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The Civil Engineering Laboratory has conducted model anchor tests, intermediate scale field tests, laboratory soil investigations, and finite element analyses to optimize procedures for estimating long-term holding capacity of statically loaded plate-like anchors embedded in cohesive seafloor soils. Also, work was done to determine relationships between long- and short-term holding capacity. This report presents the results of these tests and analyses. It is concluded that the long-term holding capacity in cohesive soils

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INTRODUCTION

Direct embedment anchors are being used more frequently in ocean construction. Their selection stems from three major advantages over conventional anchors: (1) they can efficiently resist loads in any direction, including uplift loads, (2) they can be placed at specific points, and (3) they offer a high holding capacity relative to their weight. The major disadvantages of direct embedment anchors is that they offer reduced residual holding capacity after the ultimate load has been reached. They are extracted from the seafloor by the ultimate loads. As a consequence, direct embedment anchors must be carefully designed to prevent extraction. The design of direct embedment anchorages can be separated into three primary capacity considerations: (1) short-term static, (2) long-term static, and (3) dynamic loading.

This report presents the results of physical and mathematical modeling of long-term anchor behavior and a procedure for predicting long-term static holding capacity. Also presented are the results of investigations of short-term holding capacity affecting the selection of long-term design loads.

Background

The equation used to predict anchor holding capacity (Vesic, 1969) and patterned after bearing capacity equations for footings is as follows:

$$F = A\,(c\,\overline{N}_c + \gamma_b\,D\,\overline{N}_q)\,(0.84 + 0.16\,B/L) \qquad (1)$$

where F = holding capacity

A = fluke area

c = soil cohesion

γ_b = soil buoyant unit weight

D = fluke embedment depth

B = fluke width

L = fluke length

$\overline{N}_c, \overline{N}_q$ = holding capacity factors

This is a general equation that can be applied to deep and shallow anchor embedment and short- and long-term loading. Research on holding capacity of anchors has usually centered on determining suitable values for the breakout factors $\overline{N}_c$ and $\overline{N}_q$.

"Deep anchor embedment" defines a situation in which the sediment surface is not affected when the anchor is loaded to failure. As the anchor is displaced the soil tends to flow from above to below the anchor. "Shallow anchor embedment" defines a situation in which the soil surface is bulged when the anchor is loaded to failure. As the anchor is displaced a soil plug over the anchor is pushed out of the sediment. A term called the "relative depth of embedment," (D/B) is used to help define shallow and deep anchor behavior. This term is a function of soil type and strength and determines which of these two modes of failure will govern when an anchor is extracted.

Short-term capacity is the pullout load when an anchor is loaded rapidly to failure. In fine-grained soils water flow into or out of the soil does not occur, excess pore water pressures exist, and the soil's undrained shearing strength, s_u, governs the capacity. Long-term capacity is the largest pullout load an anchor fluke can sustain at a condition where excess pore water pressures do not exist. For fine-grained soil, an extended time period is required to attain this condition. Because induced water flow through the soil is complete, the soil's drained strength properties govern the capacity For coarse-grained soil this condition is reached almost immediately, and no distinction is made between short- and long-term capacity.

Long-term capacity is, therefore, a design consideration primarily in fine-grained cohesive soil. The design goal is to avoid situations where a safe short-term load will cause anchor pullout after extended time periods. The short-term capacity in cohesive soil has received considerable attention (Adams and Hayes, 1967; Ali, 1968; Bhatnagar, 1969; Vesic, 1969; Kupferman, 1971; Bemben and Kupferman, 1974). When analysis based on the work of these researchers is combined with the work of research on cohesionless soils where drained strength characteristics govern (Adams and Hayes, 1967 and Vesic, 1969), a complete procedure for predicting anchor holding capacity can be formed (Beard and Lee, 1975). In essence, holding capacity factors for short-term capacity in cohesive soil, $\bar{N}_c$, and long-term capacity under drained conditions from research on cohesionless soil, $\bar{N}_q$, are available as functions of embedment ratio, D/B.

Adams and Hayes (1967) conducted the only previously reported long-term model anchor tests. In three tests in a remolded clay with an undrained shear strength of 100 to 170 kPa, long-term capacities ranged from one-fifth to one-fifteenth of the short-term capacity as embedment went from deep to shallow (undefined overconsolidation ratios, OCR's, estimated to range from 50 to 400). In another test an overconsolidated clay was used that was prepared as a slurry and then consolidated to an undrained shear strength of 10 to 14 kPa. For a deep anchor the long-term capacity was measured as about six-tenths of the short capacity (undefined OCR estimated to be approximately 30). Adams and Hayes demonstrated that dramatic reductions in the short-term capacity of anchors can be experienced when loaded long-term. They suggested the use of drained strength properties as a logical method for estimating long-term capacity. They applied a theory developed for sands and achieved reasonable agreement with the measured long-term capacities (measured capacities were about 80, 180, 80, and 150% of estimated values).

The data of Adams and Hayes were further analyzed by Meyerhof and Adams (1968). For shallow anchor breakout the following general semitheoretical relationships were proposed.

Circular anchors:

$$Q_u = \pi c BD + S(\pi/2)\gamma BD^2 K_u \tan\phi + W \tag{2}$$

and

Rectangular anchors:

$$Q_u = 2c D(B + L) + \gamma D^2 (2sB + L - B) K_u \tan\phi + W \tag{3}$$

where Q_u = ultimate uplift load

s = shape factor

γ = soil unit weight

K_u = uplift coefficient of earth pressure

ϕ = soil friction angle

W = weight of uplifted soil

For deep anchors, these expressions were modified:

Circular anchors:

$$Q_u = \pi c BH + s(\pi/2)\gamma B(2D - H) H K_u \tan\phi + W \tag{4}$$

and

Rectangular anchors:

$$Q_u = 2c H(B + L) + \gamma(2D - H) H (2s B + L - B) K_u \tan\phi + W \tag{5}$$

where H = limiting vertical extent of the failure surface. These equations were used to estimate the long-term capacities of the Adams and Hayes tests, and the agreement was nearly identical to that attained by Adams and Hayes.

Radhakrishna and Adams (1973) have reported the results of two long-term field tests on cylindrical footings and three long-term field tests on cylindrical footings with belled bases. The long-term capacities were from 30% to 50% less than measured short-term capacities. The soil was a fissured clay. Using the methods proposed by Meyerhof and Adams, Radhakrishna and Adams estimated the long-term capacities and compared them to test results. The estimates for the belled footing were either too high or too low, depending on whether peak or residual drained properties were used.

Three problems exist in applying either Vesic's method or Meyerhof and Adams method for estimating holding capacity in cohesive soil. The first deals with using the $\bar{N}_c$ factors that were obtained from tests using somewhat artificial and unrealistic conditions for seafloor anchors. The second deals with using $\bar{N}_q$ factors for drained cohesive soil that were obtained from tests in dry sands. The third deals with using the soil's weight in short-term failure analysis when the soil involved may be neutrally buoyant within the soil mass.

The N_c factors do not include the effect of suction. Loading an anchor creates lower than ambient pressures beneath it that help maintain the sediment in contact with the fluke. This condition is defined as suction. Since these negative pressures will dissipate with time, it has been common in previous research to neglect suction and its contribution to short-term capacity. Model studies have usually been performed with vented flukes to eliminate suction forces; if not, correction factors were applied to remove the suction force. This was done because most of the research was in support of terrestial applications where venting could take place through partially saturated soils. In the ocean, however, because total saturation is usually assured, rapid dissipation of the suction cannot take place unless cavitation occurs (limited to shallow water and gassy sediments). It is more likely that some amount of suction will act until final pore pressure equalization is obtained at the long-term condition.

The values of $\bar{N}_q$, that were obtained from model anchor tests in dry sand, control the prediction of long-term capacity in cohesive soil. It would be a large and potentially unsafe extrapolation to use these factors directly in anchorage designs without experimental verification in actual cohesive soil.

The soil weight terms in both formulas ($A\gamma_b DN_q = W$, for $N_q = 1$) are carryovers from footing and deep foundation analysis that may not apply to the case of embedment anchors.

Another problem is that of soil creep. Many cohesive soils exhibit a phenomenon where continual shear-straining occurs under a constant state of stress. Some soils have failed at sustained stresses of only 60% of their measured strength (Singh and Mitchell, 1968). A long-term static load on an anchor would create a constant state of stress in the soil. However, because of the lack of knowledge about the creep behavior of ocean soils, it is difficult to assess the impact of creep on anchor holding capacity.

Approach and Scope

The approach taken to solving these problems was a program of ocean soil testing, laboratory model anchor testing, intermediate size field anchor testing, and finite element modeling. Soil testing was done to study the behavior of several typical deep ocean soils and the soil used in the laboratory model tests under long-term constant states of stress. Also, classification and strength tests were performed on the soils used for the laboratory model anchor tests and the field tests. The laboratory model testing was conducted in two series: the first, short-term tests to study suction and pore water pressure generation; the second, long-term tests to study pore pressure dissipation, anchor deflection under sustained loading, and ultimate long-term capacity. The field testing was conducted to provide larger scale comparisons for the long-term laboratory model tests. Only anchor displacement under sustained loading and ultimate capacity data were gathered during these tests. The finite element modeling was performed to provide help in drawing conclusions about basic anchor behavior. Pore water pressures, pore water flow, stress fields, and load deflection were studied using the finite element method. In general, the work was directed toward developing a method for confidently predicting the long-term capacity of deep ocean direct embedment anchors. Of particular importance was the identification of cases when the long-term capacity would be less than the short-term capacity.

CREEP OF SEAFLOOR SOILS

Creep is a complex stress-strain-time phenomenon dependent on soil type, soil structure, stress history, drainage conditions, type of loading, and perhaps other factors. No rheological model is general enough to model the variety of creep behavior that has been observed. However, Singh and Mitchell (1968) have presented a phenomenological relationship that appears to describe creep behavior in the range of engineering interest; i.e., applied shear stresses from 30% to 90% of failure. The Singh-Mitchell relationship is:

$$\varepsilon - a_c = \frac{A_c}{1 - m} e^{\alpha \overline{D}} t^{1-m}$$

where ε = strain

a_c = constant of integration

A_c = fictitious strain at zero deviator stress

$\overline{D}$ = stress level (deviator stress applied/deviator stress at failure)

m = slope of logarithmic/logarithmic-stress-rate/time curve

α = midrange slope of logarithm-strain-rate/deviator-stress curve

t = time

One value of this relationship is that, based on a few creep tests and the strength/water-content relationship, creep behavior can be estimated at any stress level at any water content. This relationship is valid for constant stress loading, which is the case in most long-term anchoring problems. To study the creep behavior of deep ocean sediments, samples of the two most common types - pelagic clay and calcareous ooze - were tested to determine their creep parameters for the Singh-Mitchell model. The sediments represent about 28% and 35% of the seafloor, respectively. The determined parameters were then used to estimate creep behavior under a variety of conditions. For each soil, the parameters were determined at two water contents that represented soil depths of about 20 and 60 feet. This is the expected range of anchor embedment.

For the pelagic clay, the parameters of interest were found to be about the same at each water content. They were A = 0.0064, m = 0.89, and α = 4.4. A comparison between the empirical relationship and the test data at one of the water contents is shown in Figure 1. For the calcareous ooze, a single set of parameters did not describe the creep behavior at two water contents. For softer soil (higher water content), A = 0.003, m = 0.95, and α = 5.9 gave the best empirical fit. For denser soil (lower water content) A = 0.00045, m = 0.79, and α =7.3 gave the best fit. A comparison between the data and the empirical fit for softer calcareous ooze is shown in Figure 2. For Figures 1 and 2 the strains at 1 minute have been force-fitted to the strain at that stress level in an undrained triaxial shear test.

The determined parameters were used to estimate strain after extended time periods. Estimated strain at several stress levels at time periods of 1, 2, 5, and 10 years less the instantaneous strain at 1 minute are presented in Table 1. These values represent a considerable extrapolation of the data on which the empirical parameters are based. Ten years is about 3 logarithmic cycles of time greater than the maximum test time. The data do cover about 3 logarithmic cycles of time from 1 to 5×10^3 minutes (10 years is 5.25×10^6 minutes). Such extrapolations are, however, the only practical method of estimating behavior for such long time periods. Table 1 suggests that creep will not be a significant problem of anchor behavior in these soils. Strains after periods of 10 years should not exceed 10%. When other factors such as drainage and resulting volumetric changes are considered, creep may be of minor importance. Also, it should be noted that for deviator stress levels of 0.5 (equivalent to a factor of safety of 2 to short-term failure), the 10-year strains do not exceed 3%.

Creep rupture, defined by an increasing rate of shear until failure is reached, cannot be described by the Singh-Mitchell model. Their model can be used only for estimating the amount of strain to be expected under sustained stress in the range of engineering interest, which is normally between 30% to 90% of the rupture stress. To learn more about the possibility of creep rupture at high stress levels, four creep tests were performed at stress levels of 70% to 90% of failure. Creep rupture was not observed under these sustained stresses.

