ages of concrete were chosen, 7, 14, and 28 days, on which anchors were tested for each air content.

The program was first divided into two major phases. Phase I investigated the effects of concrete air content and compressive strength as they affected the behavior of wedge bolt anchors. Phase II investigated the effects of concrete air content and age on wedge bolt anchors. The loss of anchor preload force after torquing was evaluated in both phases.

The decision to laboratory batch and mix all the concrete instead of using local ready-mixed concrete was based on several factors:

1. Adequate control of ready-mixed concrete air content and compressive strength could not have been maintained to the levels desired.

2. Laboratory space limitations prohibited the placing of large slabs, thus larger volumes of concrete associated with ready-mix were not needed.

The materials chosen to batch the concrete are listed in Table 1, Appendix A. One basic class of concrete, a 300.75A mixture, was utilized throughout the program. This is a standard TVA concrete mixture that has been used in construction. As outlined in TVA General Construction Specification No. G-2, this is a concrete which attains approximately 3000 psi at 28 days of age. The maximum aggregate size is no greater than three-fourth inch. A nonfly ash concrete was utilized, although normally a pozzlan (fly ash) is used in most TVA concrete. Fly ash was omitted due to the slower rate of strength gain which would have required a longer test period, as well as the need to minimize as many factors as possible which might affect the results. A standard type I portland cement that is common to this area and readily available was chosen. A sufficient quantity was purchased at the start of the program to ensure that the same cement would be used throughout. This helped to assure relatively consistent amounts of entrained air from batch to batch, since different lots of the same brand and type of cement can entrain varying amounts of air. A further precaution was taken to ensure uniformity by emptying all the cement together in a pile on a clean concrete slab and blending them together. The resultant mixture was placed in air tight drums and stored in a warm dry location until needed.

The aggregate was a crushed limestone obtained from Vulcan Material Company's local quarry in Blount County, Tennessee. The coarse aggregate was processed over vibrating sieves and separated into the various individual sizes required for the concrete mix. Then each size was washed over a screen, dried, and placed in a sheltered storage bin. The sand, or fine aggregate, was similarly sieved to the various required sizes and placed in storage bins. When a concrete mixture was to be batched the sand and coarse aggregate were weighed according to the gradation given on Table 1, Appendix A. These gradations were obtained from TVA specifications. The aggregate could have been used as delivered; however, close control of all materials in the concrete was desired. Also, the aggregate was somwhat dirty, and washing was considered necessary.

The air entraining agent was a standard vinsol resin manufactured by Protex Industries and is used by TVA. Water reducing agents were not used. Since spacing of the coarse aggregate in the concrete matrix was felt to be an important factor in the variability of results, the coarse aggregate volume was held constant for each mixture. As the air content was adjusted from mix to mix the water and sand were adjusted accordingly to maintain the proper mortar volume. A slump, as determined by ASTM C-143, of 3 ± 1/2 inches was held for each batch. The temperature at each time of mixing ranged from 65 to 75 degrees F.

Only one brand and size of wedge bolt anchor was utilized in these tests in an effort to minimize variations. The 1/2 inch diameter by 5-1/2 inch long anchor chosen has been used in previous research and has shown itself to be a reliable and fairly consistent anchor. They are constructed of the same steel as that in A325 bolts which has a yield point of roughly 80,000 psi. The size of the anchors chosen was a trade off between a need to keep the test slabs to a manageable size and the problem of accurately monitoring the loads in anchors smaller than 1/2 inch diameter. The anchor length was determined by the embedment depth required. A 3-1/4 inch embedment was kept standard and closely checked for each anchor. This depth was chosen since it complies with TVA requirements as set forth in G-32 (1) and was found to be adequate based on prior research. It has been found by other research that little additional strength is gained from further embedment beyond this depth, since failure does not occur due to cone pull out.

The dimensions of the eight concrete test slabs were dictated by a number of considerations. There was a desire to conduct the tests in a controlled environment and thus avoid the problems and temperature fluctuations normal to an outdoor location. The controlled atmosphere greatly facilitated the proper curing of the slabs and eliminated certain problems with delicate test equipment. Since the slabs were to be placed indoors, the size of each slab had to be kept to a minimum. However, the dimensions had to be large enough to accommodate the required number of anchors. The spacing of the anchors was based on TVA standards (1) which call for wedge bolt anchors to be located not closer to a free concrete edge than ten nominal bolt diameters, regardless of loading. The minimum spacing between 1/2 inch anchors is conservatively specified as twelve times the diameter or 6 inches. Due to space limitations, a 5-1/2 inch minimum spacing was used.

The number of anchors required per test sequence was based on several factors. ASTM E488-76, Strength of Anchors in Concrete and

Masonry Elements, calls for a minimum of five specimens. Based on statistical considerations it was decided that an even six anchors per test set would be used. Therefore, the dimensions of the slabs had to accommodate three series of six anchors each, be a minimum of 4 inches thick and be wide enough to provide a clear edge distance of over 5 inches. Based on these requirements as well as the maximum capacity of the laboratory concrete mixer, each slab was formed 36 inches square and 6 inches deep. This allowed a 5-1/2 to 6-inch clear distance between each anchor or a free edge. Figure 2, Appendix A, illustrates the configuration of the anchors on each slab. As can be seen from this figure, there is an open area in the center which was reserved for the removal of 3 x 6 inch cores for testing

The location of each anchor on the slab was established by use of a random number generator. This was done to scatter the anchors about the slab and reduce the effect of concrete strength variations. The locations generated for the eighteen anchors were maintained uniformly throughout all eight slabs unless otherwise required in certain circumstances.

The volume of concrete needed to fill one form was 4-1/2 cubic feet. Phase I required the fabrication of twenty-four 6 x 12 inch concrete test cylinders for each slab. Since the effective capacity of the laboratory mixer is approximately five cubic feet, the concrete was mixed in two consecutive identical batches. Half of the test cylinders were fabricated from each concrete batch and the remainder placed into the form. After the second batch was placed, the concrete was gently shoveled together and consolidated by internal vibration. The same person operated the vibrator on all slabs to minimize variation. Vibration was kept to the minimum necessary to achieve proper consolidation. All slabs were surface finished by floating and steel troweling. Proper finishing techniques were followed. When finishing was completed each slab was moist cured during the entire test period. The test cylinders fabricated for each slab were placed next to each companion slab and moist cured alongside the slab until tested. After mixing and prior to placement, each batch was checked for proper consistency by the ASTM C-143 slump test and air content by the pressure method, ASTM C-231.

The physical properties of the hardened concrete, as contained on Table 2, Appendix A, were determined by tests conducted on the 6 x 12 inch cylinders as well as 3 x 6 inch cores drilled from the slabs, Figure 3, Appendix A. Two cylinders each were tested in a universal testing machine, Figure 4, Appendix A, for compressive strength at various ages to establish the strength of the slab and its rate of strength gain. Two cores were drilled and removed from a slab whenever tests were performed on anchor bolts to verify the concrete strength at time of test. Since Phase I required the testing of anchors at specific strengths, a series of cylinders were broken at regular intervals and the results plotted to enable proper determination of when to commence anchor testing.

The universal testing machine was also used to evaluate the modulus of elasticity. A compressometer, Figure 5, Appendix A, attached to a cylinder and the chart recorder, Figure 6, Appendix A, continuously monitored the load-deflection characteristics during a test. From these curves the chord modulus of elasticity was calculated according to ASTM C-469.

Determination of tensile strength for each concrete was made by means of splitting tensile strength according to ASTM C-496. These determinations were made on Phase I concrete only, since it was assumed that Phase II concrete would possess similar properties. An approximation of hardened air content was calculated from the unit weight of the cores removed from each slab as presented in sample calculations in Appendix B.

The determination of load loss subsequent to torquing was evaluated by three different methods. One method, utilized in Phase I, is commonly referred to as "lift off." When a preload is applied to a wedge anchor, the tensile force in the anchor develops an equal compressive reaction between the washer and the concrete. Any external load must overcome this internal compressive force before the anchor will receive or react to an applied load.

After a preload had been applied to a wedge anchor by torquing the nut against the washer underneath, a monitored load was applied gradually to the top of the anchor until the washer under the nut could first be moved. The lifting of the nut from the washer indicated that the applied force had just balanced the existing force in the wedge anchor.

The Phase II method consisted of using a 600 inch-pound (in-lb) torque wrench to measure directly the level of preload torque remaining in an anchor. The torque was slowly applied until the nut on the anchor bolt first began to rotate. This indicated that the residual level of pretorque had been reached.

The third method of preload loss monitoring was used in both phases and permitted continuous monitoring of load loss in an anchor.

This method utilized a Strainsert Strain Transducer readout and two one-half inch diameter SJ-Q standard studs threated at each end. These studs, when connected to the strainsert transducer, measure the micro-strain that exists in each stud. The strain bolts were constructed of steel with a yield strength of 100,000 psi. Although the outside diameter of the bolts measured one-half inch, each stud had a small hole drilled through the center of the shaft parallel with the long axis for the internally mounted strain gauges. Thus the effective area used in calculations was reduced to account for this.

The strain monitoring system was calibrated by placing a stud in the universal testing machine and applying, in even increments, a known load to each stud and recording each corresponding strain reading. In this manner calibration curves were made for each stud. Square tubing of one-half inch wall thickness was used to transfer the reaction from the top of the strain bolt to the surface of the concrete, as shown in Figure 7, Appendix A. Steel plates were used as shims or washers between the top of the tubing and the stud washers and nut. A coupling nut was used to connect the strain stud to the anchor. The overall height of the system was adjusted so that the end of a stud would almost touch the top of the anchor bolt inside the coupling nut to prevent problems of insufficient threads. The strain stud system consisting of the stud, nut, washers, steel shim plates, tubing and coupling nut were assembled as a unit and torqued to the test load. The losses due to relaxation in the system with time were recorded and another series of curves generated to correct the loads.

A torque of approximately 40 foot-pounds (ft-lbs) was used for all anchors in accordance with TVA G-32 specifications (1).

The apparatus for loading the wedge anchors, shown in Figures 8 and 9, Appendix A, consisted of the following items:

1. A large capacity Enerpac PH-80 manual hydraulic pump.

2. A 0-1000 psi calibrated precision pressure gauge.

3. A sixty-ton center hole single acting hydraulic ram.

4. A 1-1/4 inch diameter threaded steel pull rod.

5. A coupling nut threaded to accept the pull rod in one end and the 1/2 inch anchor in the other.

6. A swivel head to ensure axial loading.

7. A steel tripod or stand to transfer the reactions from the ram to the concrete roughly 12 inches from the anchor center line.

On certain edge anchors space did not permit the use of the tripod, in which case an alternate loading assembly was substituted as shown in Figure 9, Appendix A. The hydraulic pump-ram system was calibrated in a universal testing machine. This was done both at the start and during the program to ensure accuracy.

The holes for the anchors were drilled with a Phillips 747 Roto-Stop Hammer Gun and a Rawl carbide tipped drill bit of 0.518 inches diameter. This diameter is at the lower end of the ANSI requirements (5). The diameter of the bit was checked periodically to determine any significant changes due to wear.

The test procedure consisted of accurately marking the embedment depth on 6 anchors. The holes were drilled at the indicated locations as nearly vertical as possible, Figure 10, Appendix A, and the dust blown from the holes afterwards with a squeeze bulb. The anchors were then driven into each hole with a large hammer to the embedment mark on the anchor. Using a shop square, the two most vertical anchors were selected for strain monitoring. The remaining four anchors were fitted with the required spacer plates and washers and the nuts torqued to 40 ft-lb. During Phase I, lift-off tests were performed on each of these four anchors within five minutes after torquing. The four anchors were then re-torqued to 40 ft-lbs and left for approximately five hours until final testing. Phase II did not involve any preliminary check of torque on these four anchors.

The remaining two anchor bolts had the strain monitoring apparatus installed. This involved centering the 1/2 inch thick square tubing over the anchor on the concrete. Next, after connecting the strain stud leads to the transducer and fitting the stud through the shim plates and washers, the coupling nut was threaded onto the end of the strain stud, the assembly lowered onto the wedge anchor and threaded into place, Figure 11, Appendix A. The preload was applied to the anchor by tightening the nut on the top of the strain stud, Figure 12, Appendix A, until the readout indicated that the predetermined tensile force of approximately 4800 pounds was reached. Subsequently, the two anchors were monitored at frequent intervals.

The required tensile force in the anchor was determined by utilizing a 40 ft-lb torque value in the relationship:

$$W = \frac{(12)\ T}{K\ D}$$

where

W = tensile load,

T = torque, ft-lb,

K = torque coefficient (K was assumed equal to 0.2 where the bearing surface of the nut was in contact with a smooth unlubricated steel washer of correct size and the threads are not damaged. The threads were not lubricated);

D = nominal bolt size (inches).

Figure 13, Appendix A, shows a typical system just before termination of monitoring.

During Phase I, each of the four wedge anchors were again tested at approximately five hours by the lift-off method. Once the lift-off load was recorded, the magnitude of anchor slip was measured relative to the concrete surface. The loading apparatus was reattached, and the anchor axially loaded to its ultimate capacity. When this point was reached, the anchor slip was again measured and the anchor then removed from the concrete.

Phase II utilized a calibrated torque wrench, as outlined earlier, to determine loss of torque. Again a determination of the amount of slip was made.

The strain apparatus was then removed from the last two anchors and the slip measured. The ultimate load and corresponding slip were determined as described earlier.

At the conclusion of all anchor testing, certain holes from which the anchors had already been removed were cored from the respective slabs. The cores were then carefully sawn open exposing the anchor hole to permit examination of the anchor hole.

CHAPTER III

RESULTS

A. SCOPE

Concrete properties and the various anchor test results are presented in this chapter. The statistical test findings determined for each of the various anchor parameters are presented and discussed along with the anchor results.

B. CONCRETE PROPERTIES

The concrete physical property results are contained in Table 2, Appendix A. The compressive strengths recorded for the 6 x 12 inch cylinders and the 3 x 6 inch cores were considered reasonably close for each test series. The only exception to this was the Phase I, 7% air content series. The average difference for the three groups was approximately 300 psi. The specific gravities for Phase I varied from 2.39 at 0% to 2.25 at 10%. Phase II varied from 2.44 at 0% to 2.31 at 10%. Generally, the specific gravities in Phase II were slightly higher than in Phase I.

The age at which tests were conducted in Phase I ranged from three days for all four air contents to eighty-eight days for the 10% slab to achieve its required strength. Phase II compressive strength did not vary with age by more than 1000 psi for any of the four air contents. The Phase II hardened air content determinations were all somewhat lower than those from the plastic concrete. This drop is normally encountered due to losses from handling and vibration. The hardened air contents ranged from 0.5 to 8.33%. The hardened air contents calculated in Appendix B were not exact. A number of factors can affect the outcome; moisture content and age of a specimen can change the apparent results. If moisture is lost by the core over a long period of time, then the water content used in the theoretical unit weight determinations would have to be changed to remain correct.

The splitting tensile strengths of the 6 x 12 inch cylinders ranged from 539 psi at thirty-five days for the 7% concrete to 257 psi for the 10% concrete at three days. Generally, the splitting tensile strengths varied according to air content. The 10% slab was the lowest and the 5 and 7% slabs the highest. The tensile strength computed by this method is typically 15% higher than that determined by direct tension tests. However, it was considered a reliable test which normally averages about 10% of the compressive strength.

The chord modulus of elasticity in Phase I ranged from 3.22 kips per square inch (ksi) for the 10% concrete to 4.80 kis for the 0% concrete. At thirty-five days the 7% slab had 4.77 ksi; thus the two were considered equivalent. Phase II chord moduli varied from 3.81 ksi for 10% to 5.48 for the 0% concrete. In Phase II the 0% concrete had the highest overall level of modulus while the 10% the lowest. This grouping also appears true of compressive strength in both phases.

C. ANCHOR PRETENSION LOSS

The tabulated results of Phases I and II anchor testing are contained in Tables 3 and 4, Appendix A. The results for the Phase I, 5% air content series at 2000 psi as well as the 0% air content for the five minute lift-off series were not obtained. A graphical comparison of the Phase I lift-off series conducted at five minutes and five hours are contained in Figure 14, Appendix A. Each data point is an average of four values. The 5% series was eliminated from this figure due to a lack of data. The curve for the 0% five minute series was drawn based on one point and assumptions from the other two groupings. All three curves indicate that the measured anchor lift-off strength at any air content was higher at five hours than the initial five minute tests. The average difference was approximately 200 to 300 pounds for all groups. The slightly higher values at five hours were probably a result of the earlier tests at five minutes, since the first lift-off tests may have increased the existing tension or preload in each anchor a small amount.

The magnitude of loss indicated was between 40 and 50% of the pretension force. The difference between lift-off values at five hours for each air content was further considered in Figure 15 and Table 5, Appendix A. An analysis of the effects of concrete compressive strength and air content on anchor load was uncertain. The findings did not indicate a close correlation for either factor. Due to the construction of the Phase I experiment, age effects could not be evaluated by analysis of variance. The Figure 15, Appendix A, curves tend to verify previous statistical findings in Table 5, as they were highly interactive and closely grouped. Figure 16, Appendix A, compared anchor strength-age ratios to concrete strength. Each ratio was determined by dividing an anchor's lift-off strength by its respective age at test in days. A ratio was utilized since it was believed that age may have been as significant as the other factors and could have masked the results. With the exception of the 10% curve, the other three curves all had roughly the same shape and slope. If the first reading for the 10% curve is ignored, then the four curves fall into a recognizable pattern. If this pattern is correct, then the amount of pretension force retained is highest in the 5% concrete, followed in order by the 7, 0, and 10% concretes.

The average coefficients of variation (COV) for the four concrete air content test series ranged from 17% COV for the 7% series to 10% COV for the 0% series. Normally it has been assumed that any COV from 10 to 15% was acceptable. With a coefficient of variation of this magnitude and four anchors per series, the 90% confidence interval width for the overall mean for each concrete air content varied from 15 to 25% of the sample mean.

In Phase II, part of the purpose was to better define torque loss with time. The torque method previously described was utilized. Based on the statistical analysis in Table 6, Appendix A, it was found that neither the air content nor age of concrete at time of test were considered significant factors in the amount of torque loss. Figure 17, Appendix A, is the graphical representation of torque loss versus concrete age. Figure 18, Appendix A, is anchor preload calculated from the torque values, as discussed previously, versus concrete compressive strengths. The results as depicted in these figures were difficult to interpret. It appears that the 10% series lost more torque or load than did any of the others. The other three air content curves were all closely grouped and all appeared to have the same level of preload.

The coefficients of variation encountered during the torque loss tests were roughly twice those of Phase I. In certain cases the COV ran as high as 58%. With this large a variation, the 90% confidence interval width for the population mean for this series was over half of the sample mean. Thus, it was difficult to separate variations due to significant factors from those due to error.

Once an original preload was placed in the strain bolt anchor system, the subsequent loads recorded at each time interval had to be corrected to account for the fact that these loads were greater than what would have existed in an anchor alone due to strain relaxation considerations. Thus, for a given magnitude of slip (ΔL_s) if ΔL_s occurs over an 11-inch length, the loss of load would be significantly less than if the same slip occurred only for the anchor. This same principle is considered in prestressing to compute the remaining load in a tendon after relaxation of the steel.

A two step procedure was followed to approximate the load as shown in the sample problem in Appendix C. First the magnitude of slip was calculated from the recorded load and strains. Then the respective slip values were considered as having acted on the anchor alone, exclusive of the strain bolt, and a new adjusted load calculated from this. The average slips ranged from 3 to 5 mils (1 mil = 0.001 in.).

Each anchor bolt was treated in the calculations as a shaft with two different cross sections. The upper section consisted of a 4-1/2 inch long by 1/2 inch diameter shaft. The lower portion, or wedge ramp, was a reduced diameter section around which the wedges were mounted flush with the upper body. The reduced section was not sloped within the area of influence considered. Thus, although overall anchor length was 5-1/2 inches, only 5 inches were considered in the calculations. The tapered portion of the wedge ramp below the wedges was not considered. The effects of the slightly reduced cross sectional areas in the threaded portions were neglected.

Phase I monitored loss of preloads are shown in Figures 19-21, Appendix A. These are plots of load versus log time, based on the average of two readings per series. All three series of curves have several properties in common. Roughly half or more of the initial preload was lost in the first twenty minutes or less. The rate of load loss, based on the slope of the curves, appears to be fairly uniform for all air contents, although the magnitudes vary. The 2000 psi strength curves, Figure 19, Appendix A, show the 0% air content concrete with the least loss and 10% the greatest. The 0 and 7% curves were closely grouped. Although a 5% curve was not available, it was assumed that this would have plotted somewhat higher than the other curves. The 3000 psi series, Figure 20, Appendix A, fell into roughly two levels: an upper grouping of the 5 and 0% curves and a lower of the 7 and 10% in respective order. At 4000 psi the order shifted dramatically. The 7% curve showed the least loss and 5% of the greatest. Comparison of the three groups of curves showed that, overall, 0% displayed less loss of preload at 3000 psi than at 2000 psi compressive strength. However, no further gains were made at 4000 psi. The 10% curve showed the greatest loss of preload. A significant gain was noted between the 3000 and 4000 psi strengths. At 4000 psi 7% showed the least loss for any compressive strength. The 5% curves were difficult to evaluate since only two were available. They showed the least loss at 3000 psi compressive strength and the greatest at 4000 psi. The cause of this radical change in the 5% series is uncertain. One possibility was a crack which developed in the slab during the 3000 psi ultimate anchor strength tests. Although the 4000 psi test anchors were located with the crack treated as a free edge, it was possible that concrete micro-cracking or other undetected cracks may have influenced the results. The data from all the 4000 psi strength tests show certain anomalies which may be due to this.

A comparison of corrected strain bolt loads and lift-off loads showed that the lift-off load determinations were somewhat greater. With increasing time the difference increased until, at roughly four hours, the lift-off method was twice that indicated by the corrected strain bolt loads. A certain percentage of this difference could be attributed to assumptions made in Appendix C calculations which resulted in corrected loads lower than would actually exist.

The results of Phase II strain monitored loss of preload are shown in Figures 22-24, Appendix A. In all but one case, over half of the initial preload was lost in less than fifteen minutes. Less than 20% of the initial preload still existed after five hours with the exception of the 5% series at twenty-eight days. This particular series showed the least amount of preload loss overall in either Phases I or II. The 5% air content curves at each age in Phase II had the least loss while 10% the greatest percent loss. The 10% curve at twenty-eight days indicated a total loss of preload after three hours. As the age of test increased, the grouping of the curves appears to open up along the Y-axis. At seven days all four curves are reasonably close together. The fourteen day 0 and 5% curves showed slightly less loss of load while maintaining the same general slope. After twenty-eight days the 5% curve increased almost 1000 pounds and 0% roughly 300 pounds, although the slope of the 0% curve changed somewhat at this point. No appreciable change was noted between the 7% curves.

The number of replications (two) in each strain test together with a large variation prevented a reliable forecasting of anchor preload with time. The only apparent similarities among all the Phase I and II curves were the slopes which were roughly equivalent. The greatest magnitude of preload loss overall was from the 10% air content concrete.