SHORT-TERM TESTING

Short-term anchor pullout tests were conducted to study pore water pressure generation and suction, to provide a comparative base for the long-term tests, and study the relationship between short-term and long-term anchor holding capacity.

Procedures

A total of 26 short-term anchor tests were performed - 17 by CEL; 2 by California State University, Long Beach (CSULB) (Yen, 1975); and 7 by the University of Massachusetts (UM) (Bemben and Kupferman, 1974). Many were conducted to provide specific background information for cyclic or long-term static tests, but their inclusion here helps form a more complete statistical base. The characteristics of the soils used are summarized in Table 2. The soil shear strengths for tests varied from 0.20 to 11.7 kPa, and OCR's varied from about 1 to 10. Embedment ratios ranged from 1.5 to 6. In one test, an embedment ratio of about 30 was simulated with a surcharge.

The soil was usually placed in barrels as a slurry (the exceptions are the UM tests) and kept submerged throughout the testing. The soft slurries were mixed under vacuum. Anchor plates were located at depth as the soil was built up around and over them. Barrel diameters were a minimum of 5.5 times the anchor diameters to prevent side wall influence (Erden, 1971). Higher strengths and OCR's were obtained by consolidating the soil with surface surcharges. Two lubricated layers of thin plastic film were used to prevent arching of the surcharge load to the walls of the barrels. Excess pore water pressure and surface deflection graphs versus time were used to monitor consolidation. Undrained shear strength profiles were determined with a vane shear apparatus. For some tests, water content and bulk wet density profiles were also obtained. The degree of saturation was determined for many tests and found to exceed 97% for all cases except for the UM tests.

The anchors were incrementally loaded to failure. Load was applied to the anchors through a system of cables and pulleys and a thin shaft connected to the anchor plate. For some tests where estimated OCR's were from 1 to 2 and the anchors were deeply embedded, pore water pressures were measured. The measurements were made in the soil about the anchor at up to 12 locations and were measured at each load increment. A description and discussion of the pore water pressure system was presented by Beard (1974). Anchor deflection versus load data was recorded.

Results

A summary of the test results is presented in Table 3. The trends in the data are consistent with previous work. Holding capacity increased with anchor size and soil shear strength. However, when anchor size and sediment strength were about equal, holding capacities were nearly constant with increased depth. This suggests that one holding capacity factor, $\overline{N}_c$, could be applied to almost all of the data. This is best illustrated by the data in Figure 3 where values of $\overline{N}_c$ are plotted versus values of D/B. The $\overline{N}_c$ factors were derived from the test data using Equation 1 with $\overline{N}_q = 0$. Except for tests 20-22 (UM) at a D/B of 2.1, the $\overline{N}_c$'s fell about the $\overline{N}_c = 15$ line. It is thought that for tests 20-22 (UM) the lack of complete saturation did not allow for full suction to develop. The remaining 23 tests did exhibit deep behavior. The $\overline{N}_c$'s of the tests exhibiting deep anchor behavior were satistically analyzed. The lower and upper 95% confidence limit from a log normal fit were -18% to +22% of $\overline{N}_c = 15$. In other words when the undrained shear strength is accurately known and $\overline{N}_c = 15$ is used to predict the breakout load, the test result should, 95% of the time, be within -18% to +22% of the predicted value.

To study the soil weight terms that have been included by previous researchers, the data for tests 10-19 were analyzed two ways. These tests were chosen because their soil densities were known more accurately than those of other tests. The data were analyzed both by ignoring the soil weight term and by including it. The scatter of the determined $\overline{N}_c$ factors for these 10 tests when the weight term was included was twice what it was when the term was excluded. The increased scatter of $\overline{N}_c$ would indicate that inclusion of a soil weight term is incorrect.

Pore water pressures were measured in the soil around the anchor plates in tests 3, 4, and 5. Figure 4 shows the locations of the measurement devices for these tests. The soil was near normally consolidated, and the anchors were embedded deeply. The approximate pore water pressure distribution is shown in Figure 5. The changes in pore water pressure were linearly proportional to the applied load. The pressure distribution shows that the soil above the anchor is in a general state of compression and that the soil below the anchor is in a general state of tension.

LONG-TERM TESTING

Two types of long-term model anchor tests were performed. In one type, in near normally consolidated soil, a load short of causing short-term failure was applied and anchor and soil behavior monitored with time. In the other type, many load increments were applied to each anchor with pore water pressure allowed to dissipate between each increment. This was done in an attempt to achieve the ultimate long-term capacity, thereby allowing for comparison of proposed prediction procedures. Eight long-term tests were conducted in the first manner, three by CEL and five by CSULB (Yen, 1975). Ten long-term tests were conducted in the second manner, all by CEL.

Procedures

The state of the soils used for the single increment tests can be described as near normally consolidated. For the multi-increment tests, the soil was varied from normally consolidated to an overconsolidation ratio of about 35. For the single increment long-term tests, CEL used soil 1 and CSULB used soil 2 described in Table 2. For the multi-increment tests CEL used soil 3 described in Table 2. The tests were set up as described for short-term testing procedures. Loads were applied with weights through a system of cables and pulleys. Anchor displacements were monitored at selected time intervals by an automatic recording system. For the multi-increment tests, new loads were not applied until the anchors stopped moving or the movement rate became very small. The typical time between increments was 14 to 30 days. The soils were kept submerged throughout the tests.

Undrained shear strength profiles were measured for all the tests for estimating short-term capacities. For the multi-increment loading tests, water content and bulk wet density profiles were also obtained. Soil 3, used for the multi-increment tests, was tested to determine its drained friction angle and cohesion intercept and the relationship of these parameters to confining stress and overconsolidation ratio.

Results

The results are best separated into two categories: one being the single increment tests that were done primarily to study time-dependent processes of long-term capacity, and the other being the multi-increment tests that primarily were done to study the controlling factors at the ultimate long-term capacity.

Table 4 summarizes the single increment tests program. A plot of displacement versus time is given in Figure 6 for tests 1-3 in Table 4. The displacement time history is similar to that observed in a consolidation process. This similarity is also displayed by the plot of excess pore water pressure versus time in Figure 7. The data are from test 2 for two locations, one is one radius below and the other two radii above the center of the fluke. The trends in displacement and pore water pressure dissipation with time were consistent from test to test. None of the anchors pulled out, even for two tests where the long-term loads applied were estimated to be 75% of the short-term capacity, including suction. These results suggest that for near normally consolidated soil, a process of consolidation occurs above the anchor with consequent soil strengthening.

The reverse seems to occur below the anchor fluke. The test results were also examined by plotting displacement rate versus time (Figure 8). This type of plot helps to identify trends in behavior after short-term processes such as consolidation have been completed and is often used in creep studies. Not all the single increment test data appear on this plot. For tests 4 and 7 of Table 4, displacements were not large enough to determine displacement rate values for this plot. In test 1 of Table 4, displacements were not measured accurately enough to determine displacement rate values. The data for tests 5 and 8 of Table 4 were similar to those shown in Figure 8, but out of range of the plot. The upper slopes identify the consolidation phase, and the lower slopes identify the creep phase. Differences between slopes and locations are a function of soil and anchor parameters. Of importance is that the displacement rate continued to decrease with time for all the tests. Because consolidation and strengthening is occurring in the soil as the displacement rates decrease, it is apparent that these anchors were not going to pull out. The single increment tests show that anchors in near normally consolidated soils can sustain long-term loads that approach their short-term capacity without pulling out.

The multi-increment tests were conducted at varied OCR's and embedment depths using 50-mm-diameter anchors in soil 3 of Table 2. The test parameters are summarized in Table 5 along with estimated short-term capacities based on undrained shear strength profiles and the ultimate long-term capacities measured. Only for the tests at an embedment ratio of 6 were the long-term capacities greater than the short-term capacities. Figures 9, 10, and 11 are graphs of displacement versus load for these long-term tests at embedment ratios of 1.5, 3, and 6, respectively. The same trends are observed in each graph; tests at higher OCR's resulted in lower displacement at failure and higher loads at failure than tests at lower OCR's. The relationships between OCR, relative embedment depths, short-term and long-term holding capacity, is best illustrated by a graph of the ratio of the long- to short-term capacity versus the OCR's (Figure 12).

The soil used for the multi-increment anchor tests was tested to determine its drained strength properties at low confining stresses and low to moderate

OCR's, such as those acting during the model anchor tests. Figure 13 presents the relationships between the drained friction angle, $\bar{\phi}$, and the maximum past pressure, p_m. Figure 14 presents the cohesion intercept, $\bar{c}$, as a function of the maximum past pressure. These data were obtained from triaxial shear tests conducted using Berkeley type cells with air bushings. These parameters, $\bar{\phi}$ and $\bar{c}$, along with soil density are the key parameters necessary for analyzing or estimating long-term capacity.

FINITE ELEMENT MODELING

Finite element method (FEM) modeling of holding capacity was done to confirm the observed results of the model anchor tests and to study a wider scope of anchor-soil conditions than could be done with physical models. Two different FEM models were used. One by Ghaboussi and Wilson (1971) was used to study pore water pressures and direction of pore water flow. The other, a modified version of Wilson's (1965) bilinear elastic formulation, was used to study stress states and pore water pressure over a wide range of embedment ratios and overconsolidation ratios (designated by Henkel's porewater pressure parameter).* For both models, full suction was allowed to develop below the anchor.

The FEM results support the results of the physical modeling. For anchors embedded in normally consolidated soil ($a \approx 0.5$), the pore water pressures generated were positive above the anchor and negative below the anchor. This was the case even for anchors as shallow as $D/B = 2$. Flow of the pore water was generally from the top to the bottom of the anchor as illustrated by Figure 15. For heavily overconsolidated soils ($a \approx -0.6$), negative pore water pressures

*Henkel's a parameter (Henkel, 1960) was used because it is a more general pore pressure parameter than Skempton's pore pressure parameter. It is defined as the quotient of dividing the difference in the pore pressure change and the average total stress by the shear stress. The major difference between the Henkel and Skempton pore pressure parameters is the inclusion of the intermediate principal stress in Henkel's pore pressure parameter definition.

were the general rule, particularly for low embedment ratios. Even for deeper cases (i.e., D/B = 10), positive pore water pressures were limited to the hemisphere of sediment directly above the anchor. The magnitude of the negative pressures above the fluke was small compared to those pressures in the sediment below the fluke. This would result in less soil softening above than below the fluke. When the overconsolidation was reduced ($a \approx -0.3$), the negative pore water pressure zone above the anchor at higher embedment ratios did not occur. This indicates that for moderate overconsolidation with higher embedment ratios softening of the soil above the anchor would not occur. This is an indication of trends that are not easily quantifiable in terms of actual conditions. No universal relationship exists between the overconsolidation ratio and the a parameter; it varies from soil to soil. However, when $a \approx -0.6$, the overconsolidation ratio is probably 20 or greater. For $a \approx -0.3$, the overconsolidation is more moderate - in the range of 4 to 10. For embedment anchor considerations, it is unlikely that a heavily overconsolidated soil with an $a \approx -0.6$ will be encountered at embedment depths attainable with present anchor hardware. On the continental shelves it is probable that moderately overconsolidated soil will be encountered. In the deep ocean, the prevalent case will be normally or near normally consolidated soil. The results of the FEM model suggest that for the most probable embedment soil conditions, negative pore water pressures will not be generated above the anchor. Therefore, softening of the soil above the anchor will not be a problem, rather the soil will harden. This is supported by the data in Table 5; that is, tests at higher embedment ratios had long-term capacities greater than their short-term capacities even when the soil was moderately overconsolidated. This does not account for what occurs below the anchor; in all cases studied, negative pore water pressures were generated below the anchor. This pressure will lead to soil softening. Under a long-term condition, the contribution of suction will be lost.

The FEM modeling results support the results of the physical model tests concerning the contribution of suction to the short-term capacity. Figure 16 is a plot of normalized anchor displacement versus normalized load, defined as the applied load, L, divided by the product of the undrained shear strength,

s_u, and the fluke area of the anchor, A. The normalized load is the same as $\overline{N}_c$ for a purely cohesive soil. The data in Figure 16 represents a deeply embedded anchor. Note that failure, defined by a large increase in displacement with little additional load, occurs between a normalized load of 12 to 15. This compares well to $\overline{N}_c = 15$, determined from physical modeling.

LONG-TERM FIELD TESTING

Field testing was done with intermediate sized anchors to provide a large scale comparison with the laboratory anchor tests and to provide data using an anchor embedded and keyed in the typical manner. The typical embedment technique is for the anchor to be launched on edge from a gun at high velocity toward the seafloor with the kinetic energy of the anchor being used to overcome the penetration resistance of the soil. A cable attached to the anchor is dragged down into the seafloor by the anchor. Keying occurs when the cable is pulled; the anchor, through mechanical features, is rotated until its main surface is normal to the direction of pull. This method of anchor installation and keying undoubtedly disturbs the soil and results in a condition much different from a laboratory anchor where no disturbance occurs. Rocker (1977) found the short-term field capacity to be about 20% less than values predicted from laboratory tests.

The site chosen to conduct these tests was a tidal flat at the Mare Island Naval Shipyard. The soil there is a normally consolidated San Francisco Bay mud of moderate sensitivity and high compressibility. Others (Duncan and Buchignani, 1973) have shown that bay mud loses strength when subjected to sustained loads (i.e., creep susceptible). This type of loading exists during long-term anchor testing. This soil is generally similar to fine-grained ocean soils, except perhaps in its creep behavior.

In view of the test conditions and soil characteristics, it was evident that this would be a good site for investigating long-term holding capacity in the field.

Procedures

Aircraft matting was placed over the tidal flat to provide support for personnel and equipment on the very soft soil. Six 0.23 x 0.46-m test anchors were fired into the tidal flat with a gun at velocities of about 45 mps. Penetrations ranged from 3 to 4 m. Tripods, supported on the aircraft matting, were placed over the anchor locations to test the anchors. Five of these anchors were keyed under short-term loads by pulling against the tripod. These were essentially short-term tests because a load peak was reached. The other anchor was not keyed prior to the long-term testing. Long-term loads were applied to the anchors with deadloads using a block and tackle attached to the tripod. The remoteness of the site from CEL did not allow for changing the load increments based on a specific time interval or displacement rate. Rather, the condition of one test would govern when a trip was made to adjust the loads of all the tests then underway. Thus, the size of the load increments and the lengths of application varied significantly from test to test and within a given test. Displacements were measured at the cable to the anchor with a ruler.

In-situ vane shear tests were conducted and cores were taken for laboratory analysis. In the laboratory, classification tests were performed and specific gravity, bulk wet densities, natural water contents, and grain size distribution were determined. Triaxial tests were performed to determine drained strength parameters of the soil.

The long-term holding capacity was expected to exceed the short-term holding capacity because the soil was normally consolidated. This presented problems in load application. To reach the ultimate drained failure condition a careful process of incrementing the load to allow drainage to occur between each load increment was necessary to prevent rupture at intermediate conditions. The procedure to accomplish this required applying small load increments once the short-term failure load was approached. However, the remoteness of the site necessitated applying large load increments, sometimes as great as 20% of the short-term holding capacity.

Results

The soil characteristics at the site are summarized in Table 6. The undrained shear strength profile in the range of interest was nearly linear from 3.8 kPa at 1 meter to 10.7 kPa at 5 meters. The effective overburden pressure was also nearly linear from 5.3 kPa at 1 meter to 13.4 kPa at 5 meters. The anchor depths during long-term loading were between 1.5 and 3 meters. Triaxial tests were performed after consolidation to stress levels appropriate for these depths, and the drained strength parameters were found to be $\bar{c} = 1$ kPa and $\bar{\phi} = 40$ degrees. A summary of the six anchor tests is given in Table 7.

Figure 17 is a graph of anchor tests D, F, and J, showing the depth of embedment at the termination of each load increment and the time in days that each increment was applied. Also plotted is a line of the approximate mean of the short-term tests conducted by Rocker (1977) at this site and a line of predicted long-term capacities. Each of these anchors held long-term loads equalling or exceeding the short-term holding capacities. Note that for both D and J the initial loadings were placed soon after installation. The loads were small, and the short-term capacity was approached cautiously. By contrast, in test F the first increment was applied 440 days after keying and approached the short-term capacity. This load was subsequently reduced during the first day due to excessive displacements (about 0.5 meter). By careful reapplication of loads, the short-term holding capacity line was later reached. Significant in test J is that the short-term capacity curve was reached in 2 days from first long-term loading.

Figure 18 is a graph of tests G, I, and K. A problem was experienced with each of these tests. Test G was loaded long-term 440 days after keying and progressed well until the wire to the fluke parted under a load of 4.75 kN. Test I was loaded long-term 1 day after keying and was approaching the short-term holding capacity curve when the next increment applied was more than 50% of the existing long-term load (2.7 kN increment on 4.6 kN existing load). The total load then applied exceeded the short-term capacity, and the anchor failed. In test K, immediately after the anchor was keyed, a long-term load of about 60% of the short-term capacity was applied, causing large displacements (0.3 meter). That load was removed and smaller long-term loads applied; however, the anchor pulled out without ever holding any load longer than 0.3 days.

DISCUSSION

Creep

Because creep is a complex stress-strain time phenomenon dependent on a variety of factors, it is not surprising that no general procedures exist for including creep behavior in geotechnical designs. One approach is to design using the lowest creep rupture strength expected during the life of the facility. Another approach is to design for a limiting displacement using creep stress-strain data. Both of these approaches ignore the fact that drainage is occurring under the applied state of stress. This drainage alters the stress-strain time response of the soil. Because most creep tests are conducted undrained, the data are limited in their application. Drained creep tests can be performed, but it is not possible (except for simple cases) to simulate field drainage conditions. Consequently, a good deal of engineering judgment is required when dealing with soils that exhibit significant creep behavior.

The samples, whose creep behavior was studied in this work, represent two general seafloor types: pelagic clay and calcareous ooze. These general soil types cover 28% and 35% of the seafloor, respectively. For the samples tested, creep did not appear to be a problem (see Figures 1 and 2 and Table 1). While they do represent these sediment provinces, it must be kept in mind that the soils are samples from only two locations and that these sediment provinces cover vast areas of the seafloor. Exceptions to any generalizations will be found. The character of pelagic clays has been found to be more consistent than calcareous oozes, which can vary considerably in grain size and shape. However, two factors allow some generalizations: (1) drainage and (2) the general state of consolidation of these soil types. These soils are generally near normally consolidated. Under this condition, as shown in the model anchor testing (Figure 5) and the finite element modeling of anchors (Figure 15), positive pore water pressures will be generated above anchors. Therefore, the drainage will densify the soil above the anchor, reduce expected creep strain, and increase the creep rupture strength. The factor of safety applied to the design must also be taken into account. Anchor holding capacity is essentially a bearing capacity problem, and for bearing capacity problems the factor of safety against failure is usually 3 for the normally expected maximum load and usually not less than 2 for the maximum load ever expected. In view of the lack of experience in using embedment anchors in soil, it is unlikely that any smaller factors of safety will be used than those in bearing capacity design. As a result, the deviator stress levels will be 0.5 or less. In Table 1, the 10-year strains are seen to be less than 2.5% for both soils at this stress level.

In summary, it can be generalized that creep will not be a problem in embedment anchor designs in pelagic clays and calcareous oozes and will play a minor role in overall anchor behavior. Other cohesive soils need to be evaluated case by case.

Short-Term Holding Capacity

Short-term holding capacity is important in designing an embedment anchor for long-term loading because it is the holding capacity base that cannot be exceeded even though the long-term capacity may be greater. Therefore, an accurate estimate of the short-term capacity is necessary to properly utilize an embedment anchor's efficiency. The most significant consideration in estimating the short-term holding capacity is whether or not suction will be developed under the anchor plate. Previous researchers have usually conducted their tests so that suction did not occur, or they subtracted its effect from measured breakout forces. The holding capacity factor, $\overline{N}_c$, thus obtained for deep anchors was about 9. Using this value as the limiting holding capacity factor for estimating short-term holding capacity was recommended by Taylor and Lee (1972). The results of this work, both in model testing (Figure 3) and finite element analysis (Figure 16), have shown that with suction the limiting breakout factor is about 15.

The logic behind using $\overline{N}_c = 9$ is that the suction must be lost with time and, therefore, it should not be included. However, the same process of drainage that dissipates suction transfers the anchors from a short-term to a long-term condition. That is, the water flow to relieve the suction pressure comes from above the anchor (see Figure 15). Consequently, using an $\overline{N}_c$ greater than 9 as the limiting breakout factor for short-term loading will not give overestimations. Rather the larger breakout factor will lead to better evaluations of short-term capacity. Some verification of this line of reasoning is given by tests 6 and 9 in Table 4 and Figures 6 and 8. In these tests, single increment long-term loads equivalent to 75% of the short-term capacity estimated, using $\overline{N}_c = 15$, did not cause failure. These loads were 25% larger than the short-term capacity estimated, using $\overline{N}_c = 9$.

The results of the model tests gave a limiting $\overline{N}_c$ of 15 (Figure 3), but the results need to be tempered with the results of field tests. The laboratory tests represent an ideal condition that, for practical reasons, is not reproduced in the field. In the field the anchor flukes must be inserted and keyed. In cohesive soils, which usually exhibit a sensitivity, the effect of insertion and keying is a reduction in the shear strength of the soil. It is not presently possible to quantify the amount of strength reduction relative to the sensitivity nor to specify where strength reductions are occurring relative to the fluke. Rocker (1977) found about a 20% reduction in capacity from these effects. Valent (1978) analyzed field data from embedment anchor tests in calcareous ooze, pelagic clay, and terrigenous deposits. He found the data from the tests in calcareous ooze to be inconclusive but suggestive of significant capacity reduction. The reduction in capacity found in the pelagic clay was 30% and in the terrigenous material the reduction in capacity was 20%. It is unfortunate that the changes that occur in the soil during penetration and keying cannot be accounted for systematically. With the field data available it has not been possible to determine a relationship between soil type and sensitivity and the reductions observed. In a soil with a particularly high sensitivity (that of the calcareous ooze was 10), the reduction in the limiting breakout factor appears to be severe. Conversely, in a soil with very low sensitivity, no reduction in the limiting breakout factor may be found. It may also be possible to obtain the ideal condition in the field by leaving the anchor unloaded after keying until the soil regains the strength lost during penetration. It should be stressed that the problem is not a lack of knowledge about breakout factors but rather a lack of knowledge about soil strength after penetration and keying.

In view of the fact that a semitheoretical approach to this problem cannot be developed Valent (1978) has suggested using a single empirical correction factor for different sediments to account for these effects. The factors would modify the undrained shear strength used in Equation 1. The factors recommended by Valent for short-term capacity are: $f = 0.8$ for terrigenous silty clays and clayey silts, $f = 0.7$ for pelagic clays, and $f = 0.25$ for calcareous ooze. For calcareous oozes the factor is very low, but should be used until data supporting a higher value is available.

A term, $\gamma_b DA$, for the weight of the soil above the anchor is also included in short-term holding capacity equations. There are several reasons to doubt that this term applies to anchors in submerged soils such as seafloor soils. An identical term is used in pile capacity equations for cohesive soils to account for the contribution to the pile capacity by soil displaced by the pile; a buoyancy term that increases pile capacity just as the weight of the pile reduces the capacity. For embedment anchors little soil is displaced by the anchor and as a result any buoyancy term would be negligibly small as is the anchor weight.

The equation for estimating short-term holding capacity in cohesive soils would therefore be

$$F = \bar{N}_c A f s_u (0.84 + 0.16 B/L) \qquad (6)$$

where f = correction factor to account for soil disturbance

The value of $\bar{N}_c$ is usually provided in a graph of this factor versus relative embedment depth, D/B. The value of $\bar{N}_c$ increases as depth of embedment increases until deep behavior is obtained. The work of others (Ali, 1968; Kupferman, 1971; Bhatnager, 1968; and Adams and Hayes, 1967) has shown that the relative depth at which deep behavior is obtained is also a function of soil strength. As soil strength increases, the relative embedment depth at which deep behavior occurs also increases. Taylor and Lee (1972) have graphically presented these relationships. Figure 19 is similar to that presented by Taylor and Lee except the effect of suction has been included. The increases in $\overline{N}_c$ are equal to the difference in $\overline{N}_c$ for deep anchors with suction (Figure 15) and for deep anchors without suction (Figure 9). In doing this, an assumption has been made that suction force is independent of relative embedment depth. This assumption is reasonable except for shallow anchors at D/B's less than 1.

Long-Term Holding Capacity

Estimating long-term holding capacity in cohesive soils is based on the principle that behavior of cohesive and cohesionless soils is basically the same. Hence, in cohesive soils with full drainage, the effective stress principle can be applied using the drained strength parameters $\overline{\phi}$ and $\overline{c}$. Long-term holding capacity is defined as a situation where full drainage has occurred (excess pore water pressures have been dissipated). Using this principle suggests that holding capacity factors for drained cohesionless soils can be applied to the analysis of long-term holding capacity in cohesive deposits. To verify this extrapolation the multi-increment long-term tests were conducted. The data resulting from these tests (Figures 9, 10, and 11) were compared to predictions using the methods of Vesic (1969) and Meyerhof and Adams (1968). The values of $\overline{c}$ and $\overline{\phi}$ were taken from Figures 13 and 14 knowing maximum past pressure from the soil overburden and applied surcharge loads. $\overline{N}_c$'s were taken as those for long-term loading as given by Taylor and Lee (1972) and presented here in Figure 20. A comparison of these results is given in Table 8. The short-term capacities presented are the same as those in Table 5 and were estimated using undrained shear strengths and $\overline{N}_c$'s for short-term loading from Figure 19.