D. ULTIMATE ANCHOR STRENGTH

The ultimate anchor tensile loads contained in Table 3, Appendix A, are an average of six replications at each level. The ultimate strengths of the two anchors monitored by the strain method in each group of six were checked to ensure that they did not vary significantly from the average of the other four anchors as a result of the different method of preloading. The comparisons were made for both phases and two air contents chosen as representative of the total group. A one-way analysis of variance of the data indicated no significant differences in any of the data due to the two different methods of preloading (Tables 7-10, Appendix A). Thus all six values were used to determine mean values. The statistical analysis of the results (Table 11, Appendix A) indicated that in Phase I the concrete compressive strength and air content had a very significant effect on ultimate anchor strength. Interaction was not found to have been significant. Figure 25, Appendix A, contains Phase I anchor strength-concrete strength curves for the four concrete air contents. The 7% curve had the highest anchor strengths for any strength concrete. This was followed by the 0% series with close to the same slope or rate of anchor strength gain as 7%. The 10% curve was the lowest and evidenced a slightly different shape than the other two curves. This could indicate a leveling off of anchor strength gain with increasing compressive strength beyond a certain level.

Considering age as a factor, an anchor strength/concrete age ratio was plotted against compressive strength (Figure 26, Appendix A). This shifted the 5% curve from near the bottom to above the others. Referring back to Figure 15, Appendix A, top curves, 7% is the highest. Anchor strength gain appears to level off for the 10% series at fifteen days age. Figure 27, Appendix A, top curves, shows the relationship of Phase I ultimate anchor strength to concrete air content. These curves illustrate the effect that compressive strength has on anchor strength. The greater the compressive strength the greater the resultant anchor strength at any air content. Further, the optimum combination appears to be close to 7% and 4000 psi.

The coefficients of variation for the Phase I values were all below 14% with an overall average of 9%. This corresponds well with previous tests performed by TVA which exhibited variations of from 5 to 10%. A coefficient of variation of 9% with six anchors per set would give a 90% confidence interval width of somewhere close to 8% on either side of a typical sample mean, indicating fairly reliable results. As the concrete air contents increased, so did the average variations at each level, with the exception of the 5% group. Thus, as the air content increased, variability of results increased.

Phase II ultimate anchor strengths are contained in Table 4, Appendix A. The analysis of variance tests (Table 12, Appendix A) indicated that the percent concrete air content was a significant factor of ultimate anchor strength. Concrete age was not found to have any apparent effect. However, interaction was found to have been significant which meant that nonsignificant main effects (concrete age) could not be interpreted as the "F" test indicated. Thus, whether concrete age was a significant factor is unsure based on the data. An error analysis of these data determined that almost 80% of the variation in the age results were due to random error and unassignable causes.

The coefficients of variation for anchor strength were lower on the average in Phase II than Phase I. The overall sample was 8% versus the 9% in Phase I. There was no pattern to the averages as in Phase II; however, the highest variability of the series was again the 10% group as in Phase I.

The plot of air content versus anchor strength (Figure 27, Appendix A), lower curves, did not show anything clearly due to interaction and the variability mentioned before. The graph showing the effects of age on anchor strength at various air contents (Figure 28, Appendix A) showed little change from 7 to 28 days age. At 28 days the 0% air content had the highest ultimate strength and 10% the lowest. Figure 29, Appendix A, shows the effects of concrete strength and air content on anchor strength. The 10 and 7% air contents had the lower anchor strengths while 0 and 5% the highest. To determine the effects of age at test, the anchor strength/concrete age ratio was plotted against compressive strength (Figure 30, Appendix A). With the exception of the 5% plot, all three other curves had the same relationship to each other based on the relative slopes. If both maturity and strength are considered then at earlier ages and lower compressive strengths the 5% air content had the highest ultimate anchor strength. At later ages and higher compressive strengths, the 0% air content anchor strengths were greater.

E. ANCHOR EXTENSIONS

Statistical analysis of Phase I anchor slip during torquing revealed that the concrete air content had a very significant effect (Table 13, Appendix A). This indicated that the percentage air content of any particular concrete had a definite effect on the magnitude of slip encountered after torquing and before any external load is applied. The concrete compressive strength did not have a significant effect on anchor slip, indicating that the amount of slip after torquing was independent of compressive strength. Interaction was not found to have been significant. The coefficients of variation from this series ranged from 0 to 40%, with an overall average of 26%. There was no discernible pattern to the variation. The 7% group had the highest average variation at 39% and 0% the lowest at 18%. This level of variation, with only four anchors per test, at a 90% confidence interval for the population mean resulted in roughly a 30% variation of the sample mean for each test. The 7% group's variation was almost half of the sample mean. Thus, the changes in slip from anchor to anchor may have occurred by chance alone. Figure 31, Appendix A, lower curves, are the effects of concrete air content and compressive strength on anchor slip as a result of torquing. The curves indicate that increasing air content increased the amount of slip.

The results of Phase II statistical analysis (Table 14, Appendix A) indicated that different levels of age or concrete air content had no significant effect on the level of anchor slip after torquing. Interaction was found to have been significant and the resulting error analysis determined that approximately 50% of the data variation was due to unassignable causes. The remaining data variation was due to air content and age. The variation ranged from 0 to 57%. No discernible trends in the variation were found. The 5 and 7% air contents had the highest average variation and 0% the lowest at 15%. Overall variation was 34%.

Figure 32, Appendix A, lower curves, are graphical representations of the data. The curves were moderately interactive and as a result of the high variability, no predictable pattern could be determined.

The torqued anchor slip results, for both phases, were evaluated with 4 anchors instead of 6 per set. The two strain monitored anchors from each set were not included due to the significantly greater amount of slip encountered versus the other 4 anchors that were not monitored. This was a result of a slightly higher than specified preload applied to the strain bolts as a result of an error in settings. The actual load was 5000 lb versus the intended 4800 lb.

Prior to the analysis of anchor slip at ultimate strength, a statistical evaluation was conducted (Tables 15-18, Appendix A) to determine if the slightly higher preloads applied to the strain monitored anchors caused a significant difference in results. Using representative data from Phase I and II it was determined that the strain monitored anchors were equivalent and no significant difference existed at the 95% level.

The analysis of variance tests from Phase I (Table 19, Appendix A) indicated that neither concrete air content nor compressive strength were significant to anchor extension at ultimate strength. However, interaction was significant which meant that the main effects calculated as nonsignificant could not be interpreted as indicated by an "F" test. An error analysis of the data indicated that 70% of the variation in data was due to air content and compressive strength. Random error comprised the remaining 30%. The coefficients of variation for this series ranged from 0 to 13%. The overall sample mean was 8%. No discernible trends in variation were observed.

Figure 31, Appendix A, shows the plots of Phase I compressive strength versus ultimate load anchor slip. The curves were highly interactive and difficult to interpret. When the data were adjusted to account for concrete age (Figure 33, Appendix A), the only apparent trend was close grouping of the curves indicating ultimate anchor slip was independent of the factors considered.

Phase II analysis of variance of ultimate strength anchor slip (Table 20, Appendix A) indicated that the percentage air content had a significant effect. Concrete age, however, was not determined to have been significant. Interaction was not significant. The coefficients of variation for Phase II ranged from 7 to 28%. The averages for each air content ranged from 12 to 17%, with an overall total sample mean of 14%. No trends in variation were observed.

Plots of ultimate strength anchor slip at the three test ages (Figure 32, Appendix A) indicated both 7 and 10% air contents had the highest slip at any age and 5% the lowest. As determined in the statistical analysis, all curves were relatively horizontal indicating little change of slip with age.

When an anchor reached its ultimate strength, the loading apparatus was left in place and as pumping of the ram continued the load fell gradually to zero as the anchor slid out of the wedges. Figure 34, Appendix A, is a typical example of this failure mechanism. Since in every case the anchor wedges remained in the hole, 7 holes were selected from the various slabs and the anchor holes containing the wedges cored out. Each core was then carefully sawn open to expose the wedges and the relative slip of each wedge set noted in Table 21, Appendix A, along with pertinent data about each anchor.

The typical anchor used in this program was installed 3-1/4 inches from the bottom of the anchor to the concrete surface. This placed the bottom of the wedges at approximately 2-3/4 inches below the surface. Thus, as can be seen in Figures 35 and 36, Appendix A, the wedges slid almost 0.2 inches. This was anchor number two from Phase I, 0% air content and 2000 psi. The overall slip of this anchor was among the highest in the set and was one of the highest recorded in Phase I. Figures 37 and 38, Appendix A, are of anchor number eighteen, Phase I. The total slip of this anchor was about average although the ultimate load was one of the lowest in that set. Figures 39 and 40, Appendix A, are of anchor number fifteen from Phase II. The total anchor slip of this anchor was among the lowest of any air content and the load the highest. The wedges in Figures 41 and 42, Appendix A, showed the greatest relative slip, 0.25 inches, overall. The total slip of the anchor itself was also among the largest of any of the anchors in Phase II.

The wedges were subsequently removed from each core half and the concrete washed in a hydrochloric acid solution to expose the concrete beneath each wedge. No significant detail could be observed, however.

CHAPTER IV

DISCUSSION

One of the problems encountered during this program was occasional cracking of the slabs during anchor tests. Specifically, in Phase I the edge anchors (number six and eleven) split the corner through the anchor hole from the main slab into a triangle shaped wedge. These failures always occurred when the anchor was at ultimate strength, and was sudden in nature. This behavior was noted only for the tests conducted at three days age and are believed due to immature concrete. Rough analysis of the data indicated no significant strength differences between these anchors and the others.

Phase II did not show this form of cracking due to later ages of testing; however, the 0 and 5% concrete slabs did develop cross slab cracking. Figure 43, Appendix A, shows a photograph taken of the 5% slab at the end of testing. This cracking was typical of both slabs. There was no cracking observed in either the 7 or 10% slabs. The primary cause of this was believed due to problems in the support of the slab. Figure 44, Appendix A, shows a side view of the method of support. Before a slab was poured, the form was shimed up to level the top surface for screeding. Then, prior to anchor testing, the shims were removed from under the cross members to allow full bearing on the floor. The shims were accidentally left in place for the 0 and 5% slabs. After the first cores were removed from the middle of these slabs and loads applied, they began to split. After corrective action was taken, further cracking was minimized but still remained a problem. These cracks did affect certain anchors, primarily at ultimate strength. When this occurred the results were removed from analysis.

When an anchor is embedded past a certain minimum depth, so that failure does not occur as a concrete cone pull out, then as a load is applied and the wedges separate, large pressures develop against the sides of the hole and localized concrete compressive failure occurs directly around the wedges. The position of the larger aggregate particles within the concrete matrix next to the wedges most likely determines the level at which this failure originates. A wedge bearing against a large aggregate particle within the matrix will necessarily require a greater force to initiate wedge movement and localized failure than if it were next to mortar. This mechanism certainly accounts for a portion of the variability encountered with wedge anchors. The properties of the surrounding mortar also affect the level of load. Based on the tests completed, it is felt that as the force of a wedge against the concrete increases, the pressure crushes the air bubbles in the mortar. Thus, as the air content in the concrete increases, the wedges would have to spread a correspondingly greater distance to achieve the same load. The stresses exerted by a wedge on the surrounding concrete were calculated to have been roughly 5700 psi for a pretorque of 40 ft-lbs.

The results from the cut sections containing the wedges indicated that as a load is applied to an anchor the wedges begin to separate and slide upward in the hole a certain amount varying from 1/16 to 1/4 inch depending on the concrete. When the wedges have slid the maximum amount, the tapered shank of the anchor continues sliding through the wedges, forcing them deeper into the surrounding concrete. Ultimately, the shaft clears the top of the wedges with a corresponding total loss of load.

The introduction of a preload in a wedge bolt anchor is normally accomplished by tightening the nut onto the stud against a restraint. The two methods advocated for accomplishing this in the field are: (a) turn-of-the-nut or (b) tightening to a predetermined torque with a torque wrench. The latter method was chosen for this program based on previous research within TVA [Cones (2)]. It was found that if a set number of turns were used for setting anchors, then due to variations in bit diameter and concrete properties, some anchors would not receive enough preload while others might exceed their ultimate capacity. Although the turn-of-the-nut method could have been used in this study, the torque method was chosen because it parallels current practice within TVA.

Once the preload was applied, the losses that occurred were believed to have been caused by several factors. Both the lift-off and strain bolt methods indicated that the majority of losses which occurred did so within twenty minutes or less. This same result has been noted in other research (6). Concrete creep, which normally occurs over a longer period of time, was not felt to have been a significant factor in the initial stages of loss. Based on the small amounts of slip determined from the strain tests, the wedges could have slid up slightly in the hole or due to a sustained load effect spread out further into the concrete. The preload loss with time curves generated with the strain bolts must be viewed with caution. As a relative indicator of the influence of air content on anchor preload loss in concrete, the results were felt to have been reasonably reliable. However, the magnitudes of losses were not strictly correct. The calculations of corrected load were based on the assumption that the amount of anchor stud movement subsequent to torquing was nearly the same whether the anchor had the strain bolt attached or not. However once the original load was applied, the strain bolt acted somewhat as a spring maintaining a slower rate of load loss, and a subsequent greater degree of slip than in an unmonitored anchor. Correspondingly, the actual loads should be somewhat greater than those calculated in Appendix C, the magnitude of difference depending on elapsed time.

Thus, if an anchor and a strain bolt-anchor combination were both preloaded to 5000 lbs, each would begin to lose load immediately. This has been confirmed by both methods. The magnitude of loss should be reasonably close during the early stages since the load and slip in each anchor are still nearly equivalent. As time continues, the slip in the simple anchor will have relaxed most of the elastic deformation; whereas, the strain bolt system will have realized less relaxation; thus greater slip should occur. It was also noted that most of the curves tend to level off somewhere around 24 hours, indicating an anchor would not lose all its preload. Over a greater time period a typical anchor (without a strain bolt) might continue to relax causing a certain additional loss of preload force. The magnitude of this additional loss is uncertain. Burdette and Tinsley (7) reported that for a concrete with 10% air content the wedge bolts lost an average of approximately 53% after 90 days.

Although the corrected strain bolt loads underestimated the actual forces remaining in an anchor, the analysis was made to define a lower limit. It was felt that lift-off loads did not completely reflect all the losses which occurred. Thus, the actual residual load at any time existing in an anchor would lie somewhere between the lift-off load and the corrected load.

The use of a torque wrench to measure torque loss directly was not consistent. The variability encountered was too high to permit accurate determinations. The source of this variation may have been a result of friction and interlock between the nut and washers so that after five hours, during the torque loss tests, these factors interacted to obscure the actual amount of residual torque. This method of loss determination was not considered a viable method. The variability was too great to be reliable.

The lift-off method of preload loss evaluation did not indicate further significant time dependent loss after the initial portion. However since a slip of 1 mil or less would have a significant effect on load at later ages, it is possible that as a result of concrete creep these losses would occur at much greater ages.

One of the benefits of wedge anchors is the ability to apply and monitor a preload. If, as determined from the tests, roughly half or more of the preload is lost, then depending on the factor of safety utilized, connections might eventually become loose. Although all of the anchors tested by the lift-off method were above the TVA specified working load requirement for 1/2 inch diameter anchors, if further reductions in loads occurred, then only the 5 and possibly 7% concretes at 28 days age and 4000 psi compressive strength might pass. The level of preload force in an anchor was not found to have any significant effect on the ultimate capacity of a wedge anchor.

CHAPTER V

CONCLUSIONS

Based on the results of the tests, certain fundamental conclusions were reached. They are briefly outlined as follows:

1. The slip of an anchor during application of torque occurs as a result of several mechanisms acting simultaneously. For a standard level of preload the magnitude of slip was most affected by the concrete air content. Compressive strength did not appear significant. The amount of slip after torquing also appears to be directly affected by air content.

2. The amount of preload lost in an anchor, subsequent to torquing, is time dependent. Roughly half of the preload is lost within five to ten minutes. Based on the strain tests it was fairly certain that air content had a significant effect on preload loss magnitude.

3. The age of concrete could not be shown statistically to have had any significant effect on the properties tested.

4. The ultimate strength of an anchor was found to have been significantly affected by air content. The higher an air content, above roughly 7%, the lower the anchor strength. An additional problem with anchors set in high air content concrete is increased variability. The chances of an anchor testing below the minimum required strength is much greater for the higher air contents. Thus, to ensure that

variability is kept to a minimum and optimum ultimate anchor strengths achieved, the hardened air content should be kept below approximately 7%. Based on the results of this and other tests (7), the air content range specified by TVA (5-1/2 ± 1-1/2) is considered a reasonable design criteria.

If the air content of a particular in situ concrete mass were found to be greater than this recommended range, then reduction of anchor design loads should be considered.

Compressive strength of the concrete also had a very significant effect. The higher the concrete strength, the greater the ultimate anchor capacity.

5. The concrete air content does appear to affect the magnitude of ultimate anchor slip. Thus, higher air contents result in greater anchor slip. The effect of compressive strength on ultimate anchor slip was unclear due to interaction effects. However, based on the other relationships, it is hypothesized that higher compressive strengths would result in less slip at ultimate anchor strength.

Although concrete age was not shown to have been significant, it is now felt ages chosen were probably too closely grouped to enable a reliable determination. Future tests to identify this effect should vary over a much wider range.

The lift-off method of preload determination still appears to be the best method available, since the equipment is fairly simple and the test quick and repeatable. However, this method does not show the further losses that are believed to occur with time.

The problems of cracking in several of the slabs was felt to have been a problem of slab size and support. Future work should utilize larger concrete slabs placed on grade to eliminate support problems and cracking.

Finally, to ensure optimum anchor behavior, concrete should have an air content of approximately 7% or less and a compressive strength above 3000 psi.

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APPENDIXES

APPENDIX A

TABLES AND FIGURES

Table 1. Concrete Properties and Mixture Proportions

Property	Design Percent Entrained Air, %			
	0	5	7	10
Cement, P.C.Y.	534	534	534	534
W/C ratio	0.62	0.56	0.54	0.48
Water, P.C.Y.	347	321	305	263
Sand, P.C.Y.	1659	1572	1526	1442
Mortar volume, cf/cy	17.9	17.9	17.9	17.9
Slump, in				
Phase I	3	2-3/4	3-1/2	3-1/4
Phase II	2-1/2	3-1/2	2-1/2	3-3/4
Air entraining agent, O.P.Y.	0	2.3	3.9	8.0
Coarse Aggregate, P.C.Y.				
3/4 in	1115.1	1115.1	1115.1	1115.1
3/8 in	477.9	477.9	477.9	477.9
Air Content, % Determined[1]				
Phase II	0.5	3.4	7.0	8.3

Materials

Cement: Ideal Cement, type I- specific gravity = 3.10
Sand: Vulcan Materials Co., manufactured limestone sand - specific gravity = 2.78
Coarse aggregate: Vulcan Materials Co., crusher run - specific gravity = 2.82
Air entraining agent: Protex Industries, Protex air entraining agent (vinsol resin)

Aggregate Gradation[2]

Sand (percent retained)

8 = 10
16 = 30
30 = 27
50 = 19
#100 = 11

Fineness Modulus = 3.00

Coarse Aggregate (percent retained)

3/4 in M.S.A.

3/4 to 1/2 = 60
1/2 to 3/8 = 40

3/8 in M.S.A.

3/8 to 4 = 92.5
4 to 8 = 7.5

[1] Determination of air content made from unit weights taken from 3 x 6 in cores.

[2] Based on TVA General Construction Specification No. G-2, R-3.

Table 2. Hardened Concrete Physical Properties

Concrete Air Content (%) Plastic	Concrete Air Content (%) Design	Age at Test (days)	Compressive Strength of Concrete Cylinders (psi) 6 x 12 in	Compressive Strength of Concrete Cylinders (psi) 3 x 6 in	Specific Gravity (3 x 6)	Splitting Tensile Strength (psi) 6 x 12 in	Chord Modulus of Elasticity (ksi x 10^3) 6 x 12 in
				Phase I			
1.8	0	3	2,070	2,065		335	3.46
		9	2,815	2,815	2.39	410	--
		35	4,210	3,965		486	4.80
4.7	5	2	2,120	--		--	--
		3	2,425	2,870	2.35	349	3.77
		21	3,960	4,040		476	--
7.2	7	3	2,280	2,610		325	3.37
		7	3,015	3,220	2.32	372	3.77
		35	4,068	4,430		539	4.77
10.4	10	3	2,060	2,040		257	3.22
		10	2,920	2,660	2.25	357	--
		88	3,875	3,920		--	4.58
				Phase II			
3.5	0	7	3,185	3,290			4.76
		14	3,575	3,540	2.44	--	5.24
		28	4,050	4,205			5.48
5.2	5	7	3,325	3,465			4.42
		14	3,625	3,645	2.43	--	4.91
		28	3,980	3,995			4.71
7.1	7	7	3,075	3,135			4.28
		14	3,415	3,460	2.35	--	4.28
		28	3,730	3,960			4.86
10.2	10	7	2,575	2,585			3.81
		14	3,135	3,020	2.31	--	4.03
		28	3,590	3,255			4.39

Table 3. Phase I Summary of Wedge Anchor Performance

Age (days)	Design Concrete Air Content (%)	Design Concrete Compressive Strength (psi)	Applied Tensile Load at Liftoff: 5 min (lb)	S[1] (lb)	C.O.V.[2] (%)	5 hr (lb)	S (lb)	C.O.V. (%)	Applied Tensile Load at Ultimate: Load (lb)	S (lb)	C.O.V. (%)	Relative Anchor Slip, After Torquing: (in)	S (in)	C.O.V. (%)	Relative Anchor Slip, Ultimate Capacity: (in)	S (in)	C.O.V. (%)
3	0	2000	--	--	--	2617	200	7.6	6630	295	4.4	0.13	0.0	0.0	0.79	0.06	7.6
9	0	3000	--	--	--	3195	530	16.6	7720	980	12.7	0.21	0.07	33.3	0.66	0.05	7.6
35	0	4000	2500	260	10.4	2805	160	5.7	9430	260	2.8	0.14	0.03	21.4	0.52	0.05	9.6
--	5	2000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
3	5	3000	2970	380	12.8	3045	400	13.1	7375	690	9.4	0.28	0.06	21.4	0.68	0.06	8.8
21	5	4000	2600	305	11.7	3075	595	19.3	8320	1105	13.3	0.22	0.06	27.3	0.65	0.05	7.7
3	7	2000	2725	340	8.4	2835	603	21.3	7440	790	10.6	0.19	0.07	36.8	0.67	0.06	8.9
7	7	3000	2615	230	8.8	2915	360	12.3	8435	245	2.9	0.16	0.06	37.5	0.67	0.06	9.0
35	7	4000	3105	320	10.3	3550	595	16.8	9895	1305	13.2	0.17	0.07	41.2	0.60	0.06	10.0
3	10	2000	3145	600	19.1	3425	665	19.4	5665	760	13.4	0.33	0.07	21.2	0.75	0.00	0.0
10	10	3000	2681	305	11.4	2900	390	13.4	7495	600	8.0	0.34	0.06	17.6	0.84	0.06	7.1
88	10	4000	3030	55	2.0	3165	160	5.0	8020	955	11.9	0.31	0.07	22.6	0.56	0.07	12.5

[1]Corrected by n-1 weighting.