In Table 8 it can be seen that there is not much difference in the estimated long-term holding capacities using the $\bar{N}_q$'s of Meyerhof and Adams or those of Vesic. Out of the range of the test data, however, significant differences are found in $\bar{N}_q$ values from these authors. This is shown in Figure 21 where Vesic's $\bar{N}_q$'s are plotted with $\bar{N}_q$'s derived from Meyerhof and Adam's method. The $\bar{N}_q$'s for Meyerhof and Adams method are based on buoyant unit soil weights that were varied from 640 kg/m^3 at $\phi = 20$ degrees to 880 kg/m^3 at $\phi = 40$ degrees.

Based on the test results of others (Esquivel-Diaz, 1967 and Bemben and Kupferman, 1974), better agreement between experimentally obtained N_q's and analytical methods is found (Figure 21) using $\bar{N}_q$'s derived from Meyerhof and Adam's method. In addition to not fitting the data well, Figure 21 also shows that Vesic's breakout factors increase even after considerable embedment is attained. This is inconsistent with test data that show which breakout factors remain relatively constant with increasing embedment after deep behavior is attained.

Another comparison of using breakout factors for cohesionless soil in long-term analysis of cohesive soils is made in Figure 21 where $\bar{N}_q$'s derived from the long-term multi-increment tests are plotted for comparison with the theoretical values. The estimated friction angles for these tests range from about 35 to 45 degrees. The results plot about Meyerhof and Adams $\bar{N}_q$ curve for a friction angle of 40 degrees with two exceptions: tests 4 and 8. These tests represent the extremes of the test parameters: a shallow anchor in a highly overconsolidated soil, test 4; and a deep anchor in a normally consolidated soil, test 8.

In test 4 the measured capacity was almost twice that predicted. This could result from the cohesion, $\bar{c}$, being higher than estimated or from a lack of full drainage, resulting in a partially undrained failure. Most of the error is probably coming from the cohesion intercept, not a lack of full drainage. The previous loads which were 88% and 76% of the failure load, were in place for 29 and 47 days, respectively, without causing failure. These time intervals seem long enough to achieve full drainage for an anchor only 75 mm below the soil surface. While the data in Figure 14 could support use of a higher cohesion value at the maximum past pressure of about 22 kPa, the difference could be more associated with difficulties in measuring $\bar{c}$ than the actual value of $\bar{c}$. Of importance is that the measured capacity of 38N was greater than the prediction of about 21N; a safe error.

In test 8 the opposite occurred; the measured capacity was about one-fifth the estimated capacity. A difficulty in analyzing these data is that the test soil was loose (normally consolidated) but exhibited a high friction angle. Based on the maximum past pressure, the estimated value of $\bar{\phi}$ from Figure 13 is 42 degrees. This high value is not unusual for loose cohesive soils that experienced low maximum past pressures. In this case the maximum past pressure was only about 2 kPa. In loose sands at low confining pressure, similar high friction angles are found. At failure, the fabric of these sands collapses and only a relatively small volume of sand is involved in the failure (local shear), compared to dense sands with equivalent friction angles where large volumes of sand are involved in the failure (general shear). Consequently, design values for these conditions are adjusted to account for these facts. In test 8 a similar behavior may be responsible for the result observed. The $\bar{N}_q$ derived from the test data would indicate a breakout factor for a loose soil was attained contrary to the breakout factor associated with the estimated friction angle of 42 degrees. This behavior can be deduced from other tests as well. At each embedment depth, the calculated $\bar{N}_q$'s increase with an increase in soil density (increase in OCR): low $\bar{N}_q$'s are found for loose soil, high $\bar{N}_q$'s are found for denser soil. In bearing capacity problems, the change in failure mode from dense to loose soil conditions is handled by using cohesions and friction angle tangents equal to two-thirds of measured values (Terzaghi and Peck, 1967). A similar procedure should be used for anchors in loose soils. In regard to test 8 and shown in Figure 21, the determined $\bar{N}_q$ falls close to values derived from Meyerhof and Adam's method for $\phi = 25$ degrees. The tangent of 25 degrees is about one-half the tangent of the estimated friction angle of 42 degrees, which gives some credence for making adjustments to the measured friction angle in loose or normally consolidated clays.

It should be noted that for test 8, this adjustment would still yield a long-term capacity greater than the short-term capacity. Hence, the short-term capacity would govern. Figure 12 supports this and indicates that the design of anchors in near normally consolidated soil at D/B's greater than about 4 wouid be governed by their short-term capacity.

Interpreting the results of the long-term field tests at Mare Island is difficult because each test was performed differently from the others. The estimated long-term holding capacity for the anchor size used versus depth has been plotted on Figures 17 and 18. A reduced friction angle was used in view of the preceding discussion. Only in tests D and J was this long-term holding capacity line reached by a continuous buildup of load. Each of these tests was carefully conducted with respect to the load buildup with time, and as drained tests they are reasonably valid. For the other tests, problems were encountered. An important problem was the size of the initial load increment and the effect it had on displacements. The soil at Mare Island is a San Francisco Bay mud and as such exhibits significant creep behavior. Duncan and Buchignani (1973) found that strength losses of 30% would be experienced after 1 week of sustained loading. Therefore, under a large increment of load, a short-term failure could occur from creep before the drained condition is reached. This was probably the case for the first increment of tests F and K and for the third increment placed on test I. In test F, an initial load equivalent to about 60% of short-term failure load resulted in excessive displacement that necessitated reducing the loading. (Because the first increment of load was applied 440 days after the fluke was keyed, it is assumed the soil had regained strength losses from keying, and an ideal condition existed. Therefore the load level percentage is based on using 15 for the limiting breakout factor.) In test K a similar "failure" occurred from a load of about 60% of the short-term capacity that was applied 0.01 days after the fluke was keyed. It seems that strength loss from creep under sustained loading was the most significant factor in causing the results observed in tests F and K. In test I, the third increment, which took the loading from 70% to about 110% of the short failure capacity, was excessive in regard to the soil's ability to resist that loading in view of its creep behavior.

Test J, in contrast to the other tests, was not keyed prior to long-term loading. As can be seen in Figure 17, this anchor was able to sustain a long-term load of about 60% of its short-term capacity within 0.2 days of initial loading. This result is quite unlike the results of tests F and K where initial long-term loads of 60% of their short-term capacity could not be sustained. Within 2 days the load on this anchor was at 100% of the short-term capacity. However, this result is confounded by the fact 2 days were taken to build up the load to the 100% level. What affect this had in terms of increases in the undrained strength and, hence, in increases in the creep strength is not known. The result of this one test is not sufficient for making any generalizations about what to expect in terms of initial anchor response to long-term loads between pre-keyed or unkeyed flukes.

CONCLUSIONS AND RECOMMENDATIONS

1. Suction under an anchor is an integral part of short-term holding capacity in cohesive soils and should not be ignored. The limiting breakout factor in cohesive soil under ideal conditions is about 15. For embedment anchors that are embedded, keyed, and pulled short-term the holding capacity is 20 to 30% less than predicted because of soil disturbance that is as yet not quantifiable in relation to soil type and sensitivity. The weight of the soil above the anchor should not be included in short-term analysis because of its buoyancy within its own medium.

Short-term holding capacity of embedment anchors in the field should be estimated using Equation 6 and the holding capacity factors given in Figure 19. It is suggested that Valent's reduction factors be used. The reduction factors are $f = 0.8$ for terrigenous clayey silts and silty clays, $f = 0.7$ for pelagic clays, and $f = 0.25$ for calcareous ooze. For laboratory or other ideal conditions (i.e., embedment anchors that have been keyed and then left until the soil has regained its strength), the short-term capacity should be estimated using Equation 6 and the holding capacity factors given in Figure 19 without reduction factors. Additional studies should be made of the problem of soil disturbance during anchor penetration and keying so that strength losses can be quantified to allow for a more rational approach to this significant problem. More pullout data in typical ocean soils is required, and such data should be gathered as a part of each service anchor installation.

2. Long-term holding capacity in cohesive soils can be analyzed using drained strength parameters. For loose cohesive soils (normally consolidated), drained strength parameters should be reduced by one-third before selecting breakout factors to account for a local rather than a general shear failure. Breakout factors ($\bar{N}_q$) developed for cohesionless soils can be readily applied to drained analysis of cohesive soils. The $\bar{N}_q$'s that were derived from Meyerhof and Adam's theory compared favorably with $\bar{N}_q$'s derived from the long-term model anchor tests. Using the one-third reduction in drained strength parameters seems valid - even in sensitive creep-susceptible soil-based on comparisons with the two tests at Mare Island when drained failures were obtained. Because suction does not exist under drained conditions, holding capacity factors $\bar{N}_c$ to be used with the soil's cohesion should be those developed for the "no suction" case. Based on the near normal consolidation of deep ocean seafloor sediments, long-term capacity will not be a critical factor; short-term holding capacity will usually govern designs. Long-term holding capacity in cohesive soils should be estimated using Equation 1 and the holding capacity factors of Figures 20 and 22. For loose soils drained strength parameters should be reduced one-third as they are for bearing capacity analysis. To determine $\bar{N}_q$'s the friction angle to use in Figure 22 should equal the arc tangent of two-thirds the tangent of the measured friction angle. In cohesive soils that are not creep susceptible, long-term design loads should be the lesser of the estimated long-term capacity and the estimated short-term capacity.

3. Creep-susceptible soils require an additional consideration in regard to initial long-term loading. An anchor in a creep-susceptible soil fails under an initial sustained load considerably less (40% less in San Francisco Bay mud) than its short-term capacity even though the long-term capacity based on drained strength

parameters is greater than the short-term capacity. This occurs because of losses in the soil's undrained strength from creep under the sustained load before a drained condition is reached. Creep is not expected to be a frequent problem in deep ocean sediments.

Long-term design capacity in cohesive soils that are creep-susceptible should be the lesser of the estimated long-term capacity and a short-term capacity estimated from the creep rupture strength of the soil before drained conditions are reached.

4. A Monte Carlo simulation or other statistical evaluation of the error to be expected in predicting holding capacity should be made. This evaluation would include the variances to be expected in soil properties, penetration prediction, keying distance estimation, and holding capacity prediction. The results would be recommended design factors of safety based on different scenarios of soil investigation and problem analysis. In lieu of this type of input design factors of safety are recommended consistent with present practice in bearing capacity analysis: use a factor of safety between 2 and 3 depending on the nature of the anchorage and the reliability with which the soil conditions have been determined.

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Table 1. Estimated Long-Term Creep Strain in Percent for Two Typical Deep Ocean Soils at Several Stress Levels and Time Periods

Type of Soil	Time Period (yr)	Long-Term Creep Strain (%) at Following Deviator Stress Levels[a]		
		0.3	0.5	0.7
Pelagic Clay	1	0.7	1.7	4.1
	2	0.8	1.9	4.5
	5	0.9	2.2	5.1
	10	1.0	2.4	5.6
Calcareous	1	0.3	1.1	3.5
Ooze	2	0.3	1.2	3.8
$w_c \approx 91\%$[b]	5	0.4	1.3	4.1
$\sigma_c \approx 15$ kPa[c]	10	0.4	1.4	4.4
Calcareous	1	0.3	1.2	5.3
Ooze	2	0.3	1.4	6.2
$w_c \approx 86\%$[b]	5	0.4	1.7	7.6
$\sigma_c \approx 38$ kPa[c]	10	0.5	2.0	8.9

[a] $\dfrac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f}$ where $(\sigma_1 - \sigma_3)$ is the principal stress difference and $(\sigma_1 - \sigma_3)_f$ is the principal stress difference at failure.

[b] w_c = water content.

[c] σ_c = consolidation stress.

Table 2. Summary of Test Soils Characteristics

Characteristics	Soil 1 Tests 1-7 (CEL)	Soil 2 Tests 8-9 (CSULB)	Soil 3 Tests 10-19 (CEL)	Soil 4 Tests 20-26 (UM)
Specific gravity	2.7	2.7	2.6	2.7
Liquid limit,%	53	23	32	115
Plasticity index,%	22	6	8	61
Unified Soil Classification	MH	CL/ML	ML	CH

Table 3. Summary of Short-Term Anchor Tests

Test	Anchor Diameter (mm)	D/B	Undrained Shear Strength (kPa)	Apparent OCR	Breakout Load, N	Calculated Holding Capacity Factor, $\overline{N}_c$
CEL						
1	100	5	2.04	2.1	245	14.8
2	100	5	1.58	1.6	215	16.7
3	100	5	1.51	1.6	205	16.7
4	100	5	1.70	1.8	195	14.2
5	100	5	1.65	1.7	220	16.7
6	100	5	0.92	1	135	17.8
7	100	5	1.23	1.3	170	16.8
CSULB						
8	76	5	0.81	1	49	13.2
9	76	30[a]	5.60	1	350	13.8
CEL						
10	50	1.5	0.20	1	6.7	16.4
11	50	1.5	0.53	2	14	13.2
12	50	1.5	1.05	4.5	27	12.8
13	50	1.5	1.35	10	44	16.2
14	50	3	0.29	1	8.1	13.7
15	50	3	1.10	2	36	15.9
16	50	3	1.35	4.5	43	15.8
17	50	3	2.07	10	71	17.0
18	50	6	0.48	1	13	13.4
19	50	6	1.23	2	35	14.0
UM						
20	76	2.1	2.8	NA	16	6.4
21	76	2.1	6.2	NA	51	9.2
22	76	2.1	4.8	NA	42	9.8
23	76	4.1	9.7	NA	125	14.6
24	76	4.1	7.6	NA	92	13.7
25	76	6.1	8.3	NA	115	15.7
26	76	8.1	11.7	NA	161	15.5

[a]Achieved with surcharge pressure.

Table 4. Summary of Single Increment Long-Term Tests

Test	Soil	Anchor Diameter (mm)	D/B	Applied Load as a Percent of the Short-Term Capacity
CEL				
1	1	100	5	60
2	1	100	5	44
3	1	100	5	48
CSULB				
4	2	76	5	25
5	2	76	5	50
6	2	76	5	75
7	2	76	~30[a]	25
8	2	76	~30[a]	50
9	2	76	~30[a]	75

[a]Simulated with a surcharge.

Table 5. Summary of Test Parameters for Multi-Increment Long-Term Model Tests

Test No.	D/B	OCR	Estimated Short-Term Holding Capacity, N	Ultimate Long-Term Holding Capacity, N	Ratio
1	1.5	1	5.5	5.3	0.96
2	1.5	1.8	14	6.7	0.48
3	1.5	4.9	31	11	0.36
4	1.5	36	115	38	0.33
5	3	2.7	26	20	0.77
6	3	4.1	52	40	0.77
7	3	10	81	53	0.67
8	6	1	10	20	2.0
9	6	5	73	135	1.8
10	6	9.3	84	195	2.3

Table 6. Soil Indicies at Mare Island Test Site

Soil Index	Value
Specific Gravity	2.7
Liquid Limit, %	100
Plasticity Index, %	50
Sensitivity	4
Unified Soil Classification	MH

Table 7. Summary of Mare Island Long-Term Field Tests

Test Identification	Fluke Penetration (m)	Keying Load (kN)	Keyed Depth (m)	Maximum Long-Term Load (kN)	Depth at Failure (m)
D	3.6	6.9	2.7	8.7	1.4
F	3.5	7.0	2.6	5.8	1.9
G	3.4	6.2	2.7	4.1	NA
I	3.5	7.1	2.8	6.7	2.3
J	3.6	NA	NA	8.3	1.8
K	3.2	7.1	2.8	3.9	2.1

Table 8. Comparison of Estimation Methods With Long-Term Model Test Results

Test Data No.	Test Data D/B	Test Data OCR	Measured Capacity, N	Predicted Capacity (Meyerhol and Adams), N	Predicted Capacity (Vesic), N	Estimated Short-Term Capacity, N
1	1.5	1	5.3	4.9	3.6	5.5
2	1.5	1.8	6.7	5.3	4.0	14
3	1.5	4.9	11	6.8	5.6	31
4	1.5	36	38	21	20	115
5	3	2.7	20	25	17	26
6	3	4.1	40	27	19	52
7	3	10	53	29	25	81
8	6	1	20	124	67	10
9	6	5	135	102	80	73
10	6	9.3	195	107	93	84

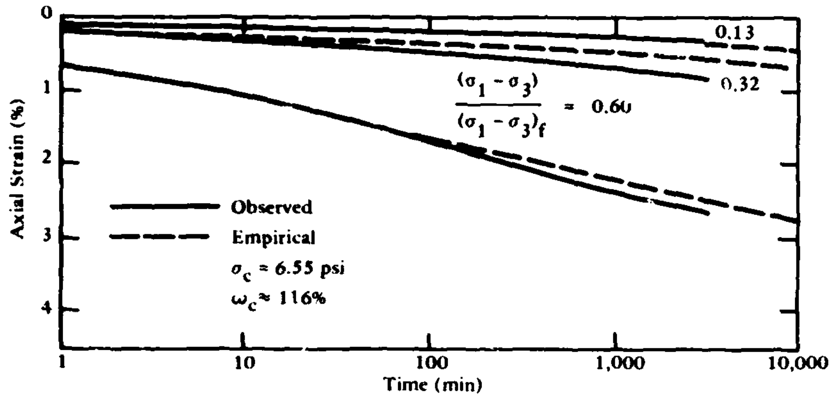

Figure 1. Comparison of observed and empirical strain versus time of a pelagic clay in undrained shear creep at different shear stress levels.

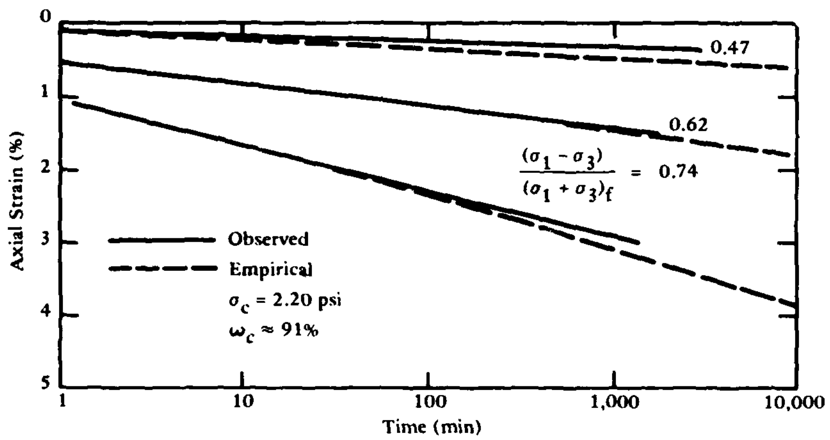

Figure 2. Comparison of observed and empirical axial strain versus time of a calcareous ooze in undrained shear creep at different shear stress levels.

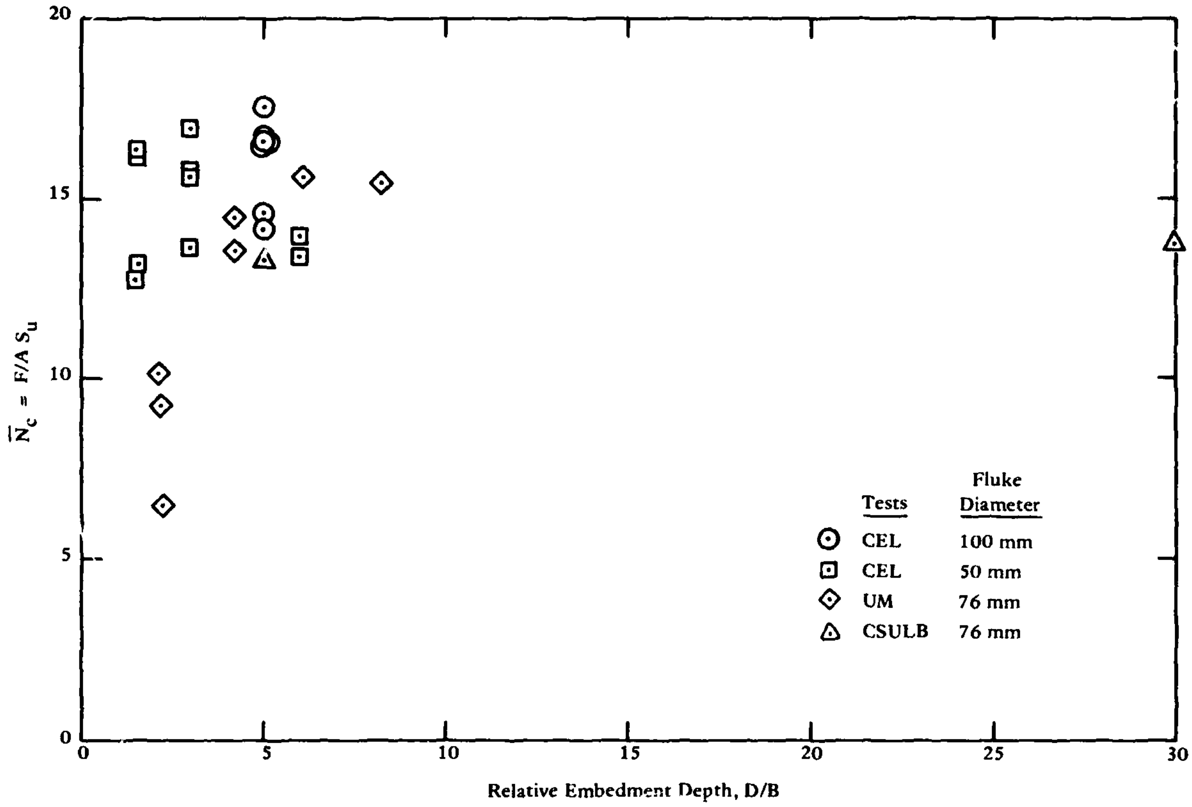

Figure 3. Plot of breakout factor $\overline{N}_c$ versus relative embedment depth for short-term tests.

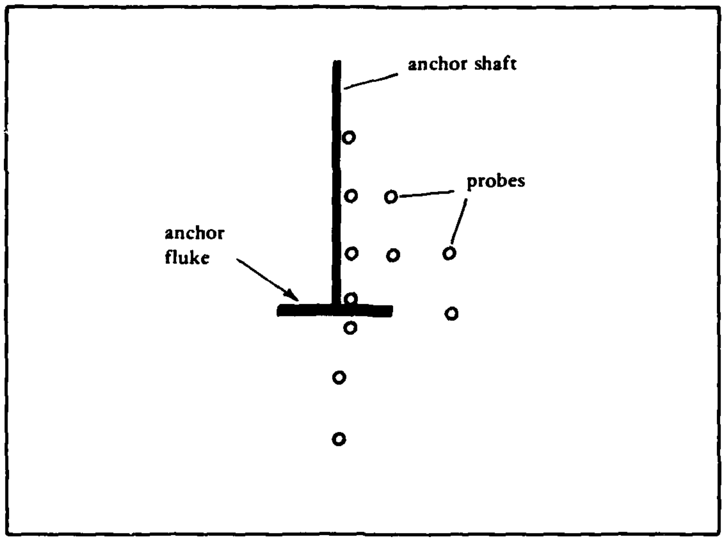

Figure 4. Location of pore water pressure measurement probes for tests 3, 4, and 5 relative to the anchor fluke.

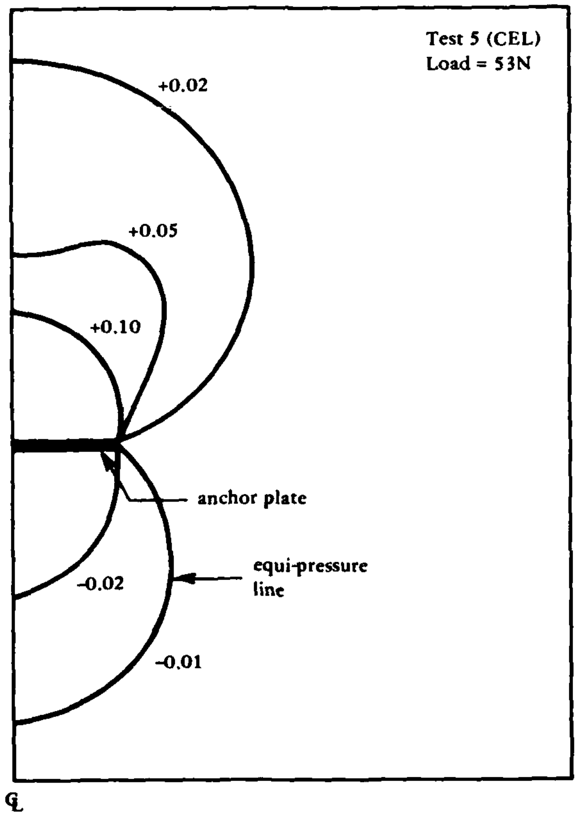

Figure 5. Pore water pressure distribution around anchor plate normalized by the pressure on the plate.