[2]Coefficient of variation = $\frac{S}{\bar{x}}$

Table 4. Phase II Summary of Wedge Anchor Performance

Age (days)	Design Concrete Air Content (%)	Design Concrete Compressive Strength (psi) 6 x 12 in	3 x 6 in	Torque Loss (in-lb)	S[1] (in-lb)	C.O.V.[2] (%)	Applied Load at Ultimate: Load (lb)	S (lb)	C.O.V. (%)	After Torquing (in)	S (in)	C.O.V. (%)	At Ultimate Capacity (in)	S (in)	C.O.V. (%)
7	0	3185	3290	81.3	19.3	23.7	8,425	945	11.2	0.09	0.04	44.4	0.63	0.08	12.7
14	0	3575	3540	62.5	23.6	37.8	9,675	426	4.4	0.06	0.00	0.0	0.61	0.17	27.9
28	0	4050	4205	91.7	12.6	13.7	10,080	358	3.6	0.13	0.00	0.0	0.60	0.06	10.0
7	5	3325	3465	110.0	28.3	25.7	9,642	448	4.6	0.17	0.06	35.3	0.66	0.11	16.7
14	5	3625	3645	67.5	27.5	40.7	9,738	320	3.3	0.06	0.00	0.0	0.54	0.06	11.1
28	5	3980	3995	58.3	7.6	13.0	9,083	629	6.9	0.13	0.06	46.2	0.54	0.07	13.0
7	7	3075	3135	77.5	41.1	53.0	7,825	436	5.6	0.09	0.04	44.4	0.69	0.06	8.7
14	7	3415	3460	66.3	11.1	16.7	8,500	391	4.6	0.17	0.06	35.3	0.73	0.05	6.8
28	7	3730	3960	75.0	19.6	26.1	9,100	690	7.6	0.06	0.00	0.0	0.73	0.15	20.5
7	10	2575	2585	77.5	21.0	27.1	7,292	640	8.8	0.13	0.00	0.0	0.75	0.11	14.7
14	10	3135	3020	83.8	21.7	25.9	7,257	1284	17.7	0.17	0.03	17.6	0.73	0.05	6.8
28	10	3590	3255	90.0	52.3	58.1	7,175	932	13.0	0.14	0.08	57.1	0.75	0.16	21.3

[1] Corrected by n-1 weighting.

[2] Coefficient of variation = $\frac{S}{\bar{x}}$.

Table 5. Phase I Summary Table for Analysis of Variance of Loads Recorded at Lift-Off

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Air content	5.68×10^5	2	2.84×10^5	5.20×10^{-1}*	6.94
Compressive strength	3.01×10^5	2	1.51×10^5	2.90×10^{-1}*	6.94
Interaction	2.16×10^6	4	5.40×10^5	3.49**	2.73
Error	4.18×10^6	27	1.55×10^5		
Total	7.21×10^6	35			

*Not significant at the 0.05 level.

**Significant at the 0.05 level

Note: Since interaction is significant, an error analysis of the data was performed to define the variation. Approximately 60% of the variation is due to error and 40% to changing levels of air content and compressive strength.

Table 6. Phase II Summary Table for Analysis of Variance of Anchor Torque Loss

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Air content	7.05×10^2	3	2.35×10^2	2.30×10^{-1}*	4.76
Concrete age	2.20×10^3	2	1.10×10^3	1.06*	5.14
Interaction	6.24×10^3	6	1.04×10^3	1.47*	2.40
Error	2.54×10^4	36	7.07×10^2		
Total	3.46×10^4	48			

*Not significant at the 0.05 level.

Table 7. Phase I Summary Table for Analysis of Variance of Ultimate Strength Load Comparison at 0% Air Content

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Monitoring method	2.20×10^4	1	2.20×10^4	1.00×10^{-2}*	7.71
Error	8.24×10^6	4	2.06×10^6		
Total	8.26×10^6	5			

*No significant difference at the 0.05 level.

Table 8. Phase I Summary Table for Analysis of Variance of Ultimate Strength Load Comparison at 7% Air Content

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Monitoring method	4.87×10^5	1	4.87×10^5	3.30×10^{-1}*	7.71
Error	5.81×10^6	4	1.45×10^6		
Total	6.30×10^6	5			

*No significant difference at the 0.05 level.

Table 9. Phase II Summary Table for Analysis of Variance of Ultimate Strength Load Comparison at 0% Air Content

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Monitoring method	1.20×10^{-2}	1	1.20×10^{-2}	1.70×10^{-2}*	7.71
Error	2.86	4	7.14×10^{-1}		
Total	2.87	5			

*No significant difference at the 0.05 level.

Table 10. Phase II Summary Table for Analysis of Variance of Ultimate Strength Load Comparison at 7% Air Content

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Monitoring method	1.84×10^{-1}	1	1.84×10^{-1}	3.50×10^{-1}*	7.71
Error	2.07	4	5.19×10^{-1}		
Total	2.26	5			

*No significant difference at the 0.05 level.

Table 11. Phase I Summary Table for Analysis of Variance of Ultimate Anchor Load

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Air content	2.12×10^{7}	2	1.06×10^{7}	1.61×10^{1}**	6.94
Compressive strength	5.78×10^{7}	2	2.89×10^{7}	4.40×10^{1}**	6.94
Interaction	2.63×10^{6}	4	6.57×10^{5}	1.26*	2.60
Error	2.34×10^{7}	45	5.20×10^{5}		
Total	1.05×10^{8}	53			

*Not significant at the 0.05 level.

**Significant at the 0.05 level.

Table 12. Phase II Summary Table for Analysis of Variance of Ultimate Anchor Load

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Air content	4.15×10^{1}	2	2.07×10^{1}	1.48×10^{1}*	6.94
Concrete age	8.05	2	4.02	2.88**	6.94
Interaction	5.59	4	1.40	2.60*	2.60
Error	2.43×10^{1}	45	5.41×10^{-1}		
Total	7.94×10^{1}	53			

*Significant at the 0.05 level.

**Not significant at the 0.05 level.

Since interaction is significant, an error analysis of the data was performed to define the variation. Approximately 80% of the variation is due to error and 20% to changing levels of air content and concrete age.

Table 13. Phase I Summary Table for Analysis of Variance of Anchor Slip during Torquing

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Air content	1.93×10^{-1}	2	9.65×10^{-2}	4.71×10^{1}*	6.94
Compressive strength	5.36×10^{-3}	2	2.70×10^{-3}	1.32**	6.94
Interaction	8.20×10^{-3}	4	2.05×10^{-3}	5.33×10^{-1}**	2.73
Error	1.04×10^{-1}	27	3.85×10^{-3}		
Total	3.10×10^{-1}	35			

*Significant at the 0.05 level.

**Not significant at the 0.05 level.

Table 14. Phase II Summary Table for Analysis of Variance of Anchor Slip during Torquing

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Air content	2.00×10^{-2}	3	6.67×10^{-3}	8.40×10^{-1}**	4.76
Concrete age	2.04×10^{-2}	2	1.02×10^{-2}	1.29**	5.14
Interaction	4.76×10^{-2}	6	7.93×10^{-3}	5.22*	2.40
Error	5.47×10^{-2}	36	1.52×10^{-3}		
Total	1.43×10^{-1}	47			

*Significant at the 0.05 level.

**Not significant at the 0.05 level.

Note: Since interaction is significant, an error analysis of the data was performed to define the variation. Approximately 50% of the variation is due to error and 50% to changing levels of air content and concrete age.

Table 15. Phase I Summary Table for Analysis of Variance of Total Anchor Slip Comparison at 0% Air Content

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Monitoring method	2.00×10^{-5}	1	2.00×10^{-5}	1.00×10^{-3}*	7.71
Error	7.99×10^{-2}	4	2.00×10^{-2}		
Total	8.00×10^{-2}	5			

*No significant difference at the 0.05 level.

Table 16. Phase I Summary Table for Analysis of Variance of Total Anchor Slip Comparison at 7% Air Content

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Monitoring method	2.00×10^{-3}	1	2.00×10^{-3}	4.40×10^{-1}*	7.71
Error	1.80×10^{-2}	4	4.50×10^{-3}		
Total	2.00×10^{-2}	5			

*No significant difference at the 0.05 level.

Table 17. Phase II Summary Table for Analysis of Variance of Total Anchor Slip Comparison at 0% Air Content

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Monitoring method	1.00×10^{-2}	1	1.00×10^{-2}	2.00*	7.71
Error	2.00×10^{-2}	4	5.00×10^{-3}		
Total	3.00×10^{-2}	5			

*No significant difference at the 0.05 level.

Table 18. Phase II Summary Table for Analysis of Variance of Total Anchor Slip Comparison at 7% Air Content

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Monitoring method	1.00×10^{-2}	1	1.00×10^{-2}	1.25*	7.71
Error	3.00×10^{-2}	4	8.00×10^{-3}		
Total	4.00×10^{-2}	5			

*No significant difference at the 0.05 level.

Table 19. Phase I Summary Table for Analysis of Variance of Total Anchor Slip

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Air content	6.00×10^{-2}	2	3.00×10^{-2}	7.90×10^{-1}*	6.94
Compressive strength	3.30×10^{-1}	2	1.70×10^{-1}	4.47*	6.94
Interaction	1.50×10^{-1}	4	3.80×10^{-2}	1.41×10^{1}**	2.60
Error	1.20×10^{-1}	45	2.70×10^{-3}		
Total	6.60×10^{-1}	53			

*Not significant at the 0.05 level.

**Significant at the 0.05 level.

Note: Since interaction is significant, an error analysis of the data was performed to define the variation. Approximately 30% of the variation is due to error and 70% to changing levels of air content and compressive strength.

Table 20. Phase II Summary Table for Analysis of Variance of Total Anchor Slip

Source of Variation	Sum of Squares	Degree of Freedom	Mean Square	F_0	$F_{0.05}$
Air content	3.10×10^{-1}	3	1.03×10^{-1}	1.08×10^{1}*	4.76
Concrete age	1.80×10^{-2}	2	9.00×10^{-3}	1.00**	5.14
Interaction	5.70×10^{-2}	6	9.50×10^{-3}	8.70×10^{-1}**	2.25
Error	6.55×10^{-1}	60	1.09×10^{-2}		
Total	1.04	71			

*Significant at the 0.05 level.

**Not significant at the 0.05 level.

Table 21. Magnitude of Anchor Wedge Slip Relative to Concrete[1]

Anchor No.	Concrete: Air Content, %	Concrete: Age, Days	Concrete: Strength, Psi	Ultimate Anchor: Load, Lbs.	Ultimate Anchor: Slip, In.	Ultimate Anchor: Wedge Slip, In.	Comments
					Phase I		
2	0	3	2,000	6,350	0.88	0.19	Anchor slip was one of the highest in set, load was lower than average.
4	7	3	2,000	6,250	0.63	0.13	Anchor slip was about average, lowest load of set.
18	10	88	4,000	6,245	0.63	0.13	Anchor slip about average, lowest load of set.
					Phase II		
2	0	7	3,200	8,600	0.69	0.13	Anchor slip one of the highest in set, load was about average.
15	7	28	3,700	9,800	0.50	0.13	Anchor slip was the least of the set, load one of the highest.
4	10	7	2,600	7,100	0.88	0.25	Anchor slip one of the highest in set, load was about average.
14[2]	10	28	3,600	8,900	0.75	0.19	Anchor slip about average in set, load was the highest in set.

[1]As determined from cores. [2]Strain monitored anchor.

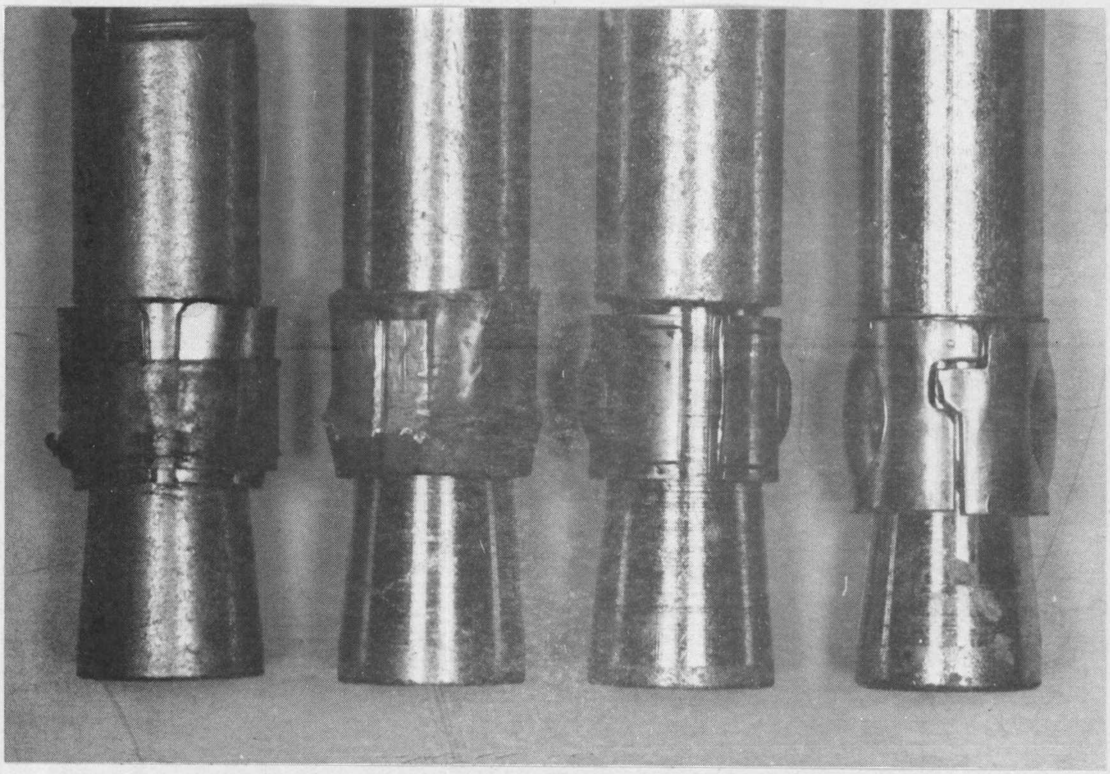

Figure 1. Typical wedge designs.

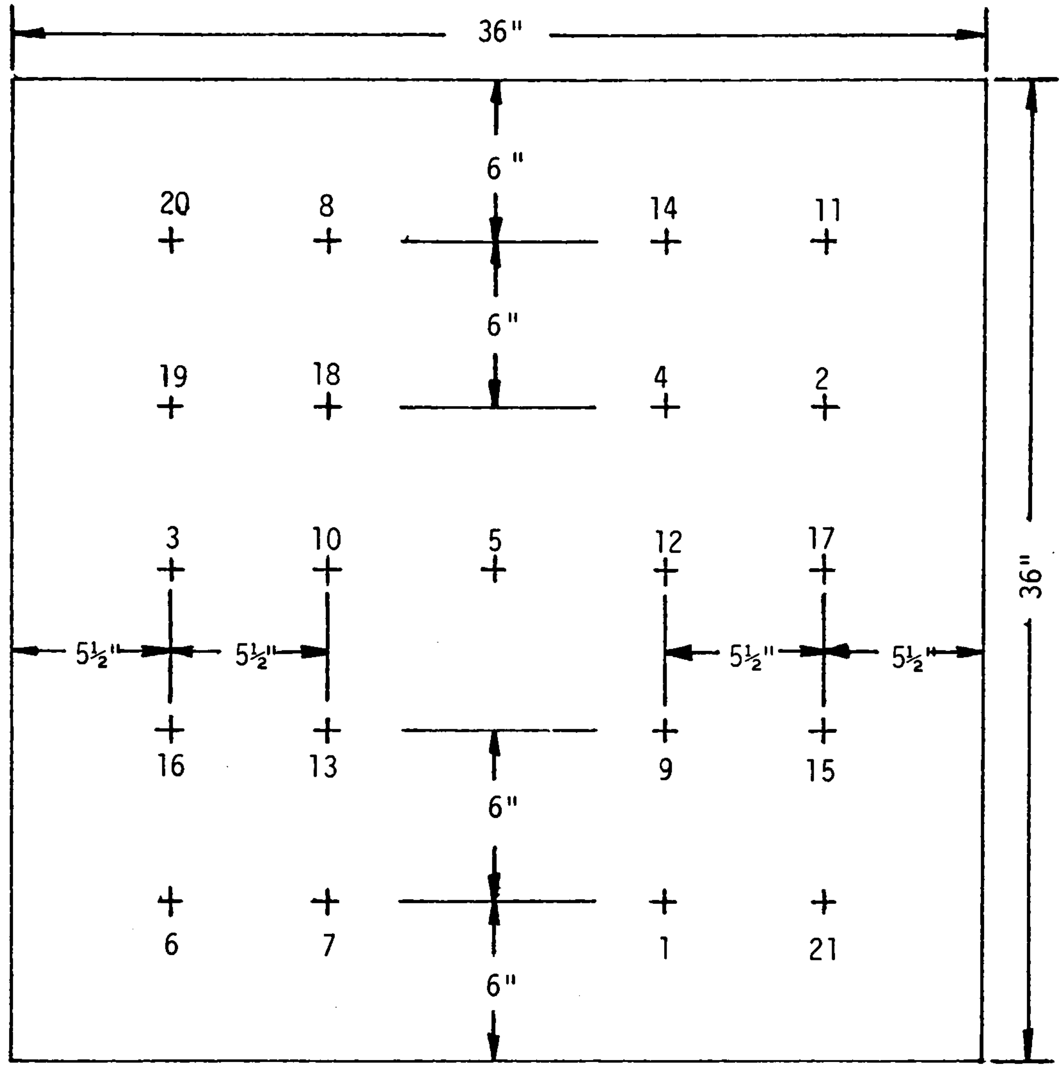

Figure 2. Layout and spacing of anchors.

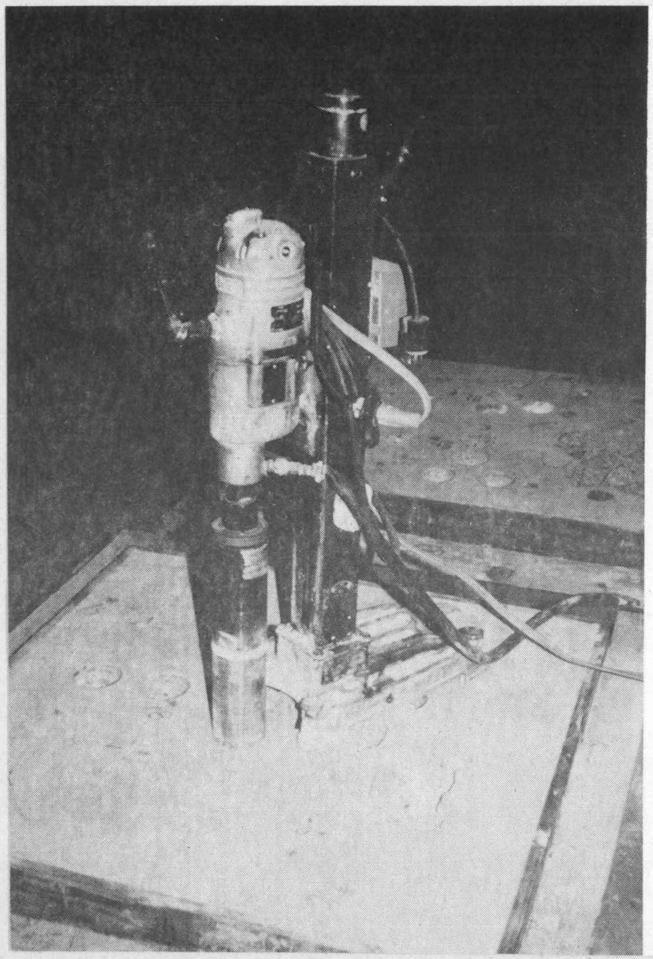

Figure 3. Drilling a core for strength testing.

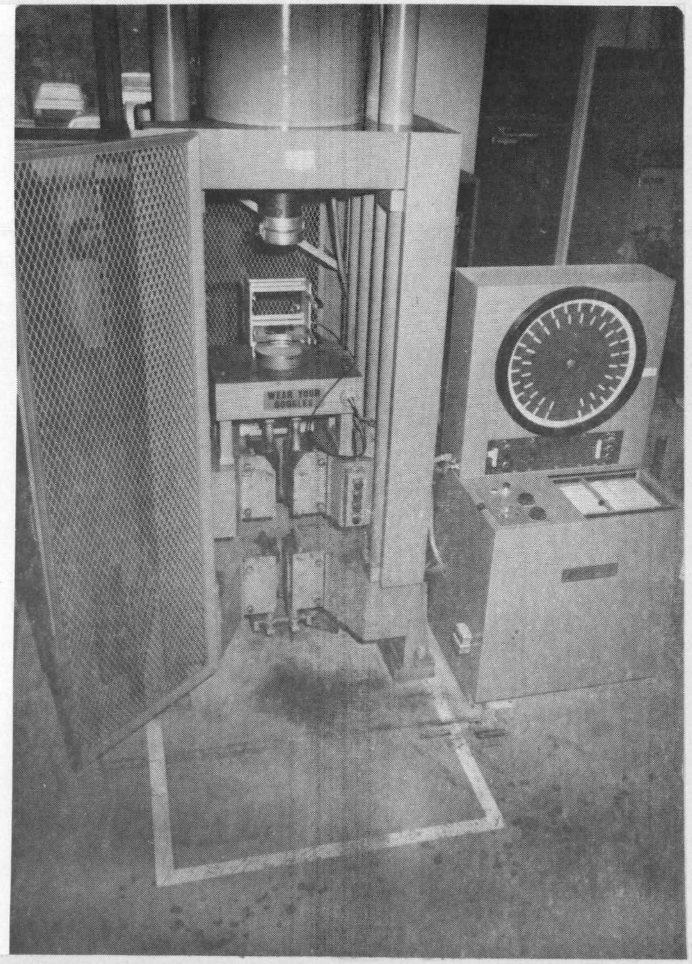

Figure 4. Baldwin universal testing machine used for concrete strength and modulus of elasticity determinations.

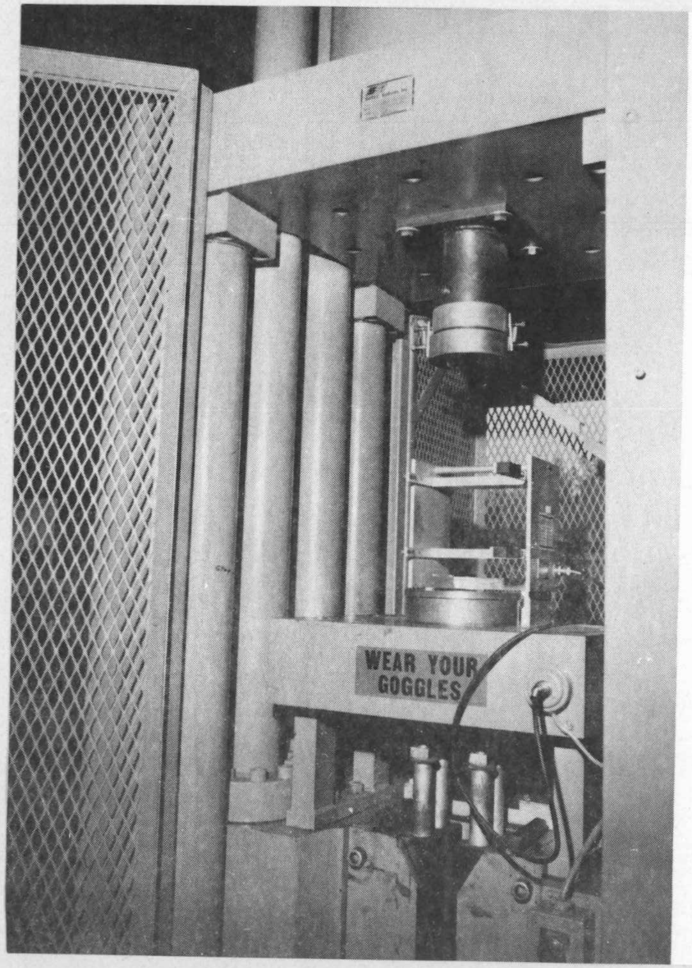

Figure 5. Compressometer used to measure deflection in cylinders.