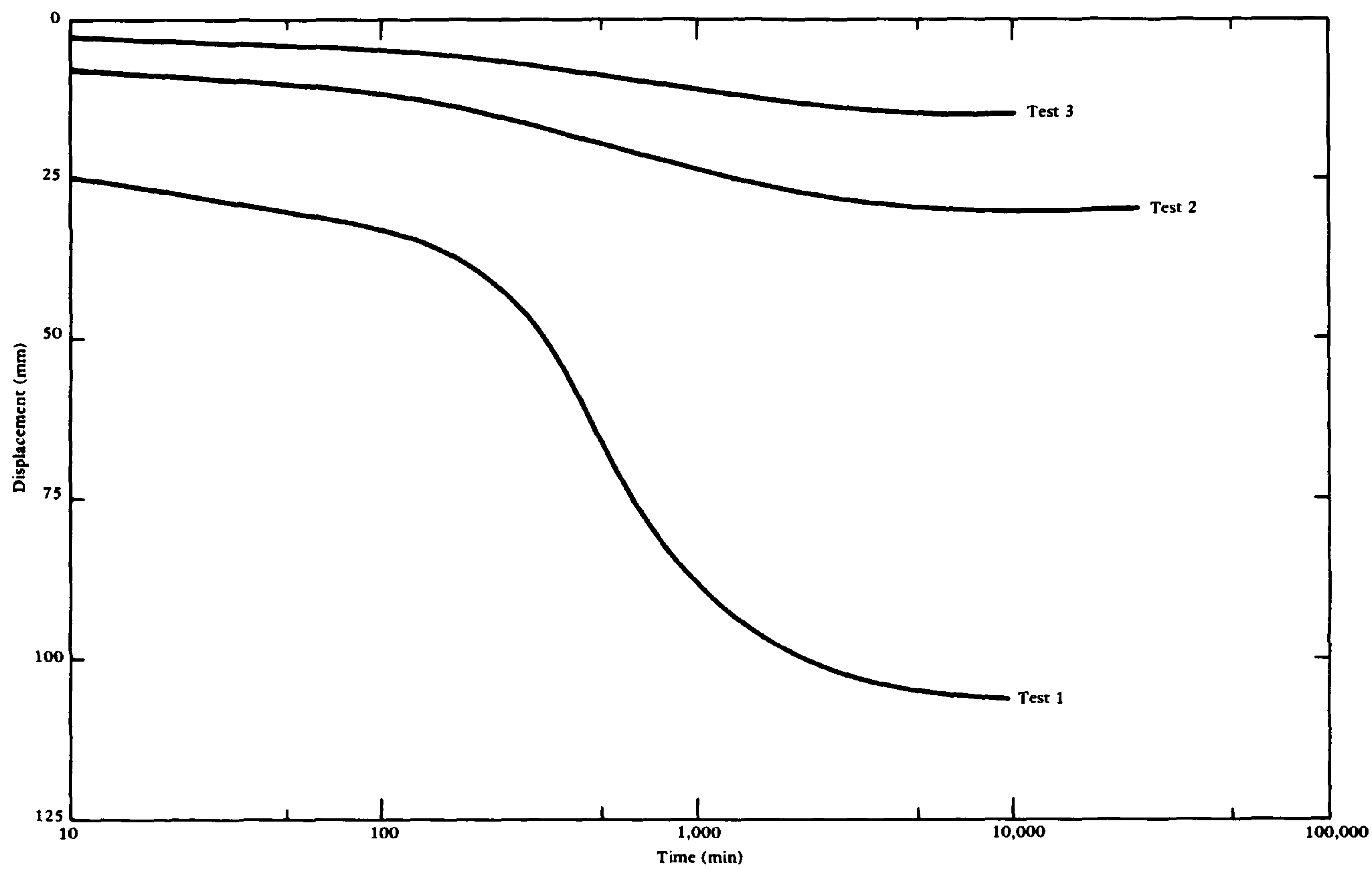

Figure 6. Displacement versus time for single-increment long-term model anchor tests 1 through 3.

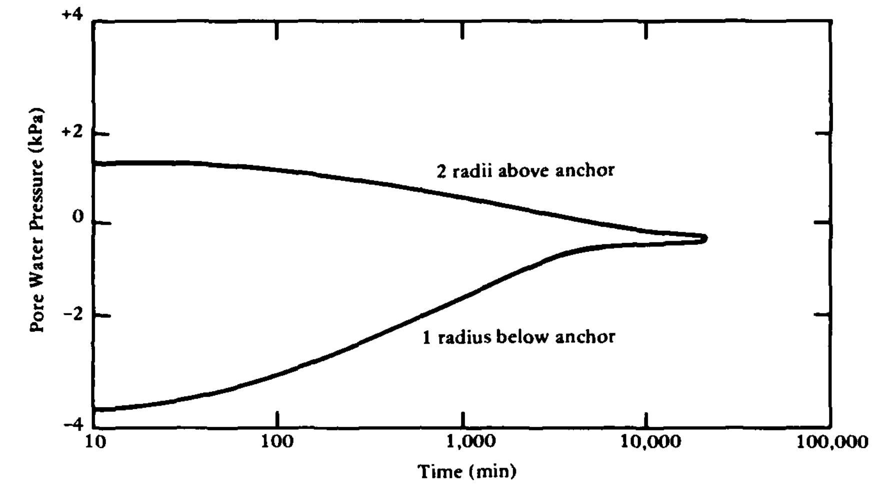

Figure 7. Pore water pressure response versus time during a long-term test at two locations near a test anchor.

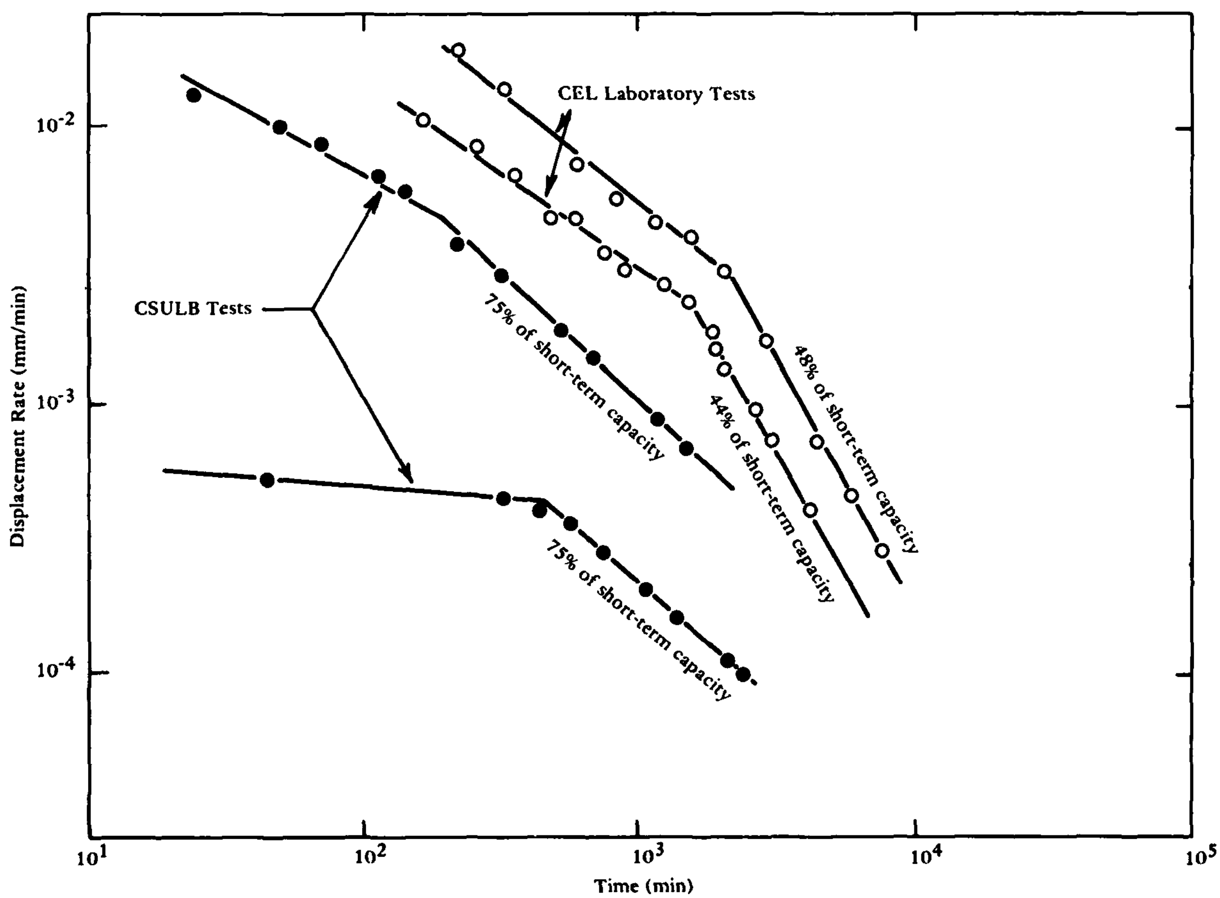

Figure 8. Rate of displacement versus elapsed time.

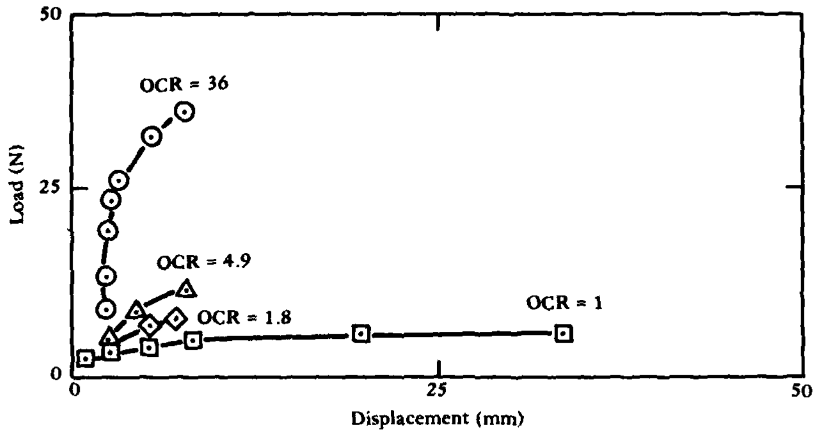

Figure 9. Displacement versus load for multi-increment long-term tests at D/B = 1.5.

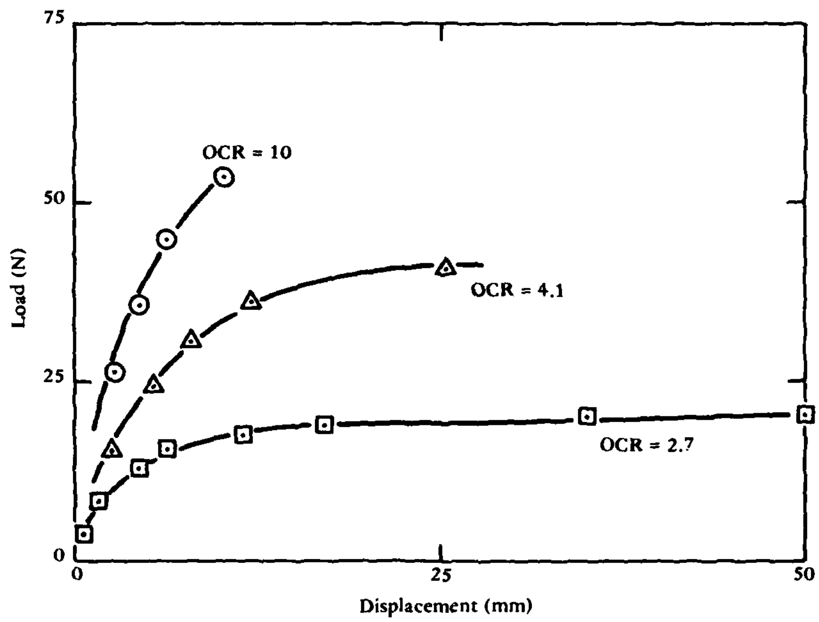

Figure 10. Displacement versus load for multi-increment long-term tests at D/B = 3.

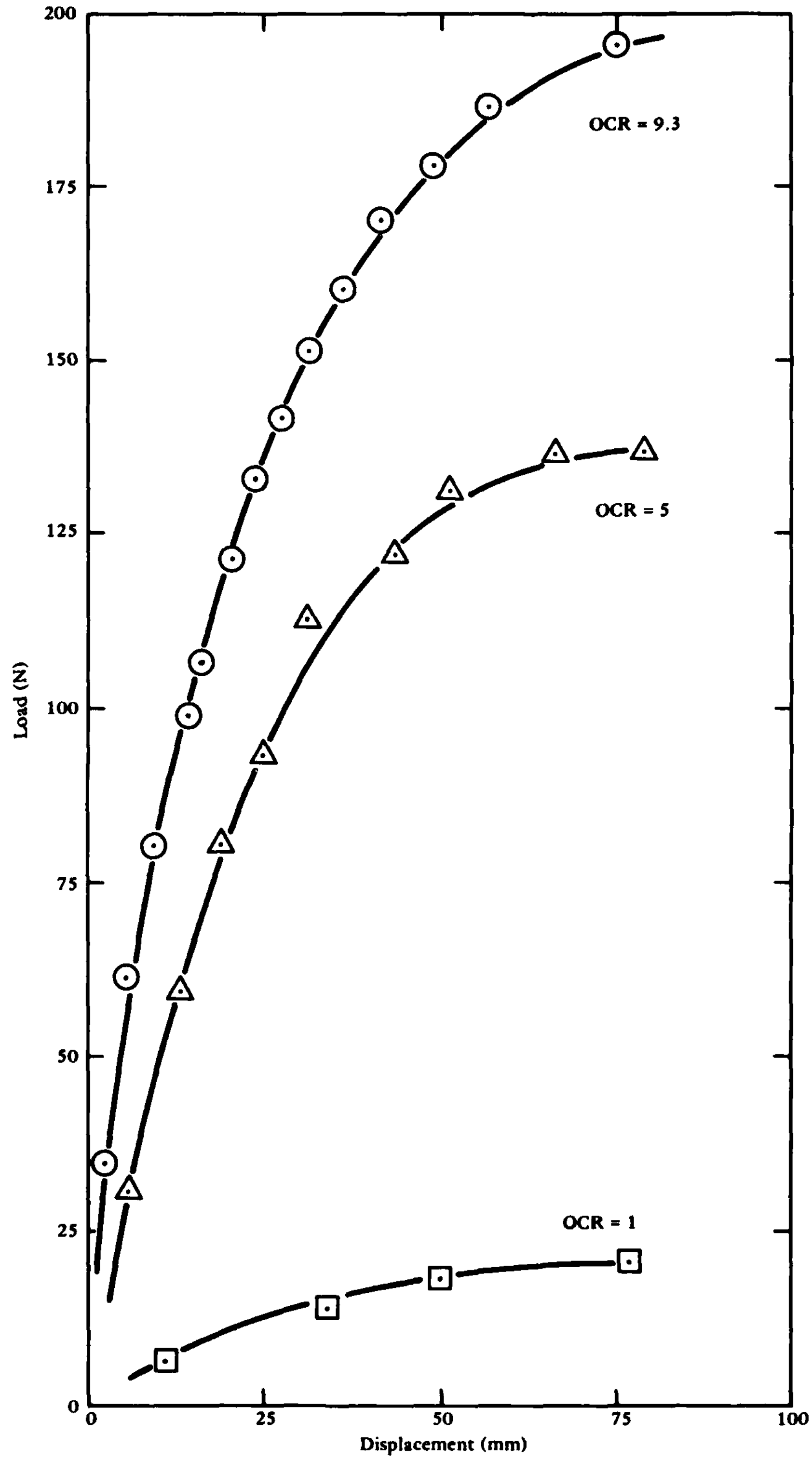

Figure 11. Displacement versus load for multi-increment long-term tests at D/B = 6.