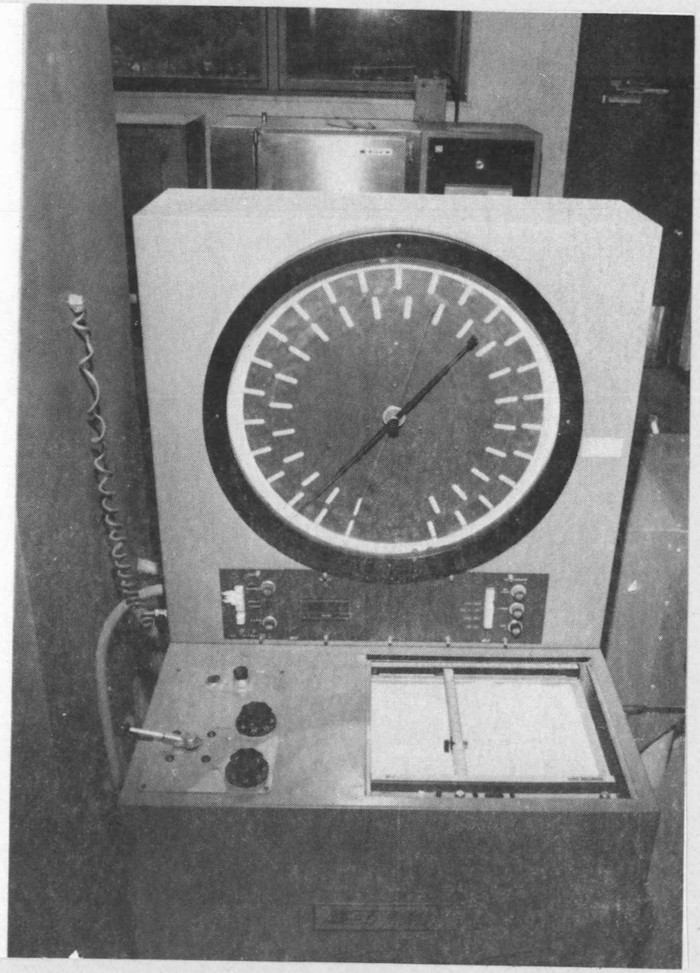

Figure 6. Chart recorder used to plot load-deflection curves.

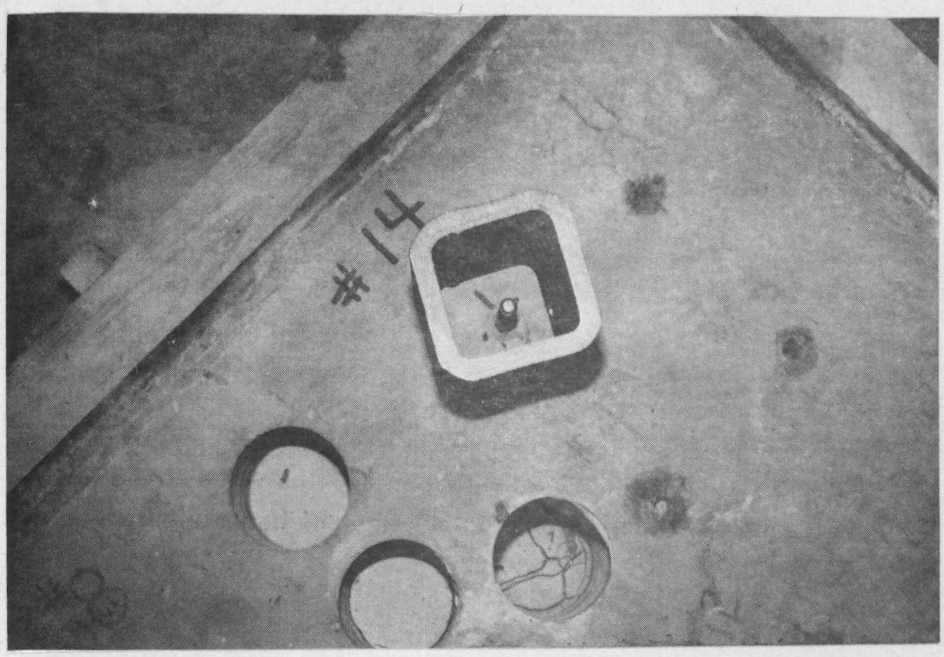

Figure 7. Steel tubing used in strain tests.

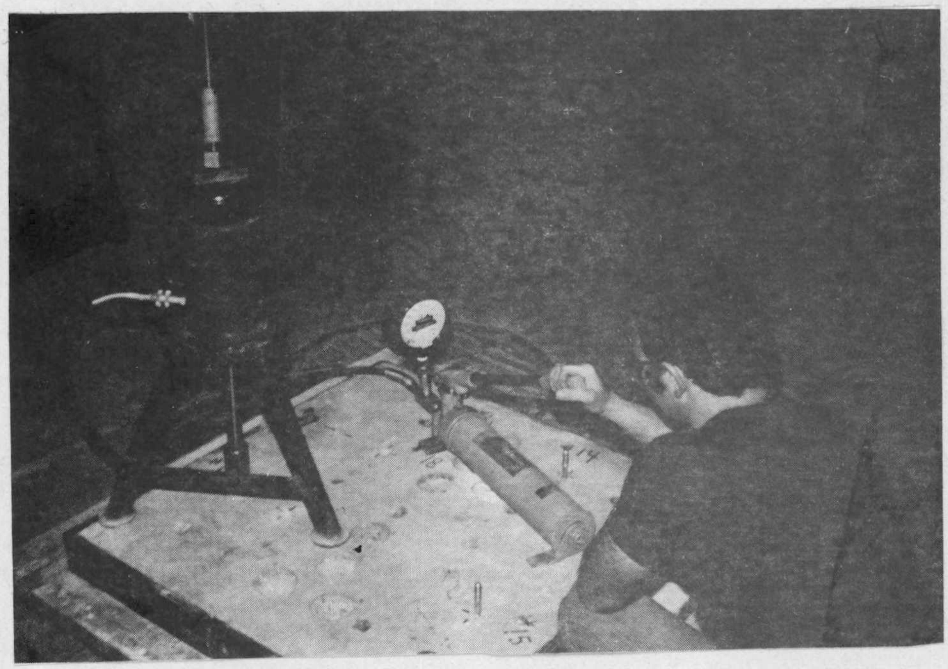

Figure 8. Anchor loading apparatus.

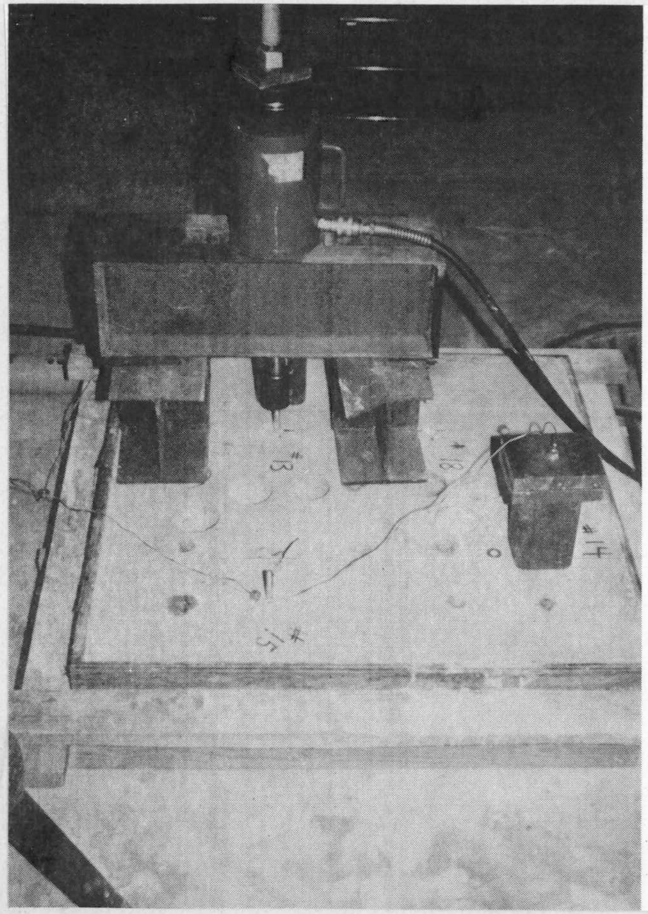

Figure 9. Alternate loading apparatus.

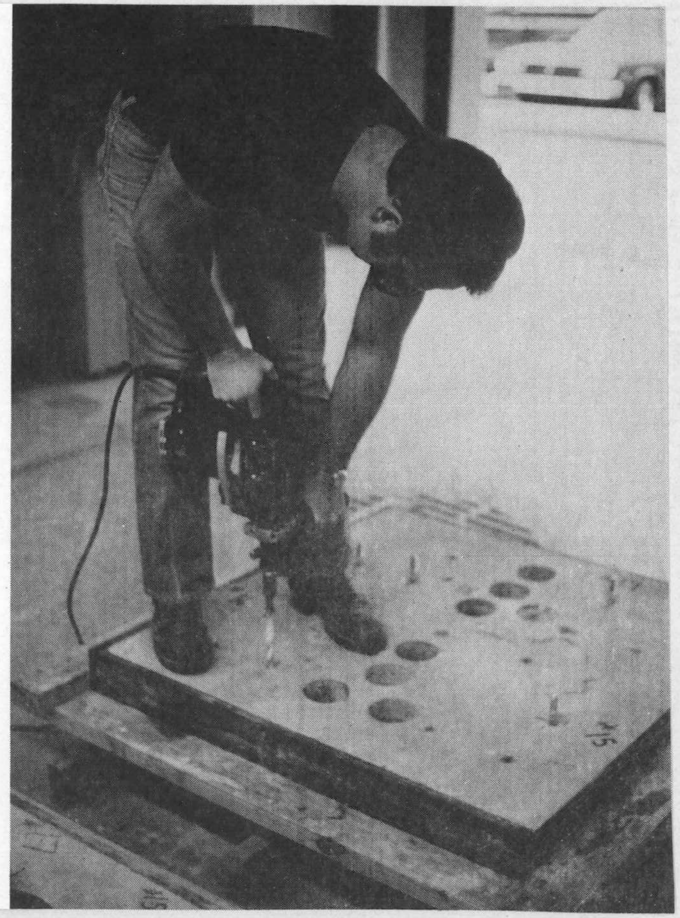

Figure 10. Drilling anchor holes.

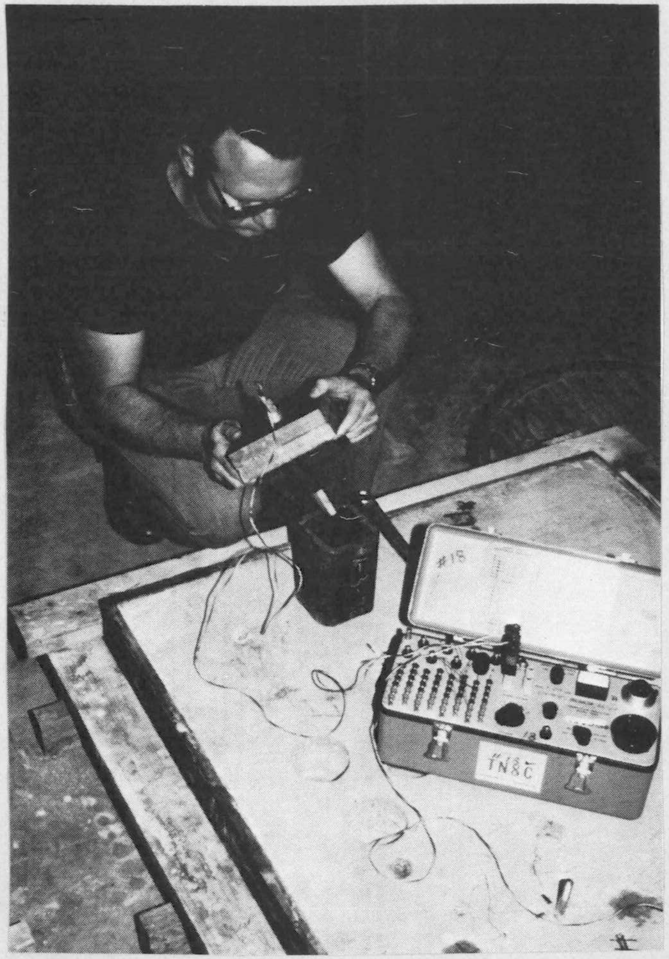

Figure 11. Attachment of strain monitoring apparatus to anchor.

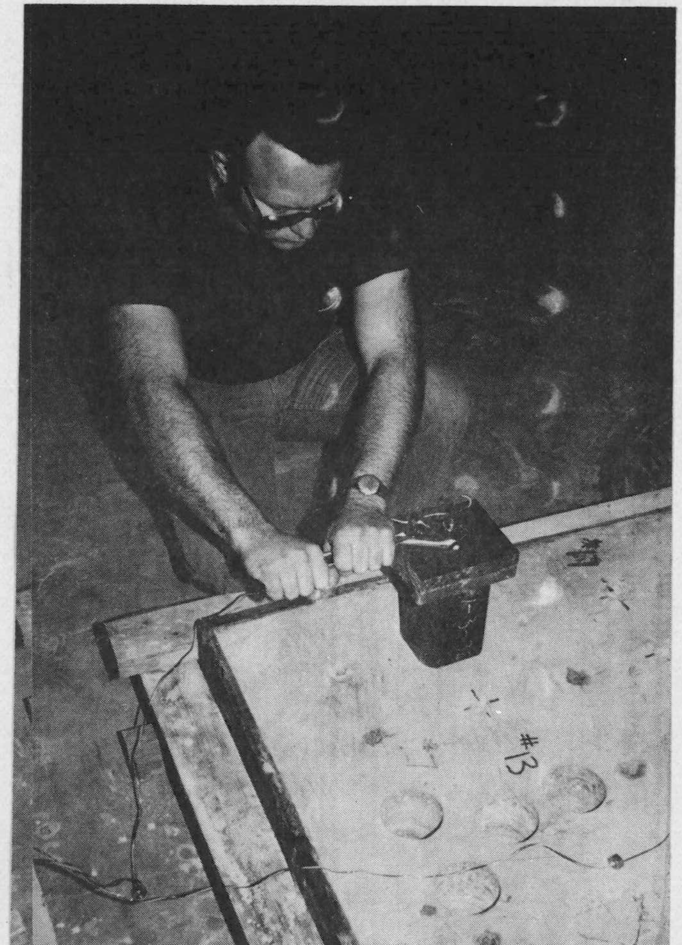

Figure 12. Application of preload.

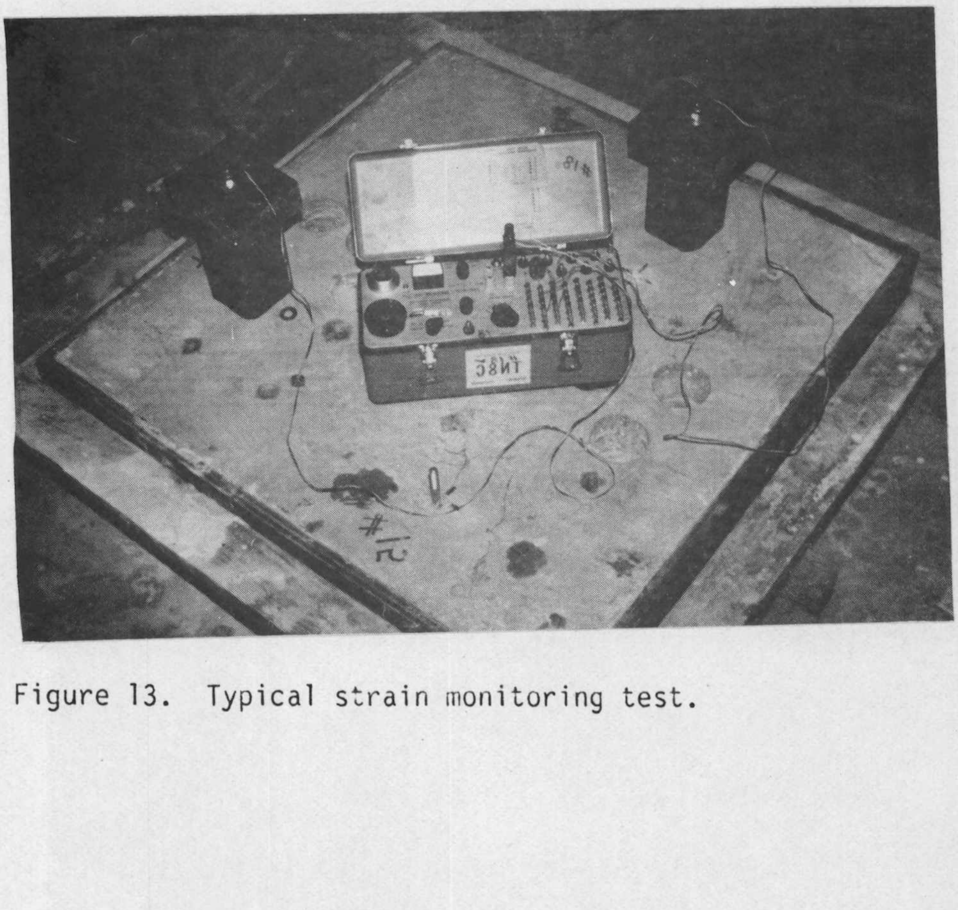

Figure 13. Typical strain monitoring test.

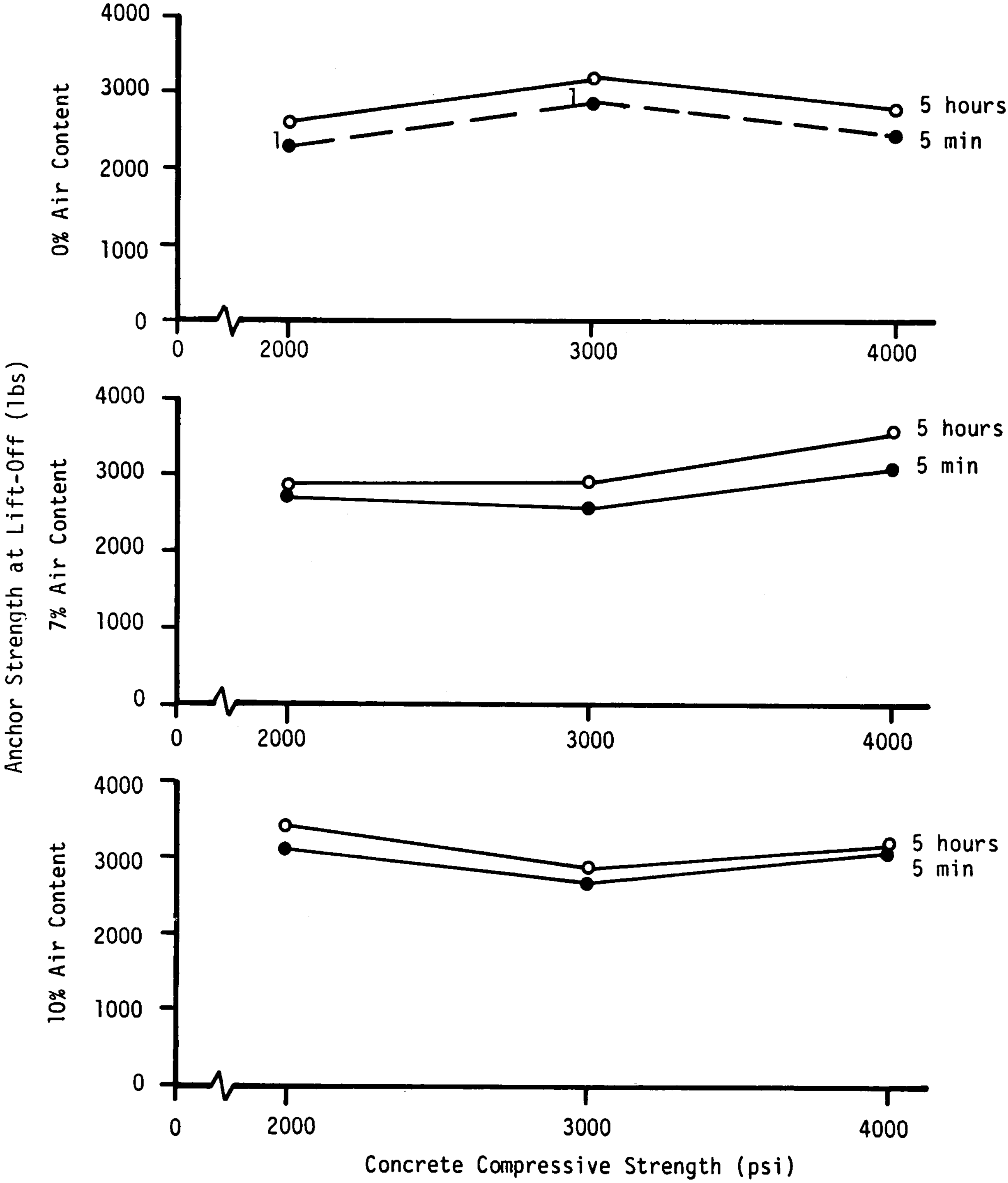

Figure 14. Comparison of anchor lift-off loads--Phase I.

[1]Assumed data point.

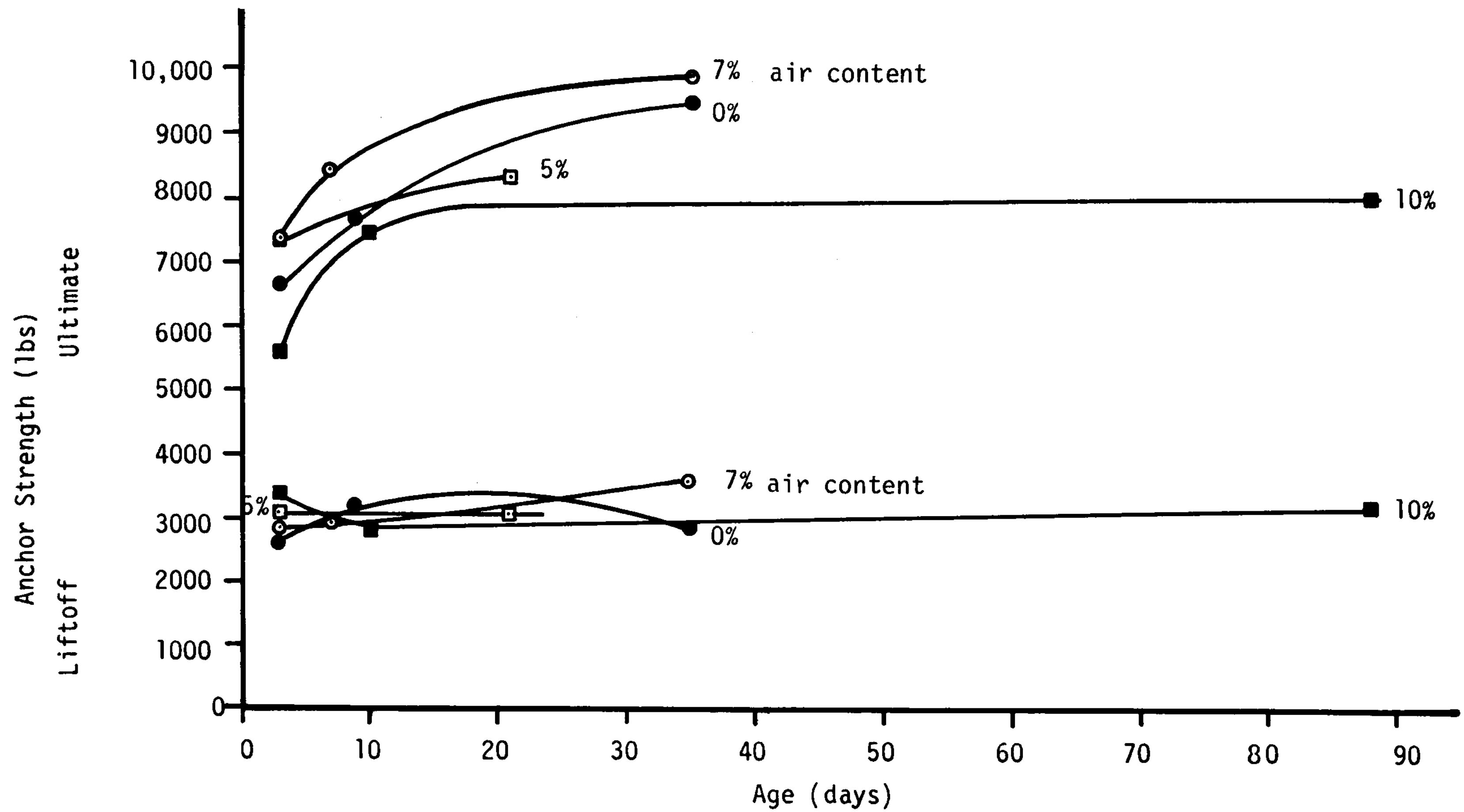

Figure 15. Effect of air content and age on anchor strength--Phase I.

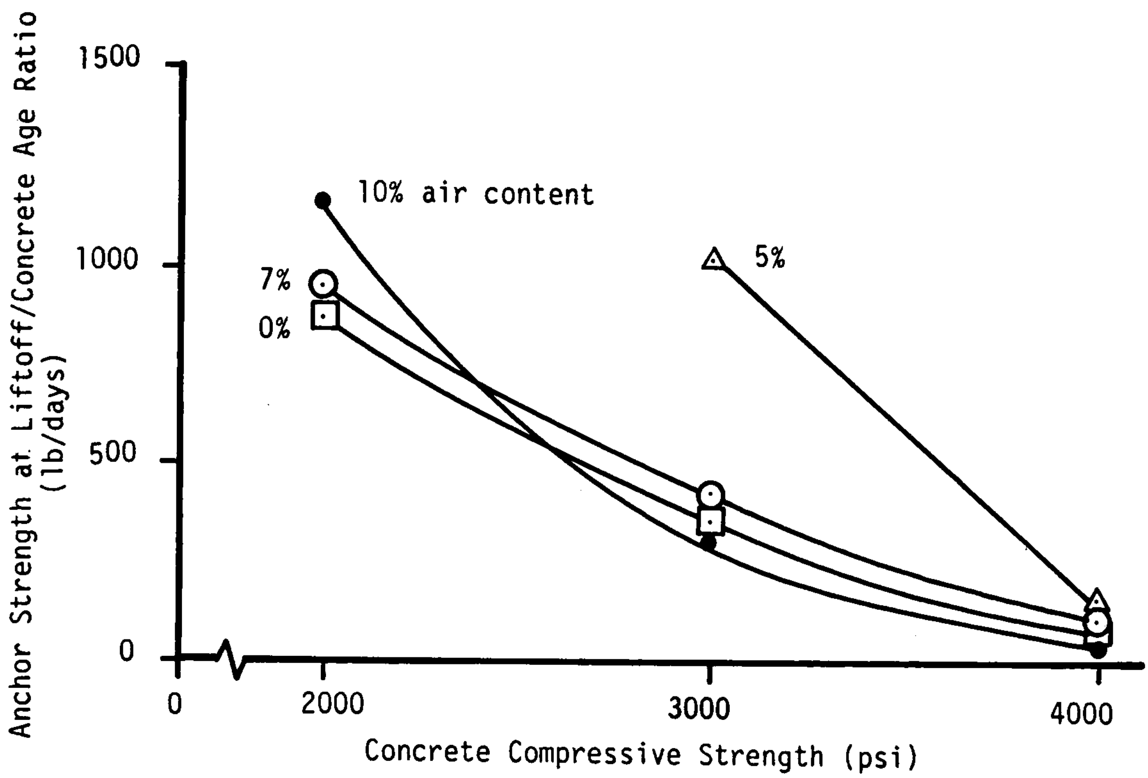

Figure 16. Effect of air content on anchor strength/concrete age ratio--Phase I.

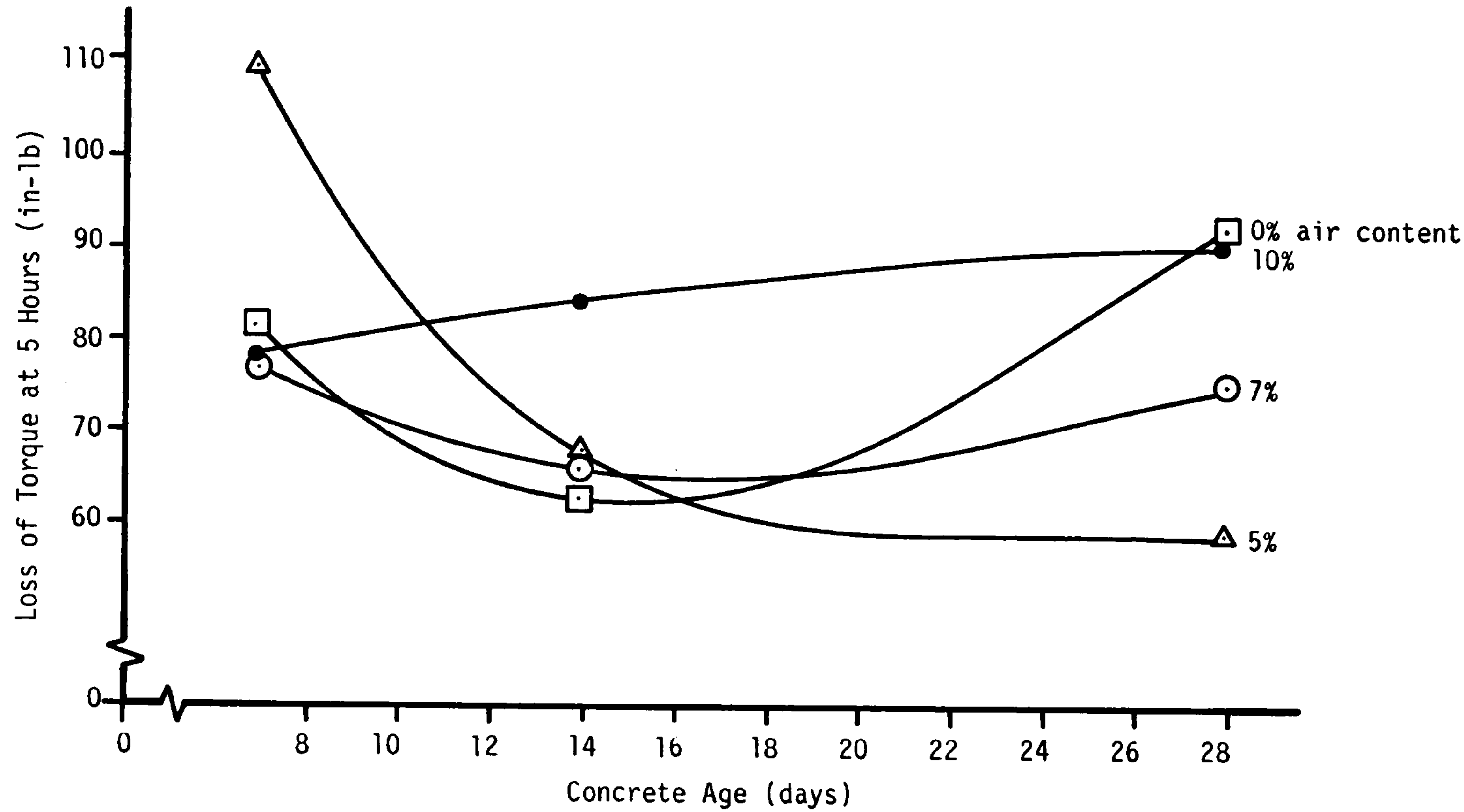

Figure 17. Effect of concrete age and air content on torque loss--Phase II.

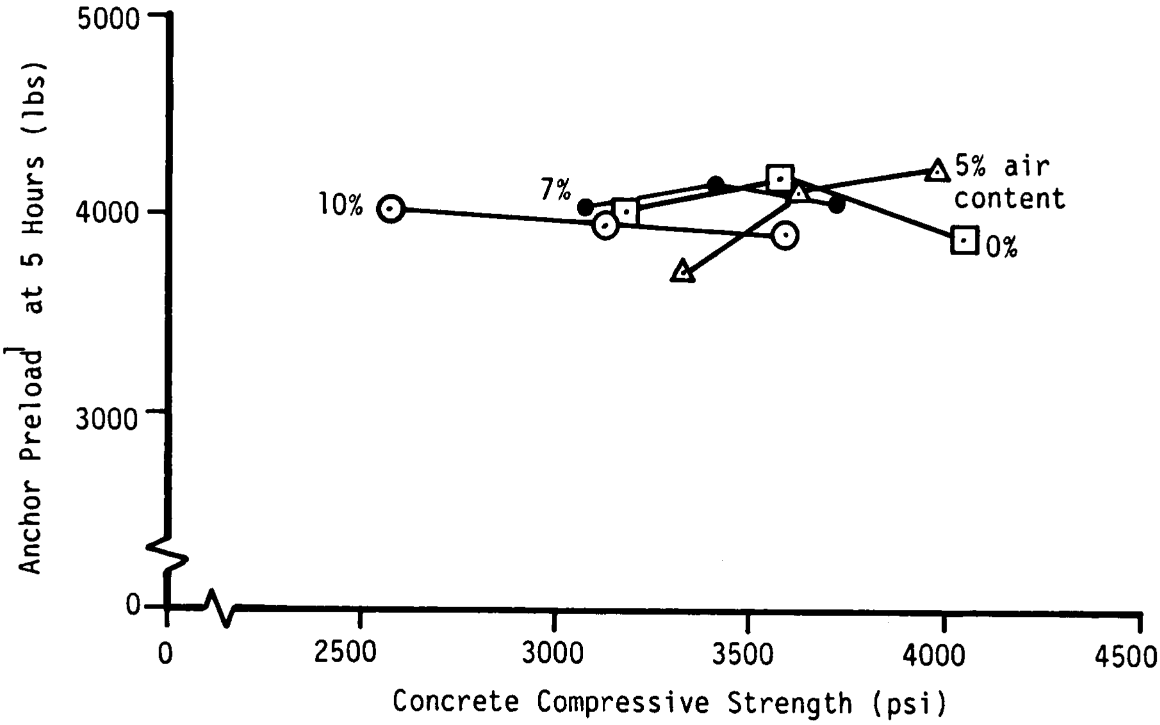

Figure 18. Changes in anchor preload for different levels of compressive strength and air content--Phase II.

[1]Loads calculated based on retorque values.

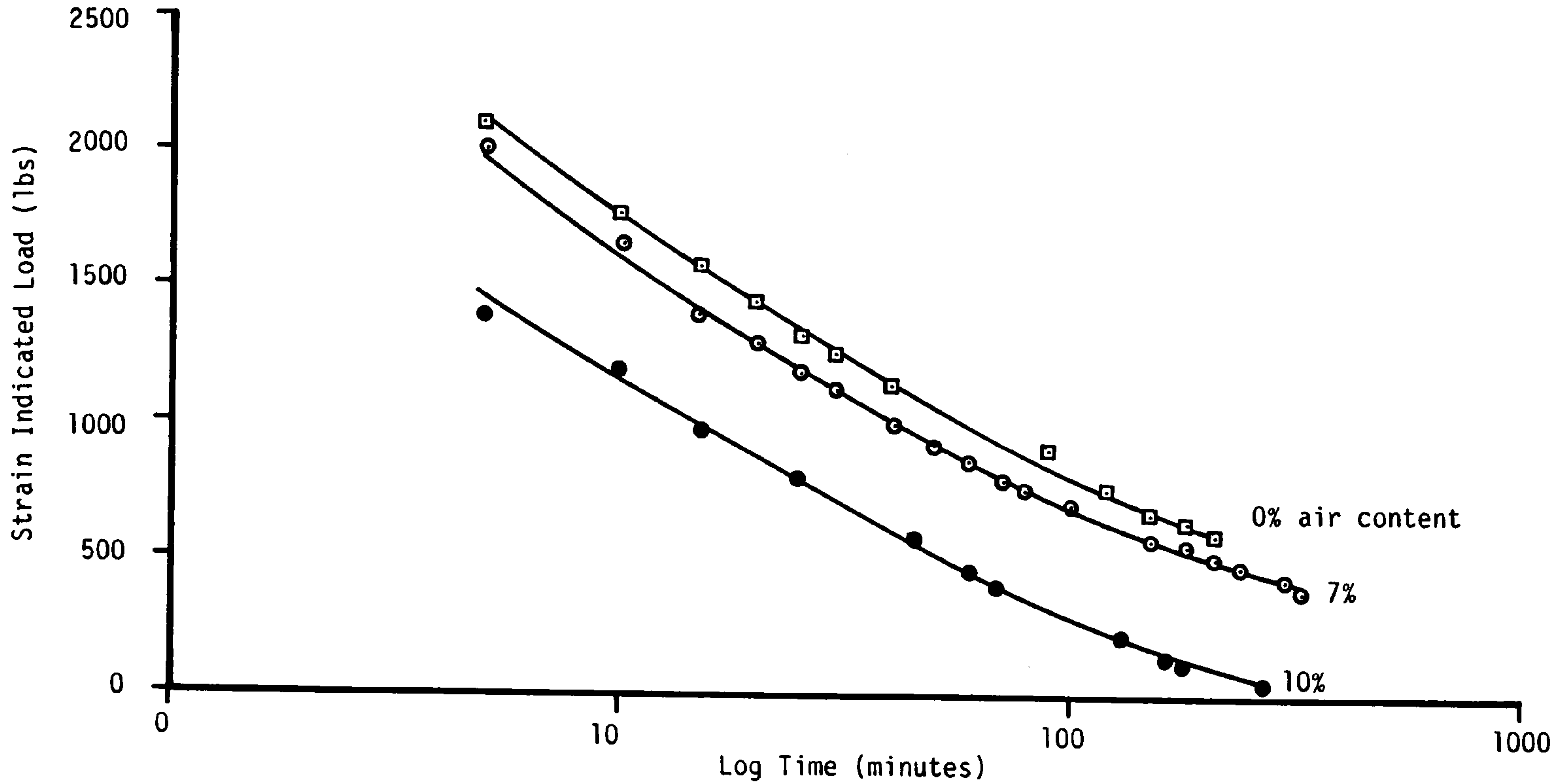

Figure 19. Corrected anchor load loss with time at 2000 psi--Phase I.

Slab Compressive Strength = 2000 psi

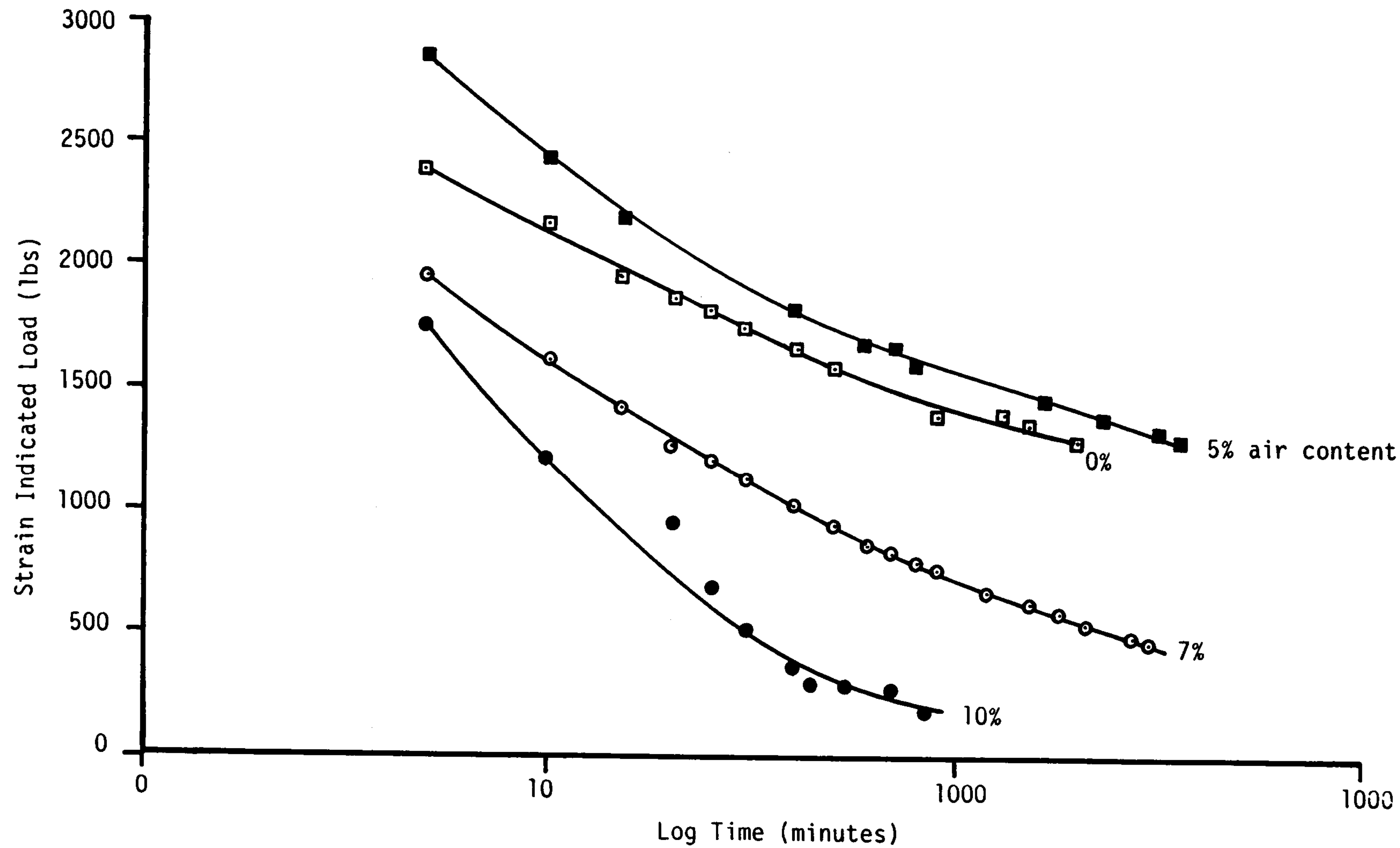

Figure 20. Corrected anchor load loss with time at 3000 psi--Phase I.

Slab Compressive Strength = 3000 psi

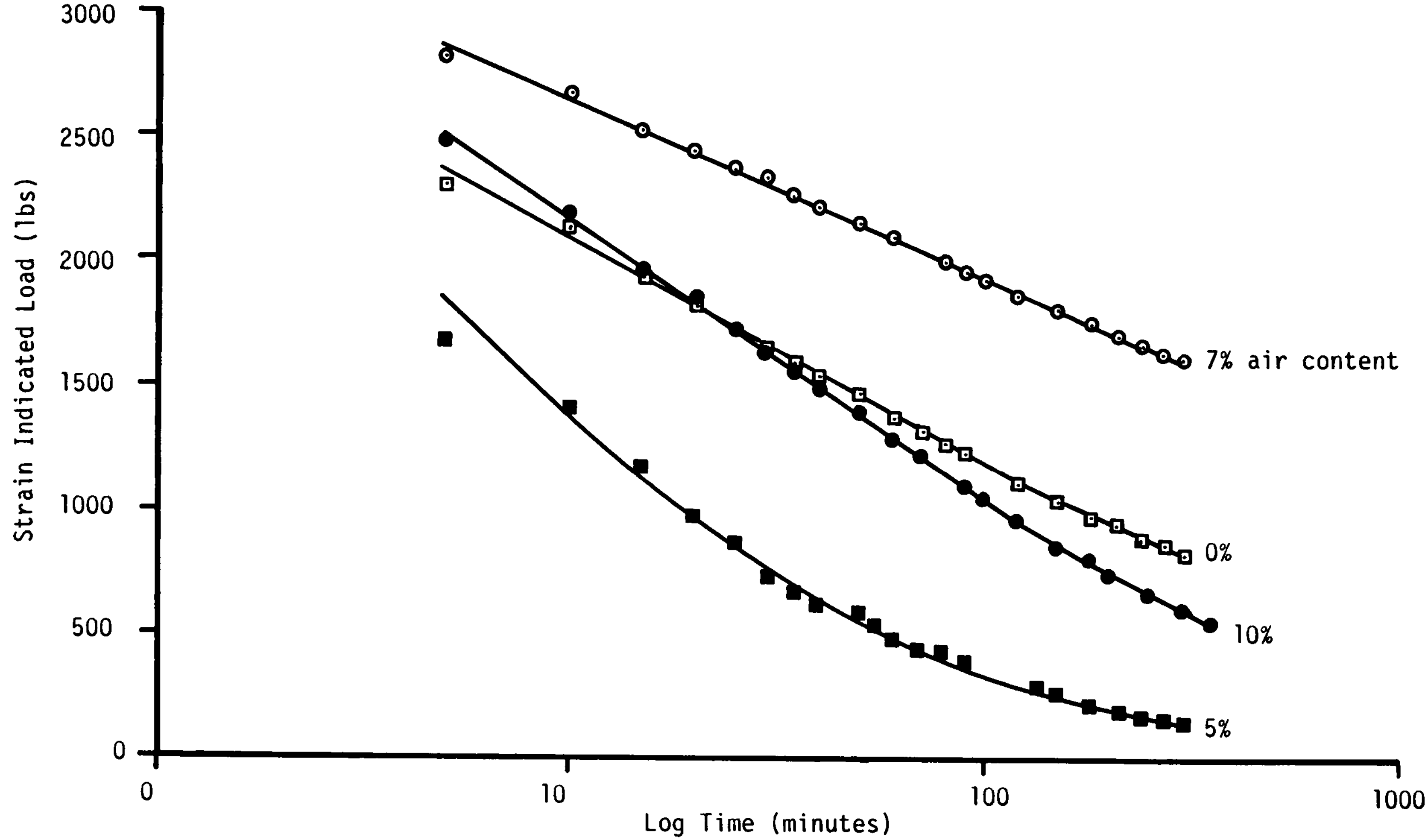

Figure 21. Corrected anchor load base with time at 4000 psi--Phase I.