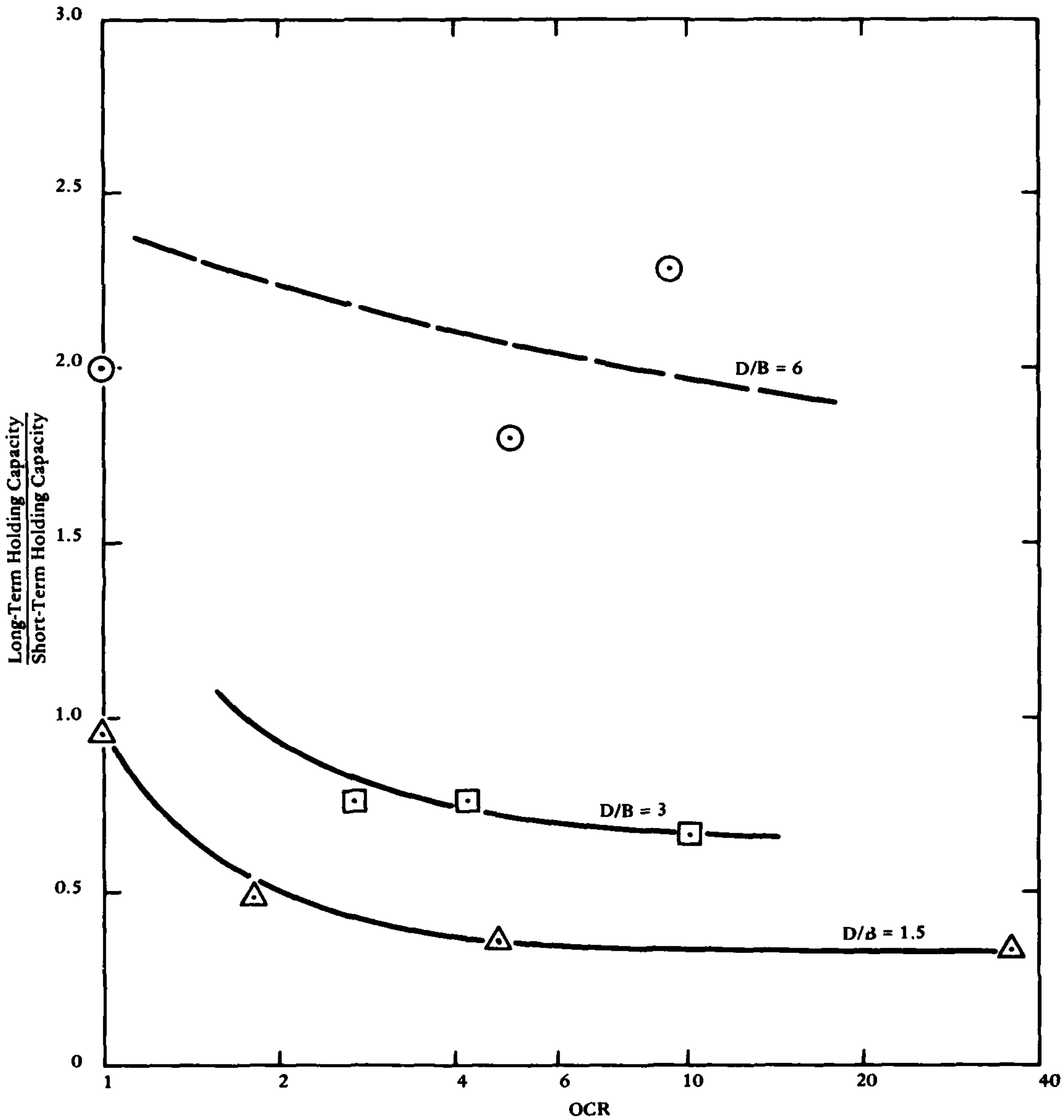

Figure 12. Overconsolidation ratio versus the ratio of long- to short-term holding capacity for different embedment ratios.

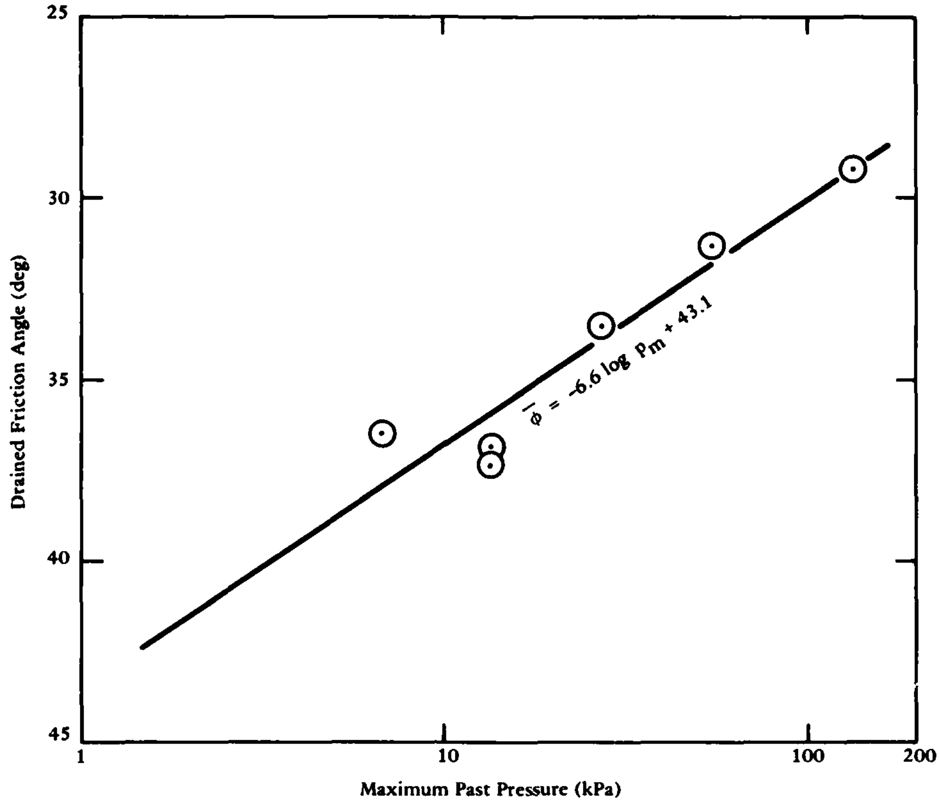

Figure 13. Drained friction angle versus maximum past pressures for soil used in multi-increment long-term model anchor tests.

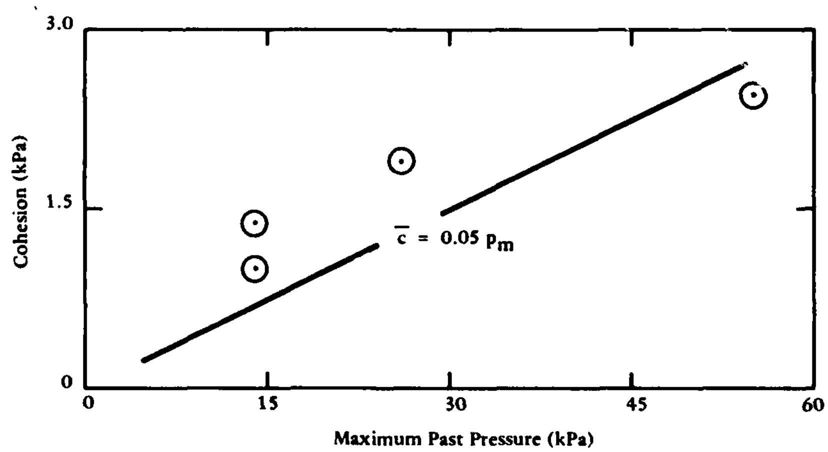

Figure 14. Cohesion versus maximum past pressure for soil used in multi-increment long-term model anchor tests.

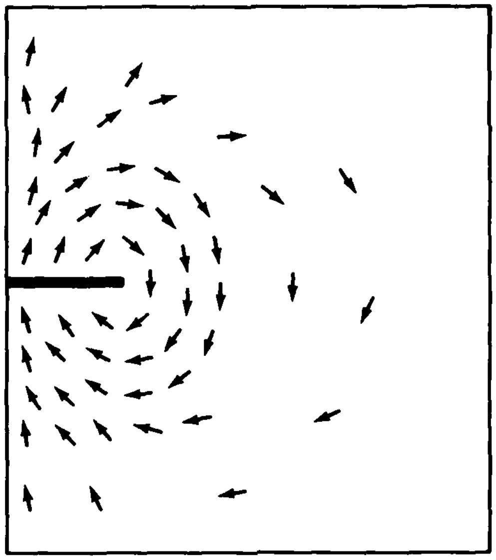

Figure 15. Diagram of direction of fluid flow from analytical model at D/B = 10 in normally consolidated sediment.

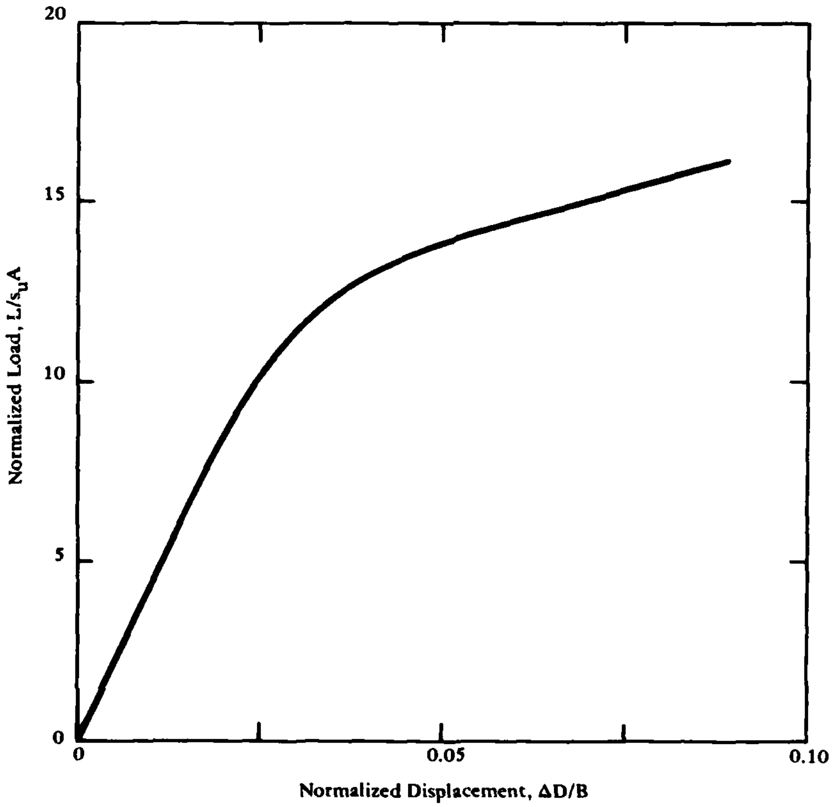

Figure 16. Load displacement curve determined with finite element model for cohesive soil with suction.