Slab Compressive Strength = 4000 psi

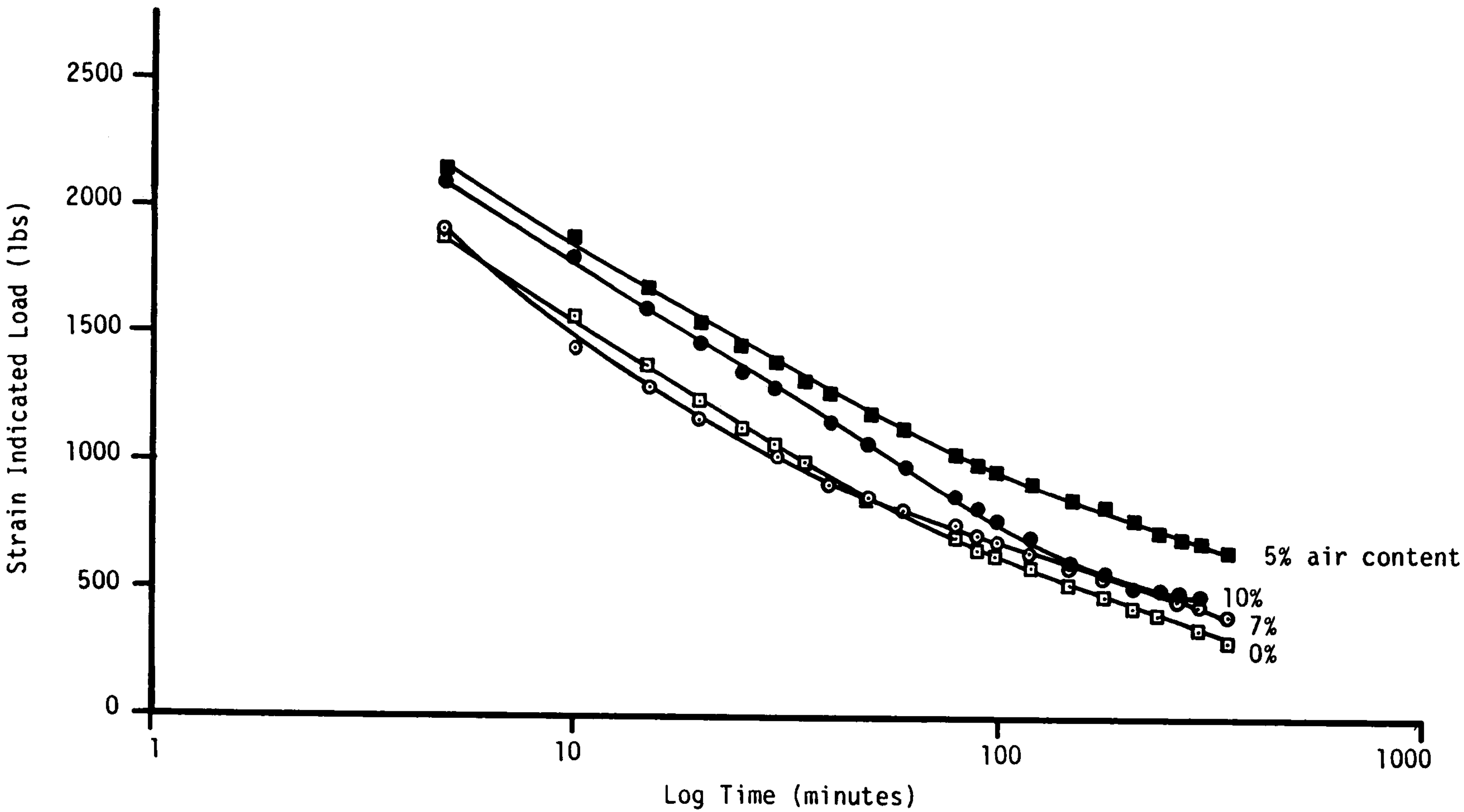

Figure 22. Corrected anchor load loss with time at seven days--Phase II.

Age of Test = 7 days

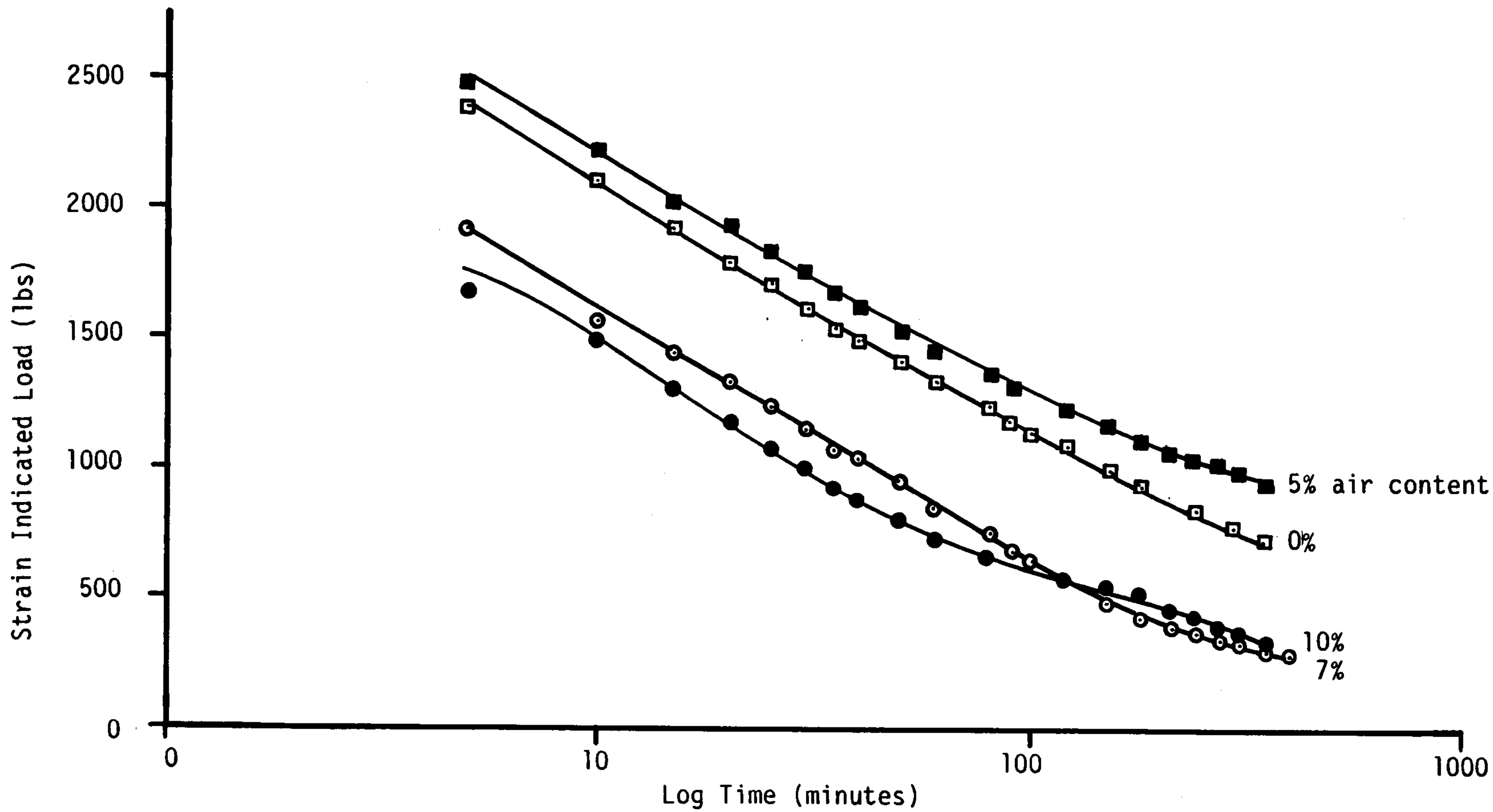

Figure 23. Corrected anchor load loss with time at 14 days--Phase II.

Age of Test = 14 days

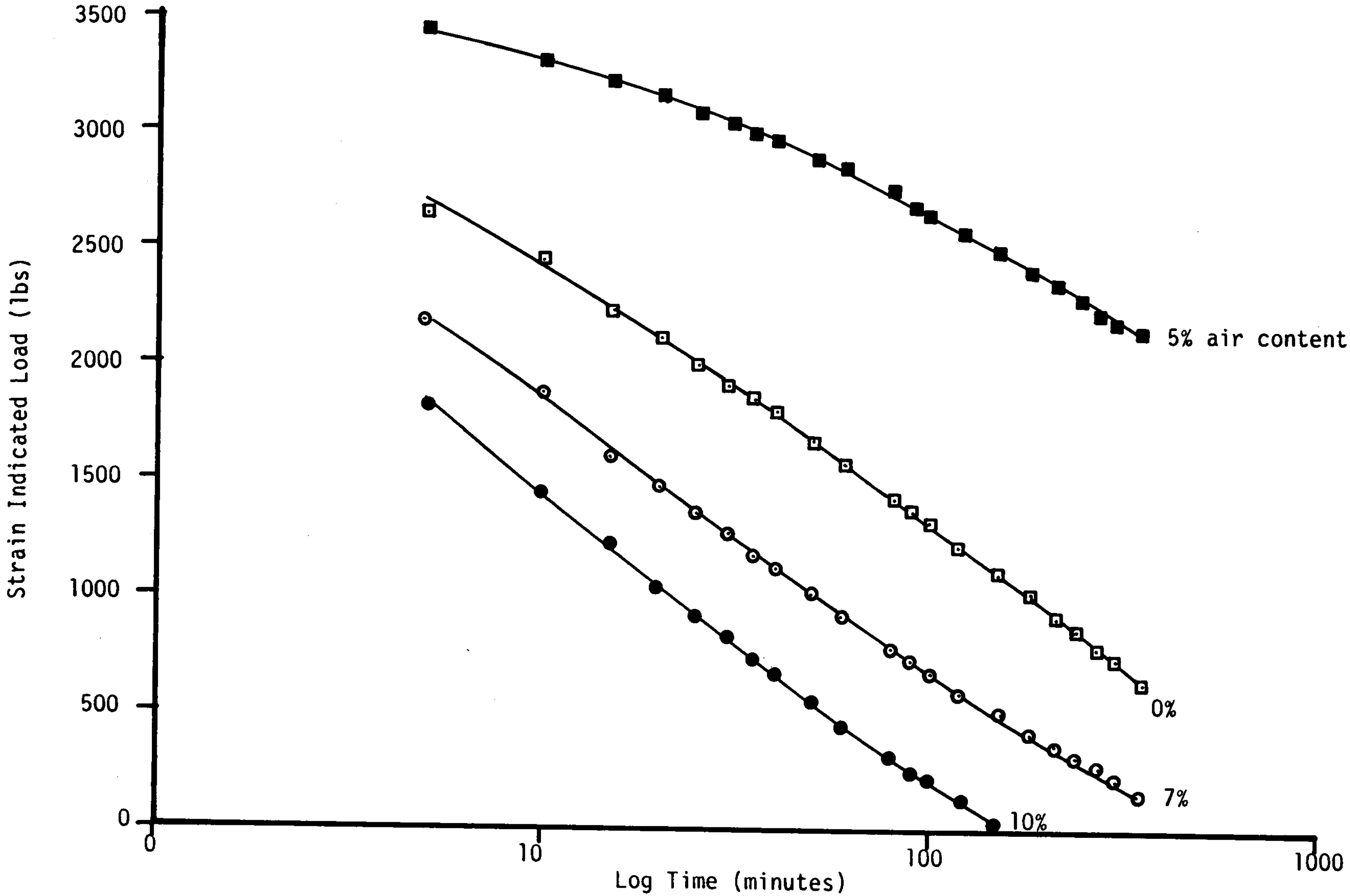

Figure 24. Corrected anchor load loss with time 28 days--Phase II.

Age of Test = 28 days

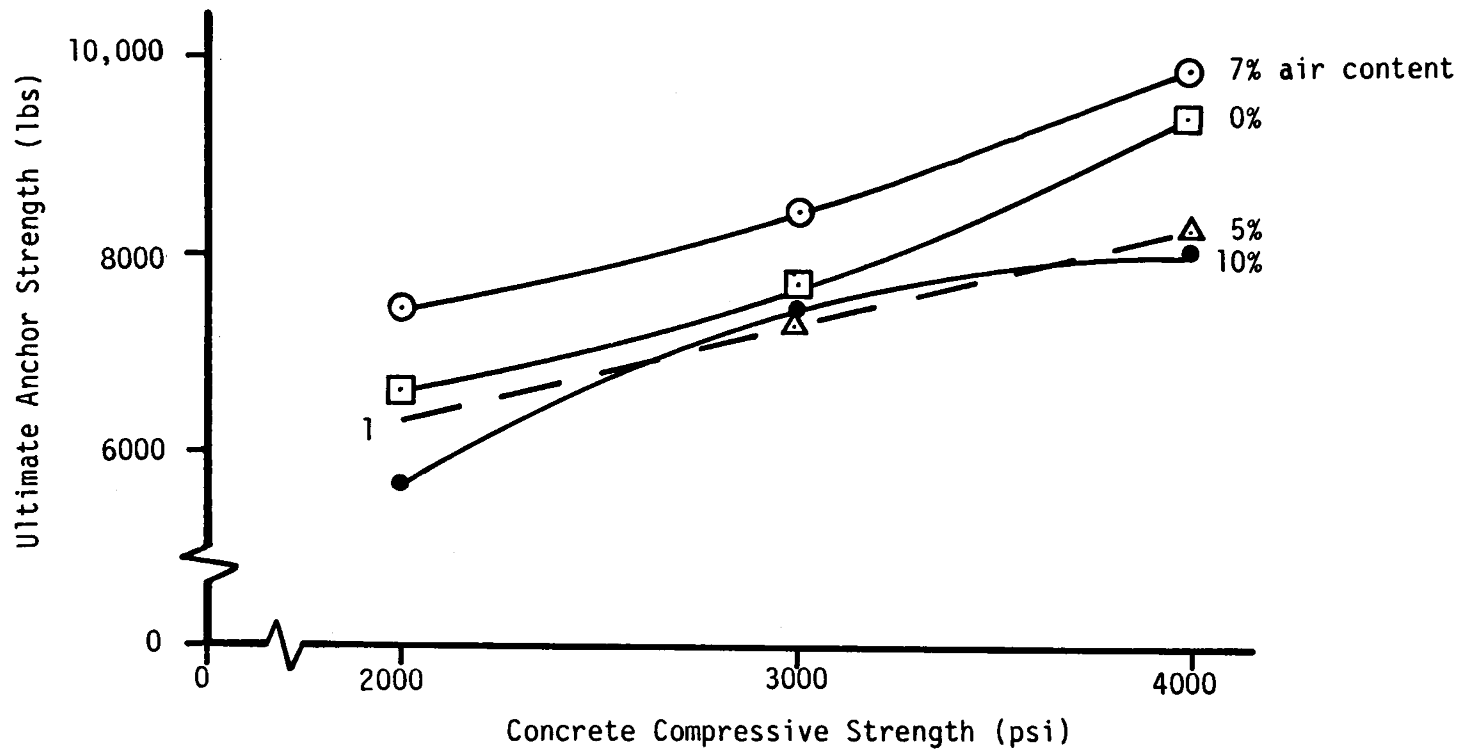

Figure 25. Ultimate anchor strength for varying levels of air content and concrete strength--Phase I.

[1] Assumed curve based on two points.

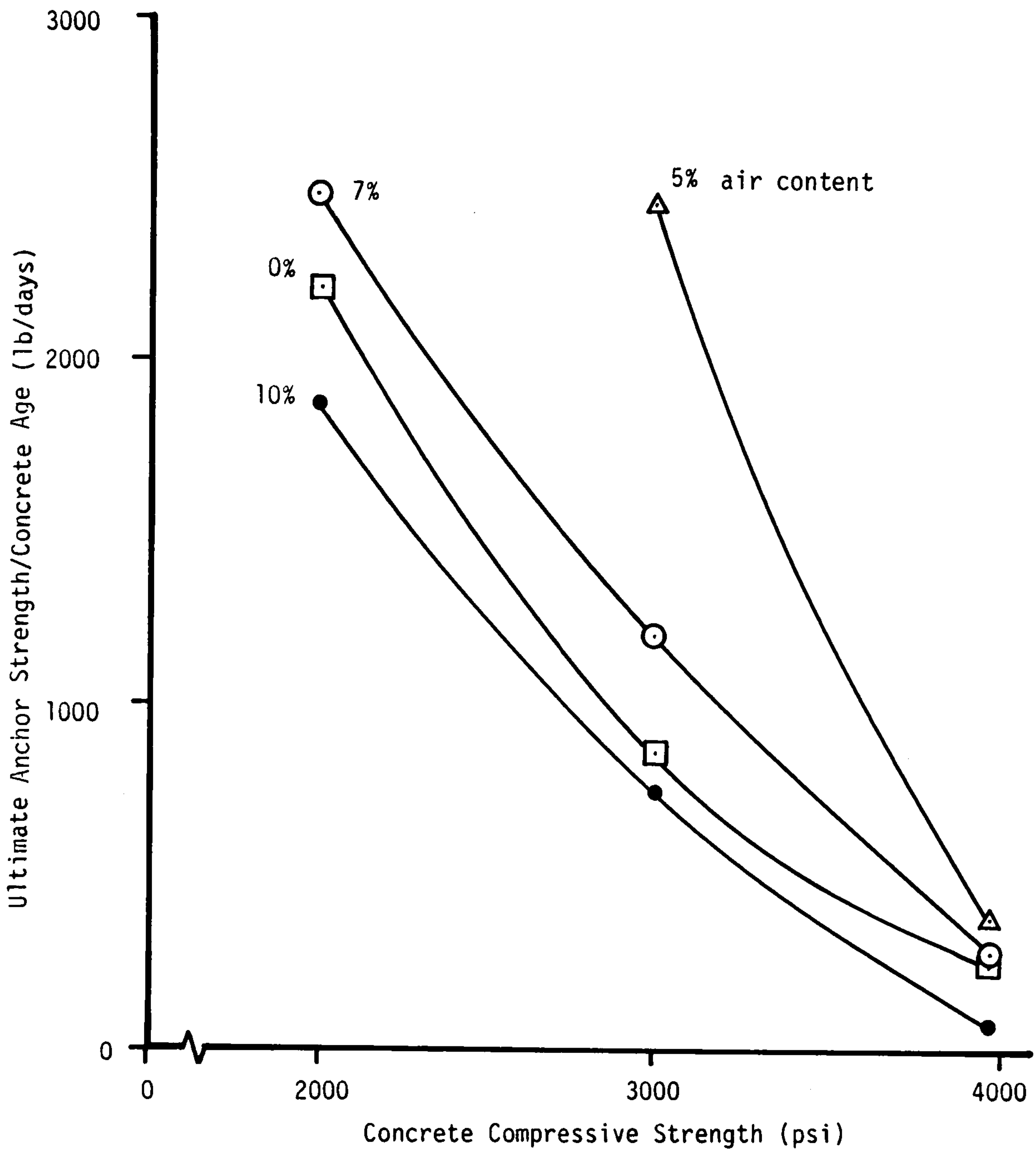

Figure 26. Effect of air content and compressive strength on ultimate anchor strength/concrete age ratio--Phase I.

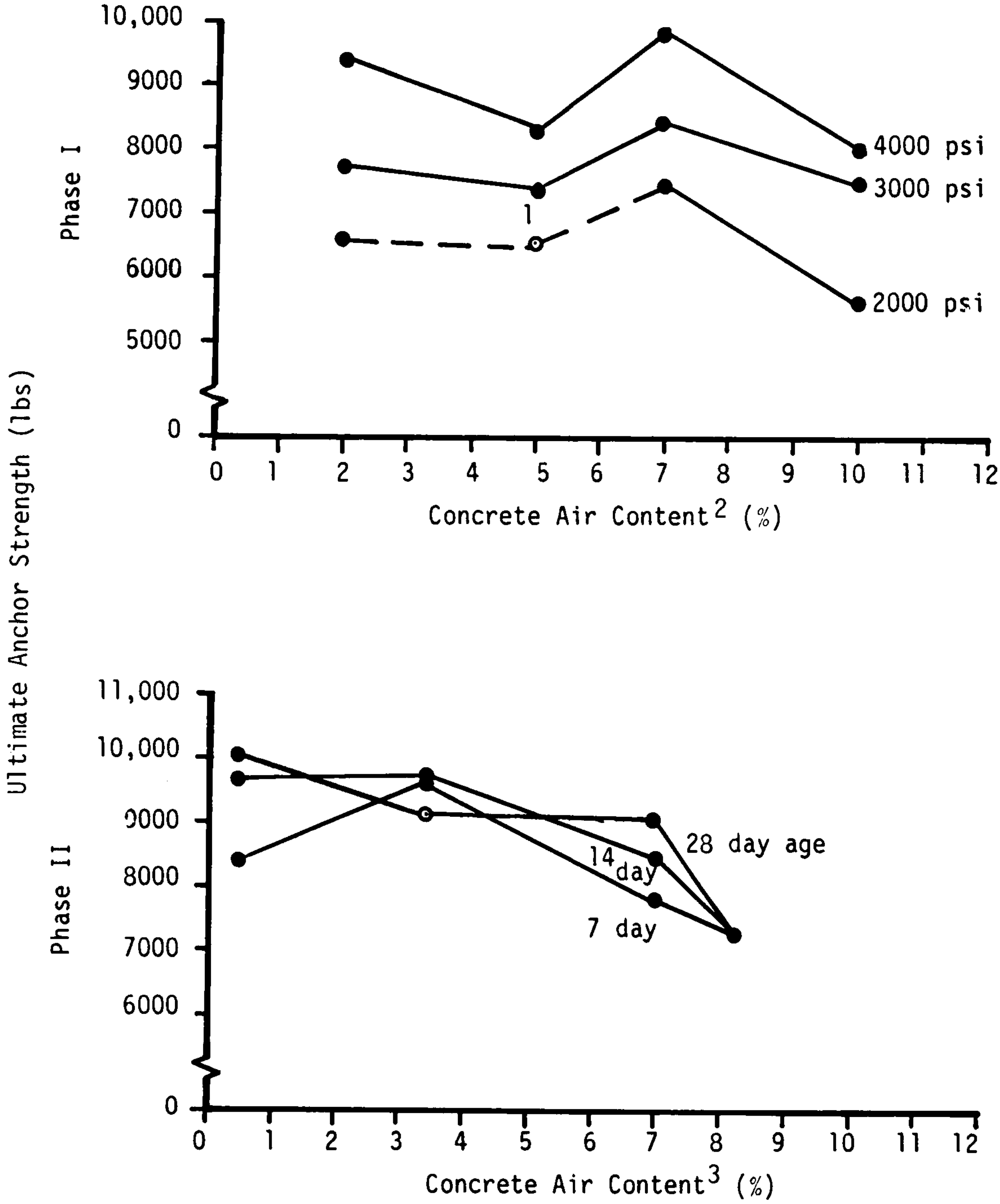

Figure 27. Effect of air content on ultimate anchor strength--Phase I and Phase II.

[1]Assumed data point.

[2]Design air contents.

[3]Hardened air contents.

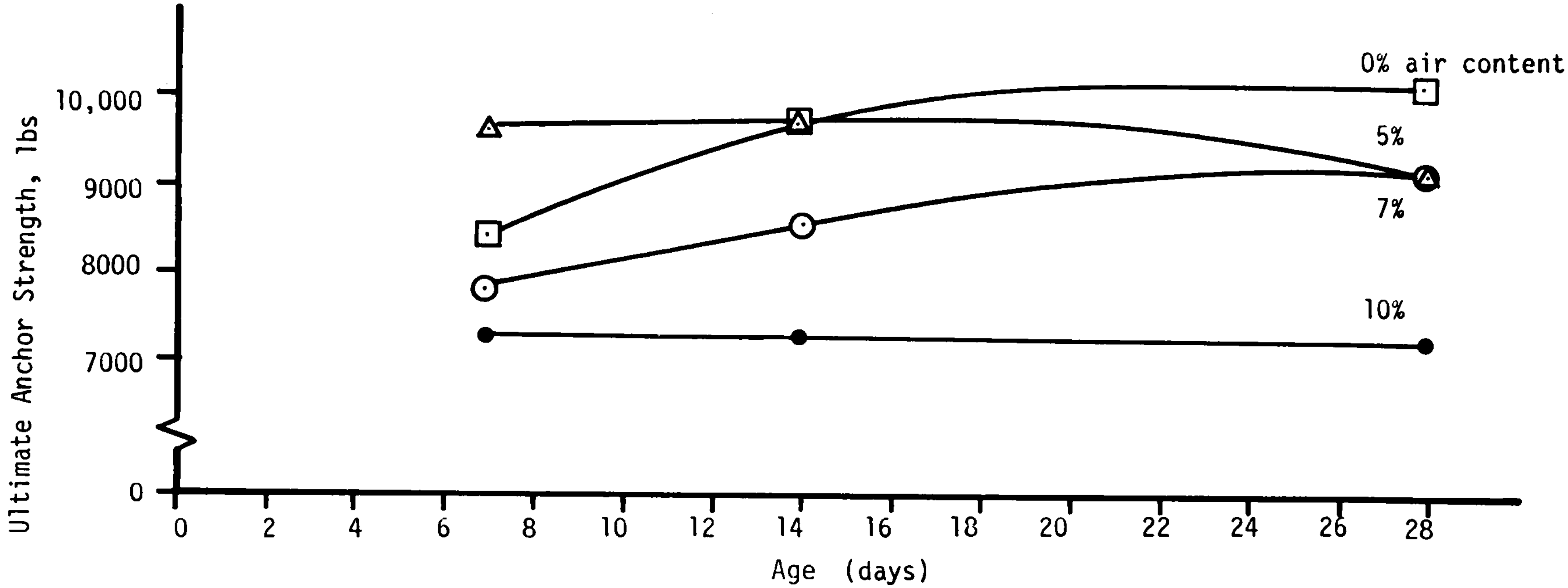

Figure 28. Effects of concrete age on anchor strength at various air contents--Phase II.

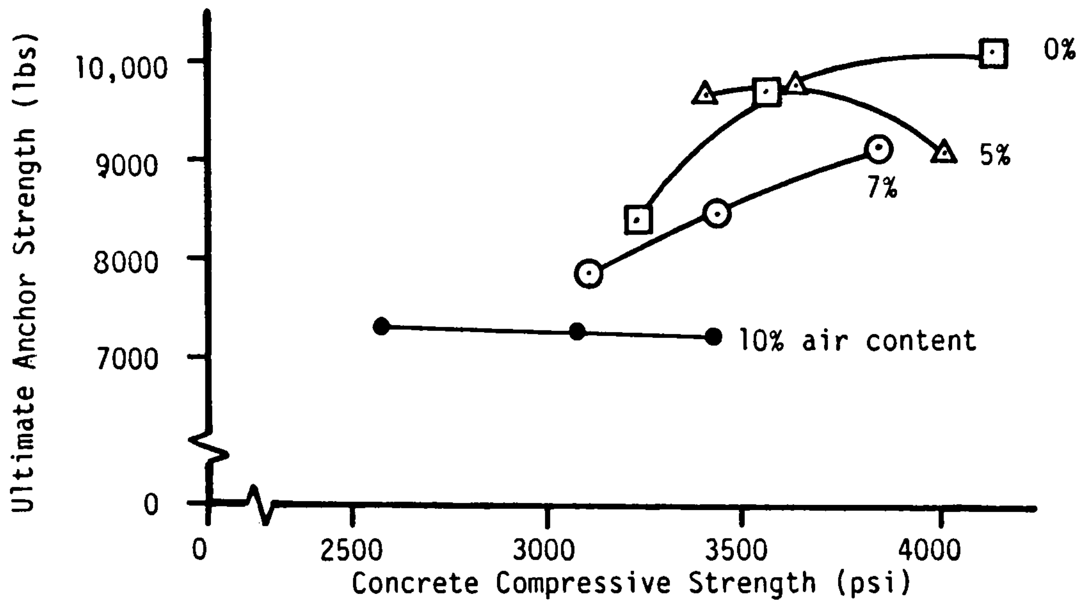

Figure 29. Ultimate anchor strength for varying levels of air content and concrete strength--Phase II.

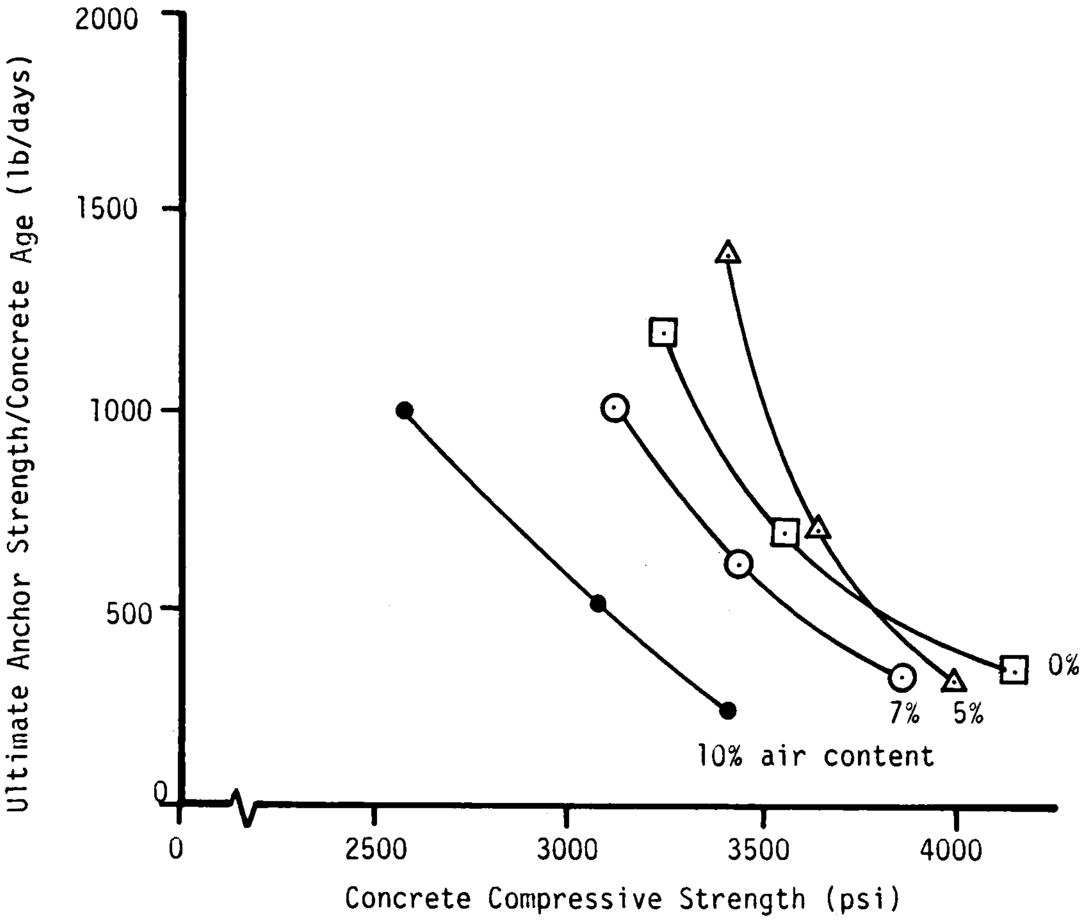

Figure 30. Effect of air content and compressive strength on ultimate anchor strength/concrete age ratio--Phase II.

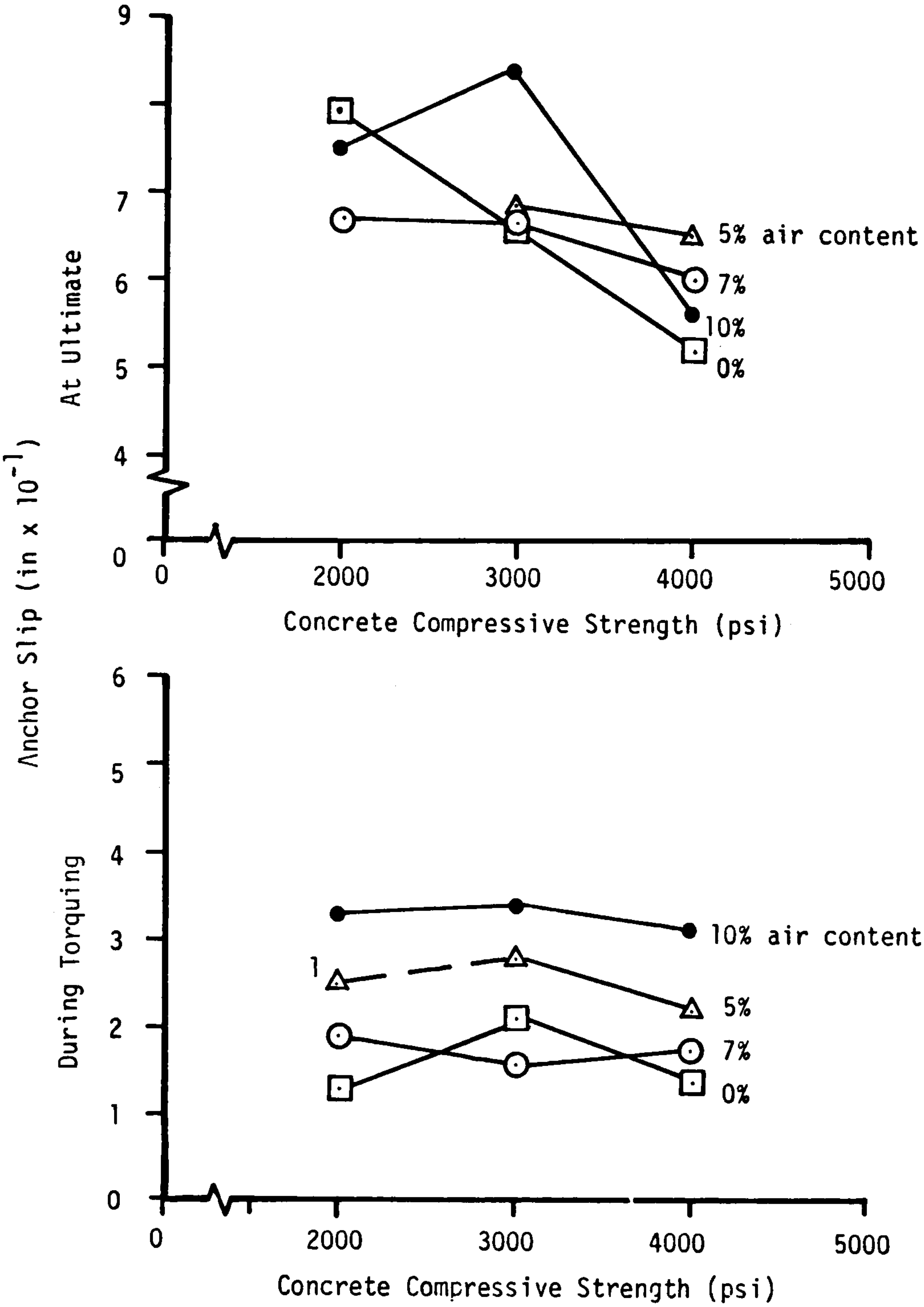

Figure 31. Effect of compressive strength and air content on anchor slip during torquing and at ultimate strength--Phase I.

[1]Assumed data point.

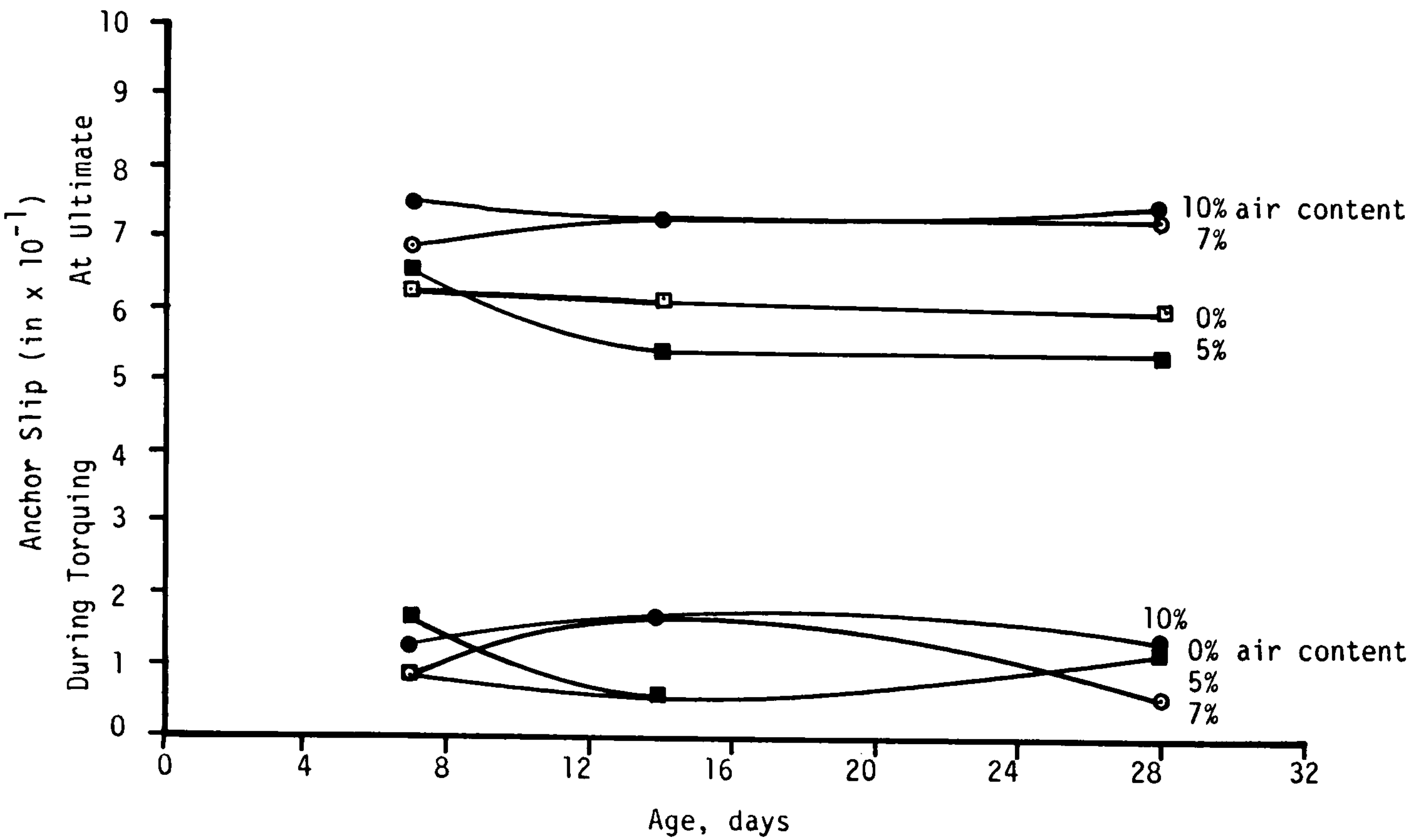

Figure 32. Effect of concrete age and air content on anchor slip during torquing and at ultimate strength--Phase II.

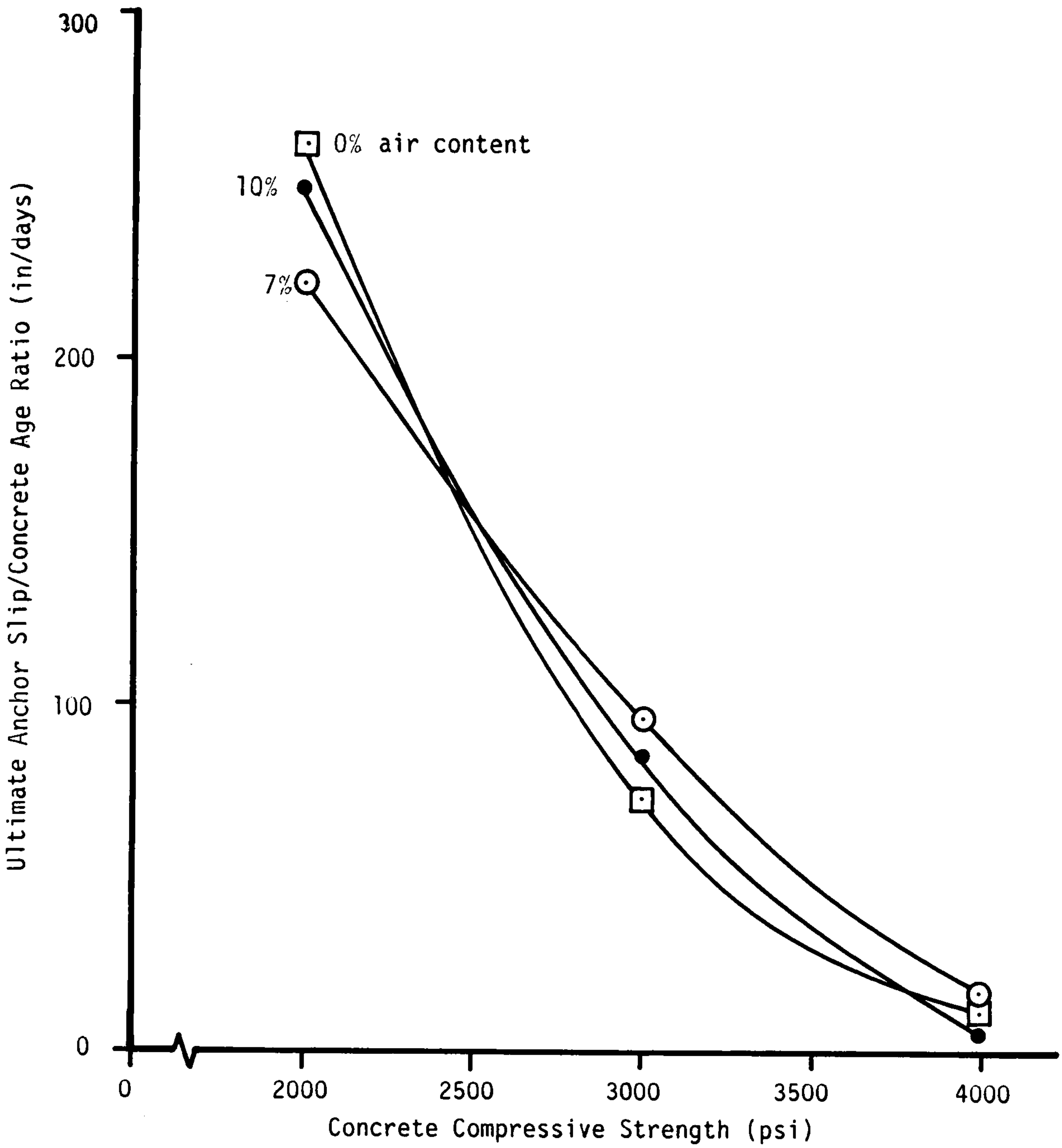

Figure 33. Effect of air content and compressive strength on ultimate anchor slip/concrete age ratio--Phase I.

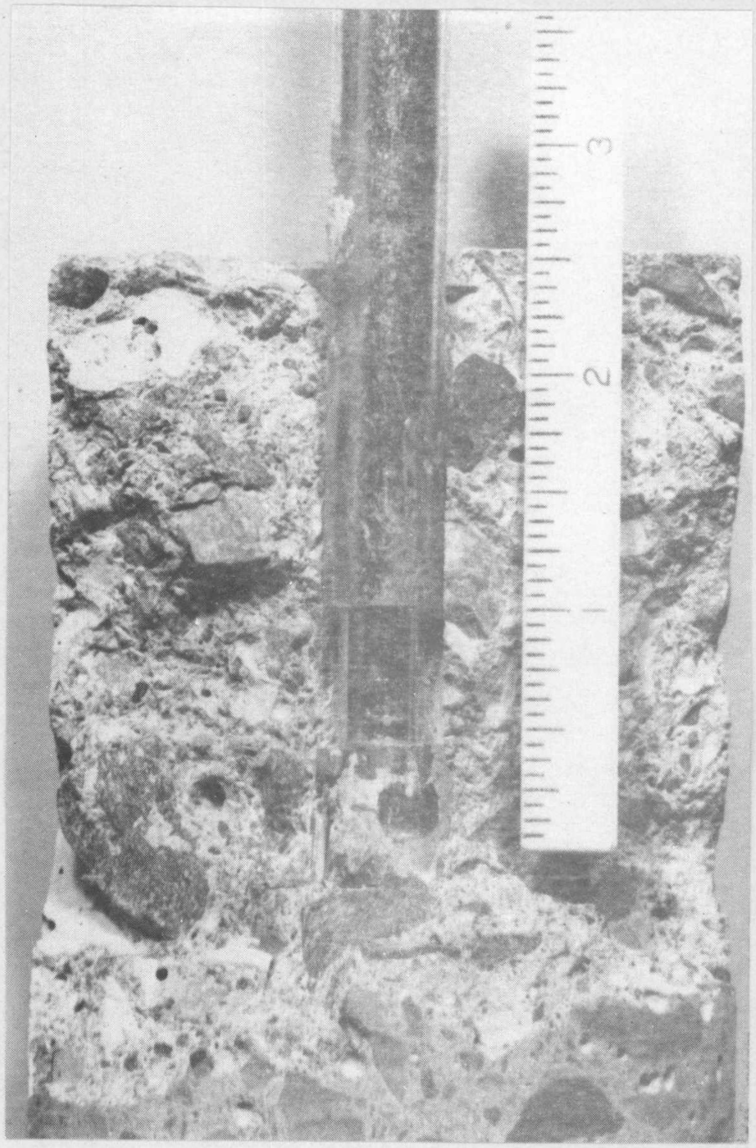

Figure 34. Closeup of typical wedge anchor approaching failure.

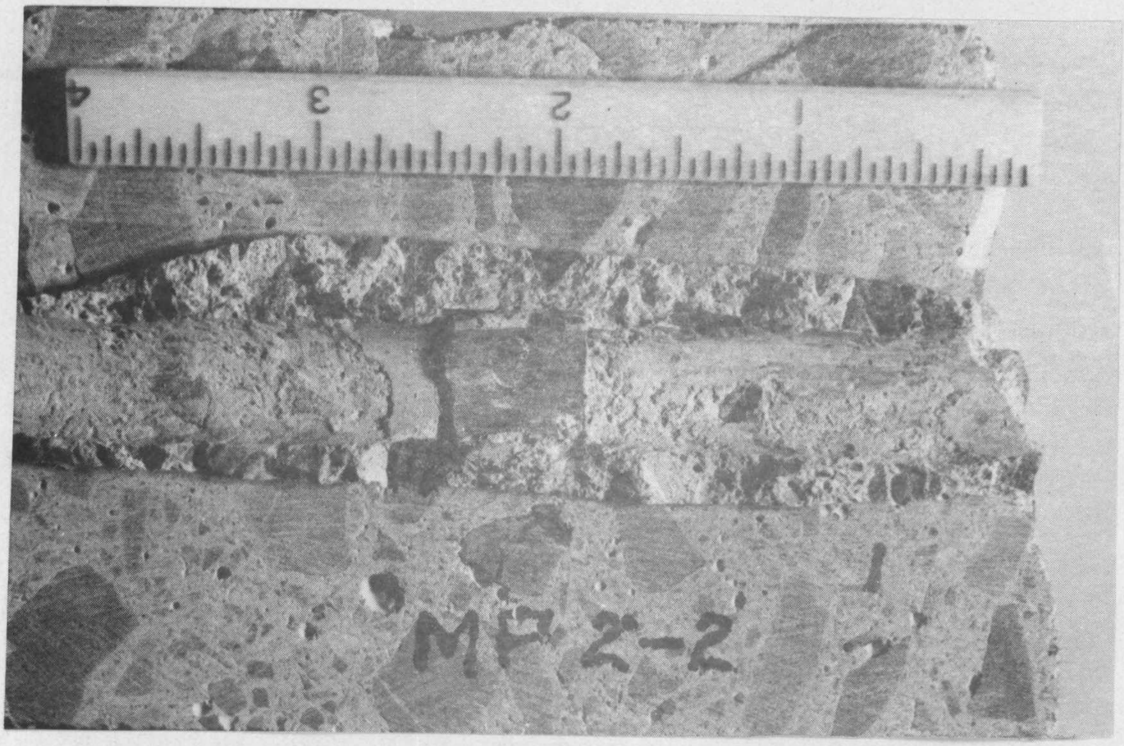

Figure 35. Closeup of 0% air content core containing wedge. Top of core is located to right--Phase I.

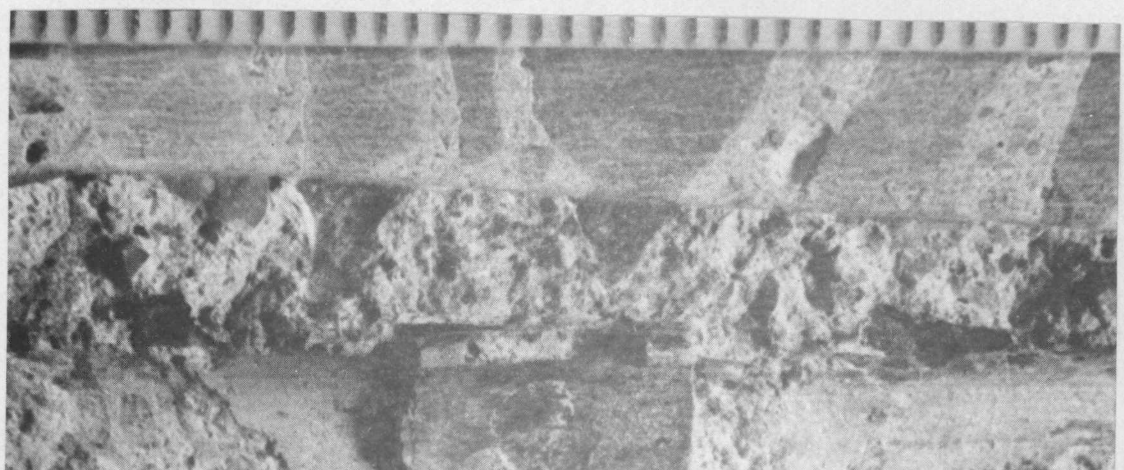

Figure 36. Extreme closeup of wedge in Figure 35.

Figure 37. Closeup of 10% air content core containing wedge. Top of core is located to right--Phase I.

Figure 38. Extreme closeup of wedge in Figure 37.

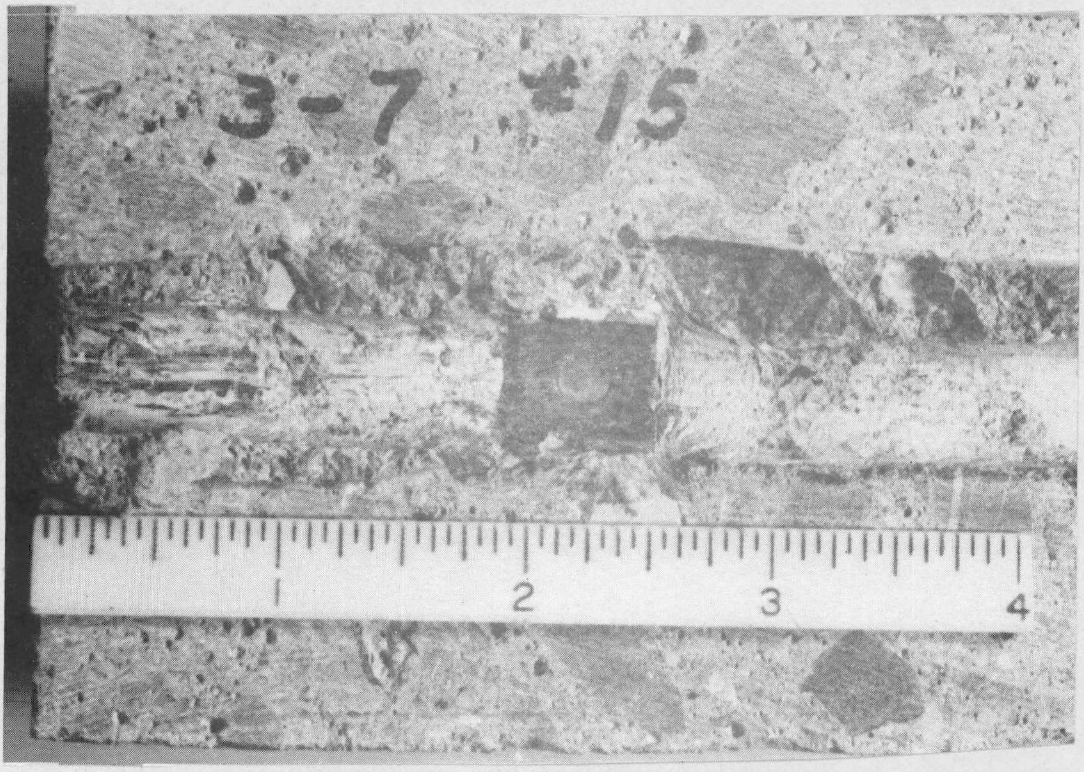

Figure 39. Closeup of 7% air content core containing wedge. Top of core is located to the left--Phase II.

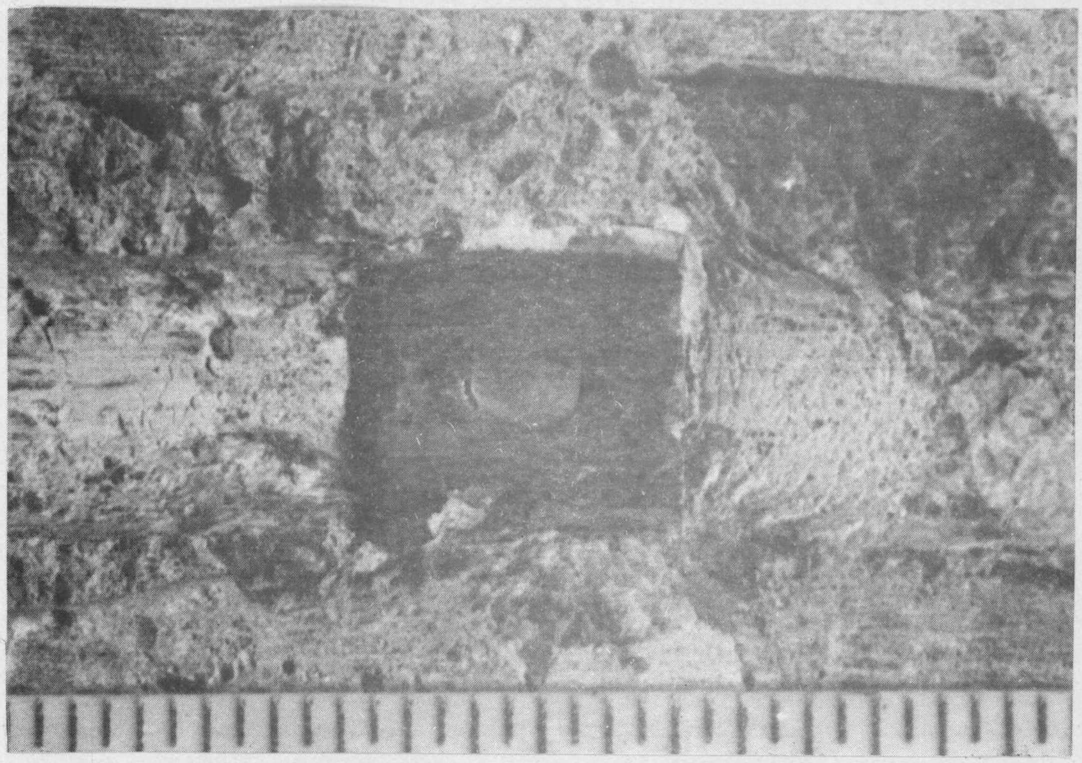

Figure 40. Extreme closeup of wedge in Figure 39.

Figure 41. Closeup of 10% air content core containing wedge. Top of core is located to the left--Phase II.

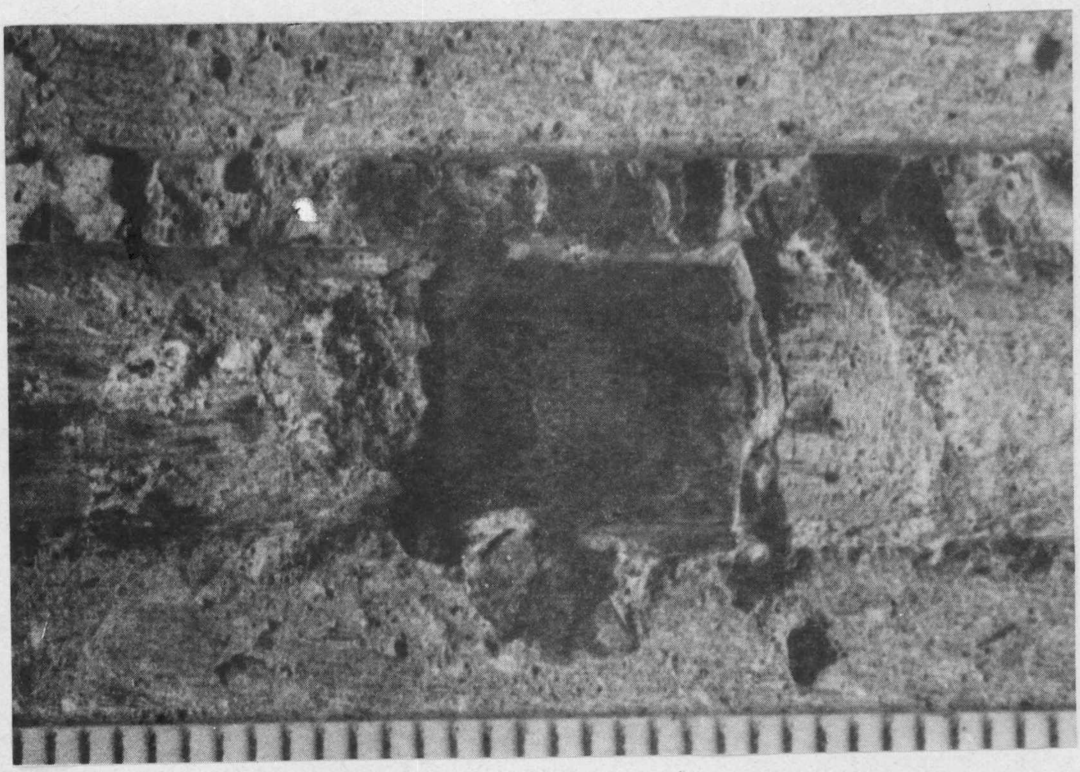

Figure 42. Extreme closeup of wedge in Figure 41.

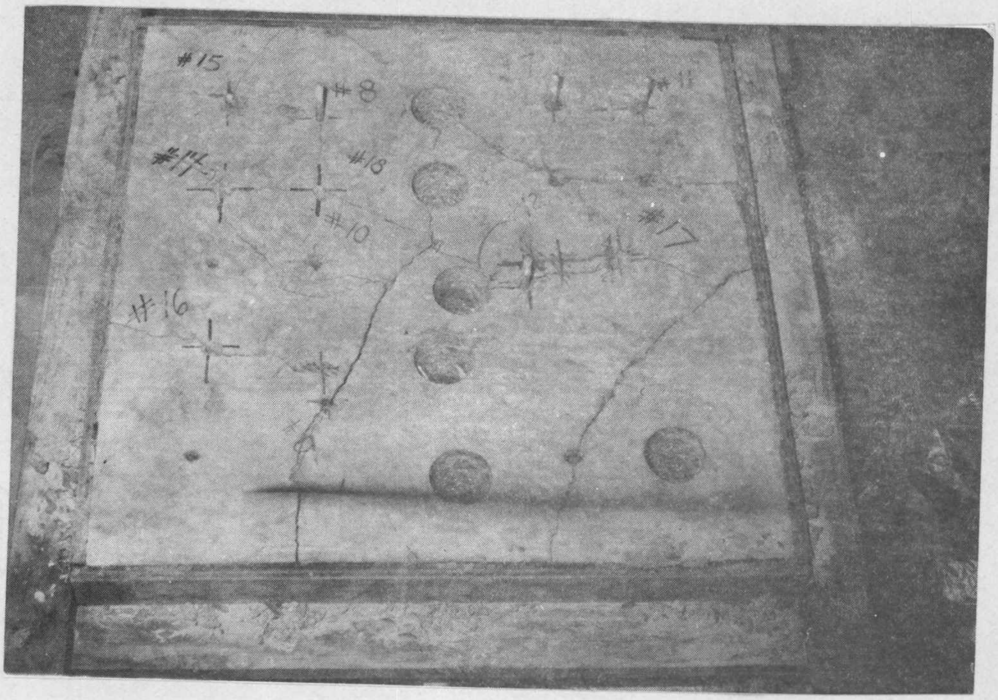

Figure 43. The 5% Phase II slab after conclusion of testing.

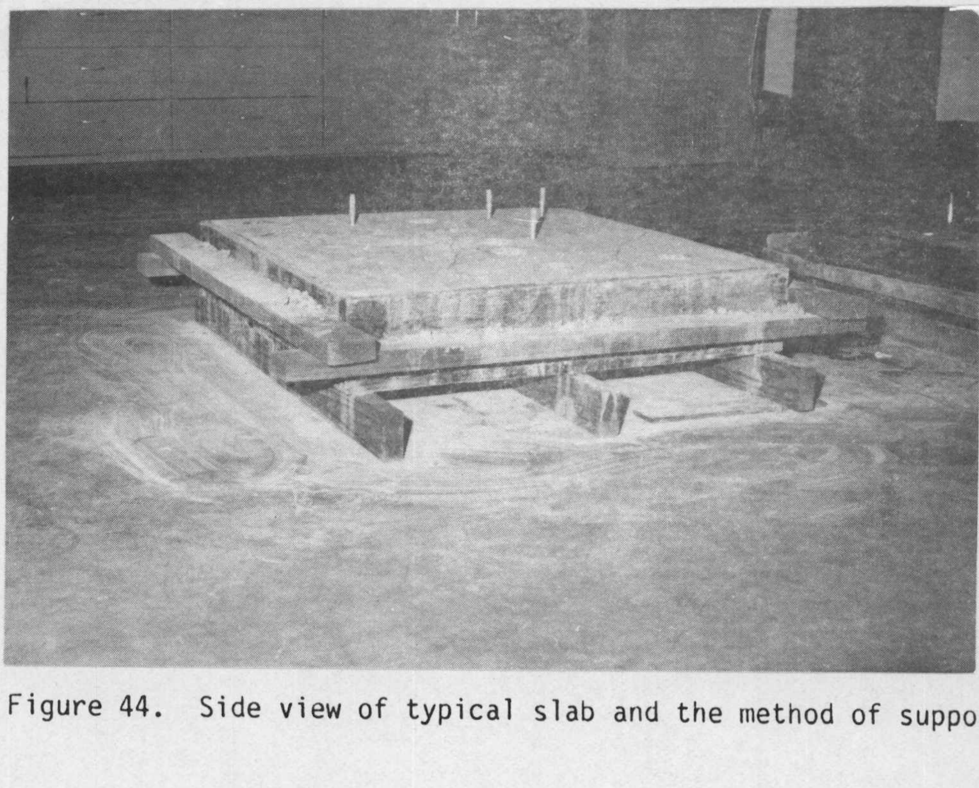

Figure 44. Side view of typical slab and the method of support.

APPENDIX B

METHOD OF DETERMINATION OF AIR CONTENT OF HARDENED CONCRETE

EXAMPLE

If a concrete mixture is designed to have a 5% air content, then:

1. Take the theoretical unit weight from the design mix, which for this case is:

Cement	-	534 pcy
Water	-	321 pcy
Sand	-	1572 pcy
Rock	-	1593 pcy
	Σ	4020 pcy

$$\frac{4020 \text{ pcy}}{27 \text{ cy}} = 148.9 \text{ pounds per cubic foot (pcf).}$$

This 148.9 pcf is equal to a volume of 0.95 cf solids + 0.05 cf air.

2. Thus to get the unit weight of a solid cubic foot:

$$\frac{148.9 \text{ pcf}}{0.95\ V_{solids}} = 156.7 \text{ lb solid air free unit weight.}$$

3. Then from laboratory tests the actual unit weight, based on the 3 inch by 6 inch cores, is used to determine the approximate hardened air content

$$\frac{151.4 \text{ pcf actual}}{156.7 \text{ pfc solid air free}} = 0.965 \text{ .}$$

4. $100\%[1.000 - 0.965] \cong 3.5\%$ air content.

APPENDIX C

LOAD CORRECTION CALCULATIONS

P_o = original preload	P_o = 5000 lb
P_t = load at some time t as indicated from strain meter	P_t = assume 4000 lb for example
P_A = load in anchor bolt only at some time t	
A_1 = area of strain bolt	$A_1 = 0.179$ in^2
A_2 = area of upper anchor	$A_2 = 0.196$ in^2
A_3 = area of wedge ramp	$A_3 = 0.105$ in^2
Δ_T = total elongation of strain bolt and anchor bolt	
Δ_{slip} = magnitude of slip	
$\Delta_{net} = \Delta_T -$ slip	
Δ_t' = anchor only	
L_1 = effective strain bolt length	L_1 = 6 in
L_2 = upper anchor length	L_2 = 4.5 in
L_3 = wedge ramp length	L_3 = 0.5 in
P_1 = load in strain bolt	
P_2 = load in anchor body	
P_3 = load in wedge ramp	
E_s = modulus of elasticity	$E_s = 29 \times 10^6$ psi

<u>Relationships</u>

1. $P_1 = P_2 = P_3$ at any time

2. $S = \frac{P}{A}$; $\varepsilon = \frac{\Delta}{L}$; $E = \frac{S}{\varepsilon}$

$$S = \varepsilon E$$

$$S = \frac{\Delta}{L} E$$

$$\frac{P}{A} = \frac{\Delta}{L} E$$

$$\Delta = \frac{PL}{EA}$$

3. $\Delta_T = \Delta_1 + \Delta_2 + \Delta_3$ and $\Delta_T' = \Delta_2 + \Delta_3$

then:

$$\Delta_{t\ initial} = \frac{PL_1}{EA_1} + \frac{PL_2}{EA_2} + \frac{PL_3}{EA_3} = \frac{P}{E}\left(\frac{L_1}{A_1} + \frac{L_2}{A_2} + \frac{L_3}{A_3}\right)$$

$$\Delta_{slip} = \Delta_{t\ initial} - \Delta_{t\ final}$$

$$= \frac{P_o}{E}\left(\frac{L_1}{A_1} + \frac{L_2}{A_2} + \frac{L_3}{A_3}\right) - \frac{P_t}{E}\left(\frac{L_1}{A_1} + \frac{L_2}{A_2} + \frac{L_3}{A_3}\right)$$

$$\left[\Delta_{slip} = \frac{\frac{L_1}{A_1} + \frac{L_2}{A_2} + \frac{L_3}{A_3}}{E_s}(P_o - P_t)\right]$$

4. $P = \frac{\Delta_{net}\ E_s\ A}{L} = \frac{\Delta_{net}\ E_s}{L/A'}$, thus assuming the slip is the same for the anchor alone:

$$\Delta_t' = \frac{P_o}{E_s}\left(\frac{L_2}{A_1} + \frac{L_3}{A_3}\right)$$

$$\left[P_A = \frac{(\Delta_t' - \Delta_{slip})E_s}{\left(\frac{\ell_2}{A_2} + \frac{\ell_3}{A_3}\right)}\right]$$

Example:

$$\Delta_{slip} = \frac{\left(\frac{6"}{0.179} + \frac{4.5"}{0.196} + \frac{0.5"}{0.105}\right)}{E_s} (5000 - 4000)$$

$$= 2.11 \times 10^{-3}$$

$$\Delta_t' = \frac{P_o}{E_s} \left(\frac{L_2}{A_2} + \frac{L_3}{A_3}\right)$$

$$= \frac{5000}{E_s} \left(\frac{4.5}{0.196} + \frac{0.5}{0.105}\right)$$

$$= 4.77 \times 10^{-3}$$

and

$$P_A = \frac{(4.77 \times 10^{-3} - 2.11 \times 10^{-3})\ 29 \times 10^6}{(22.92 + 4.78)}$$

= 2,787 lb load actual in the anchor.

VITA

Rex David Rowell was born in Lansing, Michigan, on July 27, 1946. He attended elementary schools in St. Johns, Michigan, and was graduated from Rodney B. Wilson High School in June 1964.

After a four-year tour of duty with the United States Navy, he entered Lansing Community College, Lansing, Michigan, in 1970. From there he transferred to Michigan State University and in March 1974 he received a Bachelor of Science degree in Civil Engineering. In March of that same year he accepted a position with the Tennessee Valley Authority and began work as a civil engineer.

He entered the Graduate School of The University of Tennessee, Knoxville, in January 1976 on a part-time basis while continuing to work for TVA. He received the Master of Science degree with a major in Civil Engineering in December 1980.

The author is a member of the American Concrete Institute, and the American Society of Civil Engineers.

He is married to the former Bernice Esch of Lansing, Michigan.