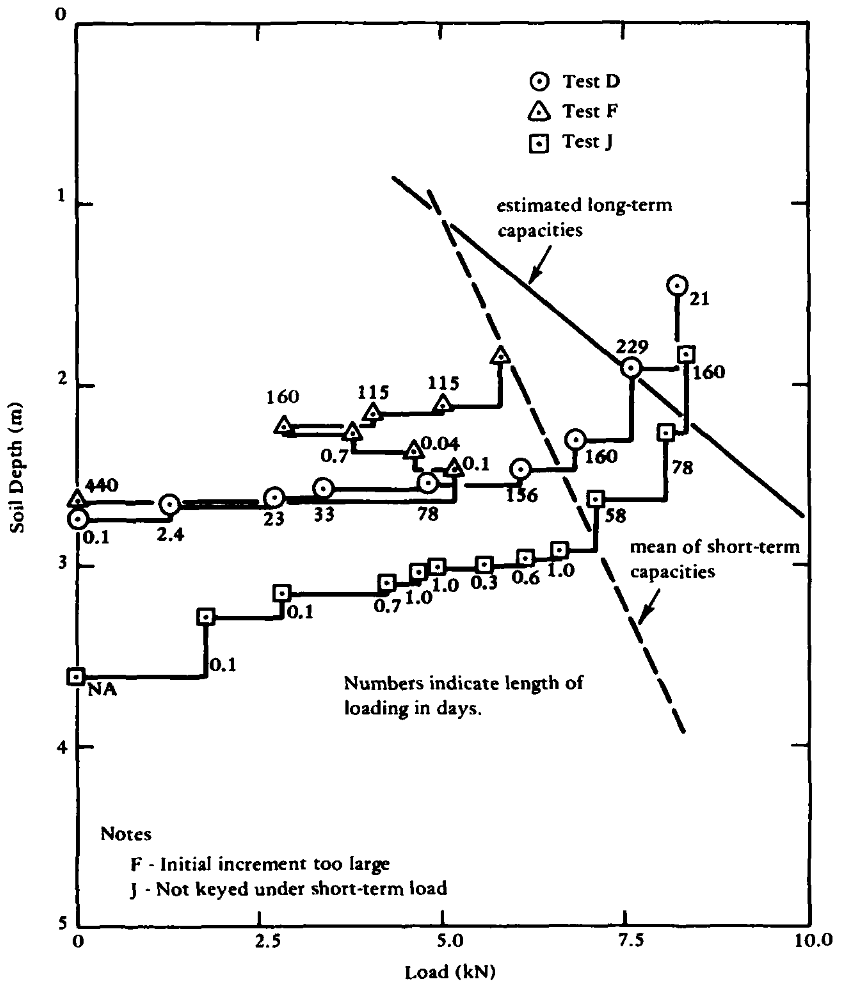

Figure 17. Long-term tests at Mare Island during which the short-term capacity was exceeded under long-term loading.

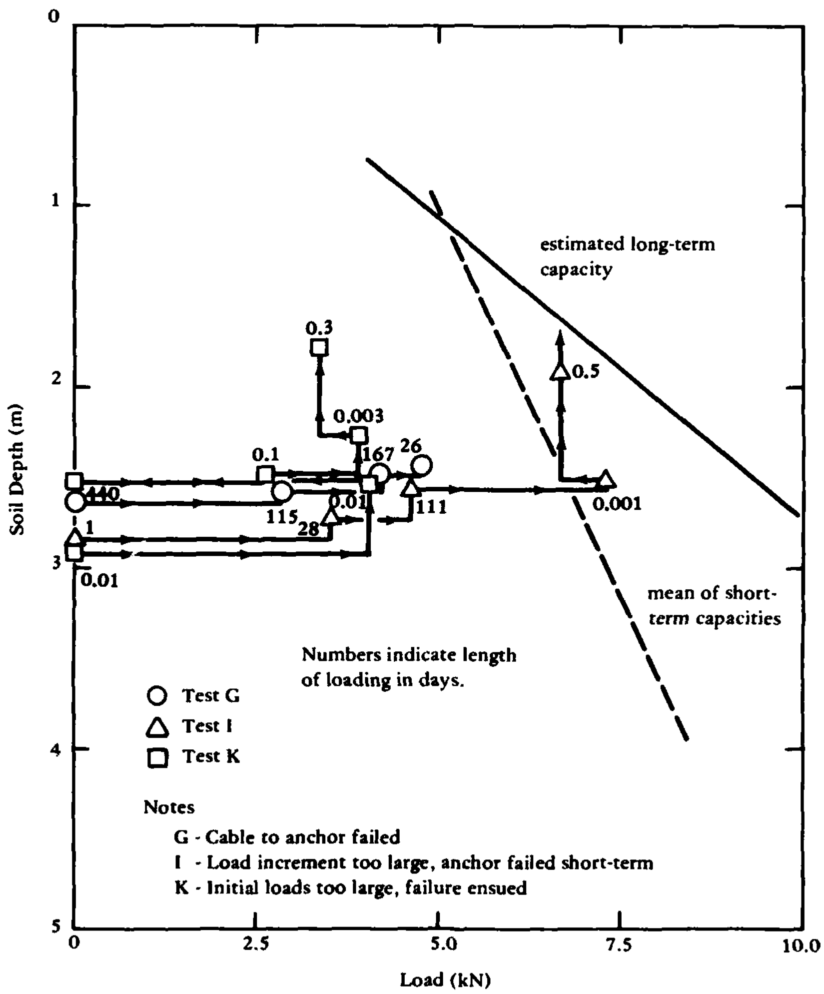

Figure 18. Long-term tests at Mare Island during which the short-term capacity was not exceeded under long-term loading.

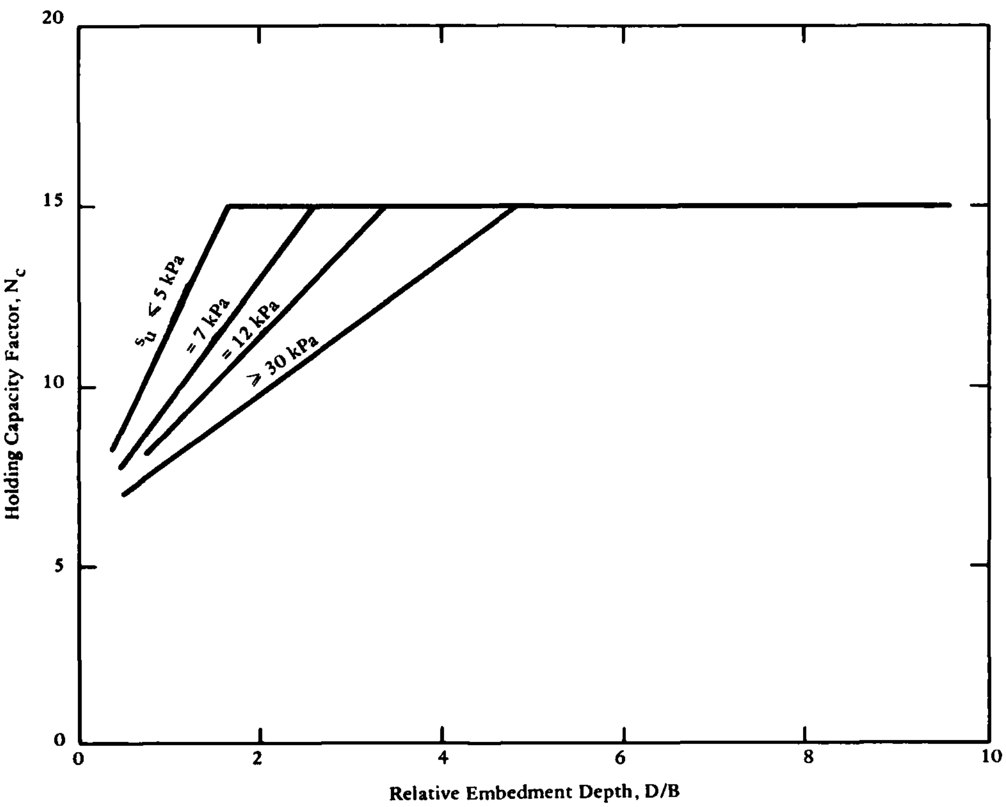

Figure 19. Short-term holding capacity factors for ideal conditions in cohesive soils under short-term loading.

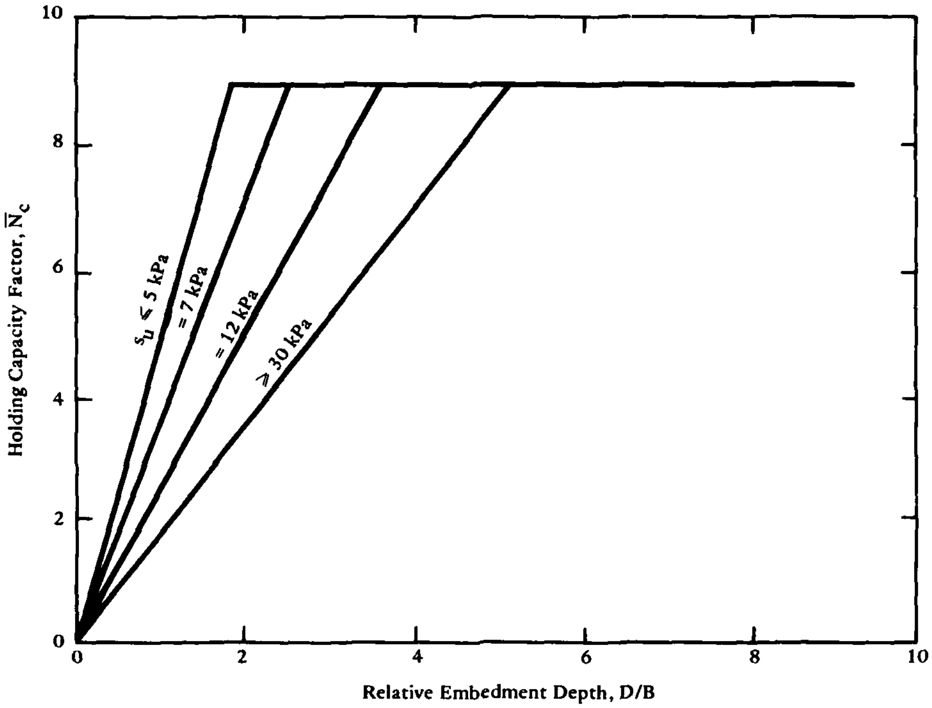

Figure 20. Holding capacity factors for cohesive soils under long-term loading.

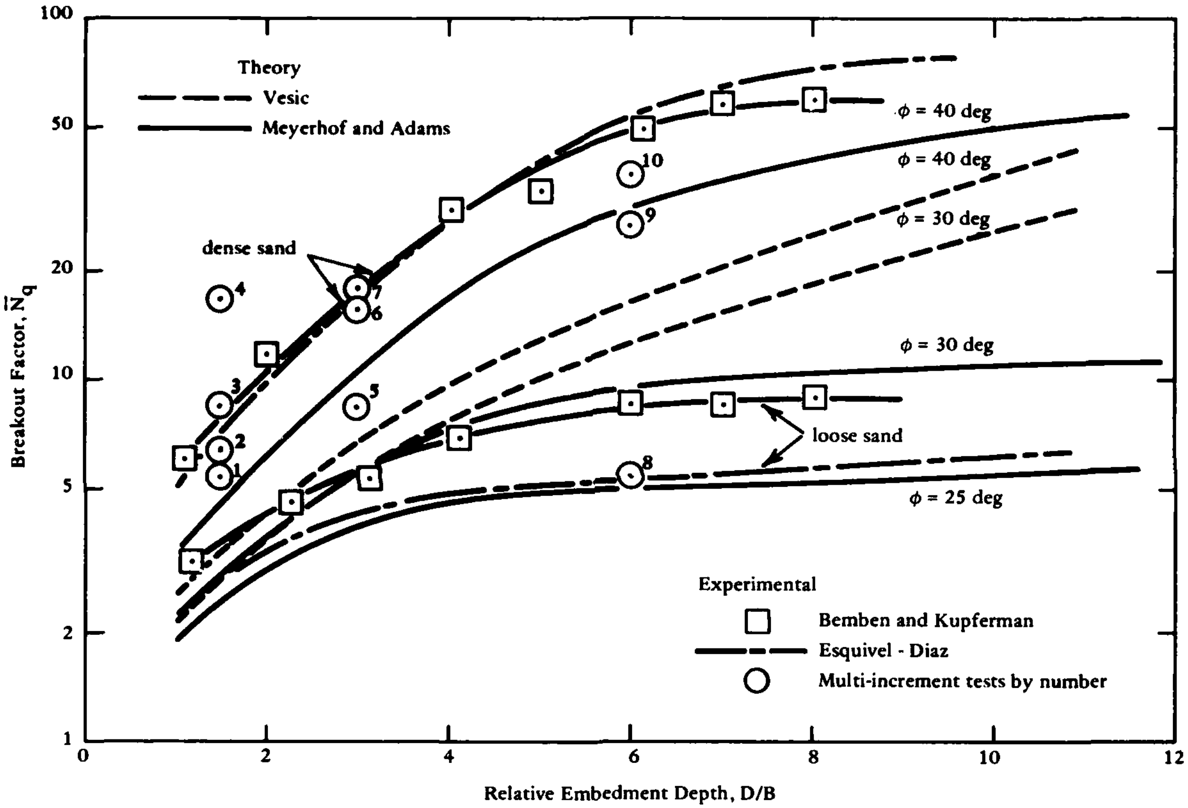

Figure 21. Comparison of Vesic's breakout factors with Meyerhof and Adam's breakout factors.

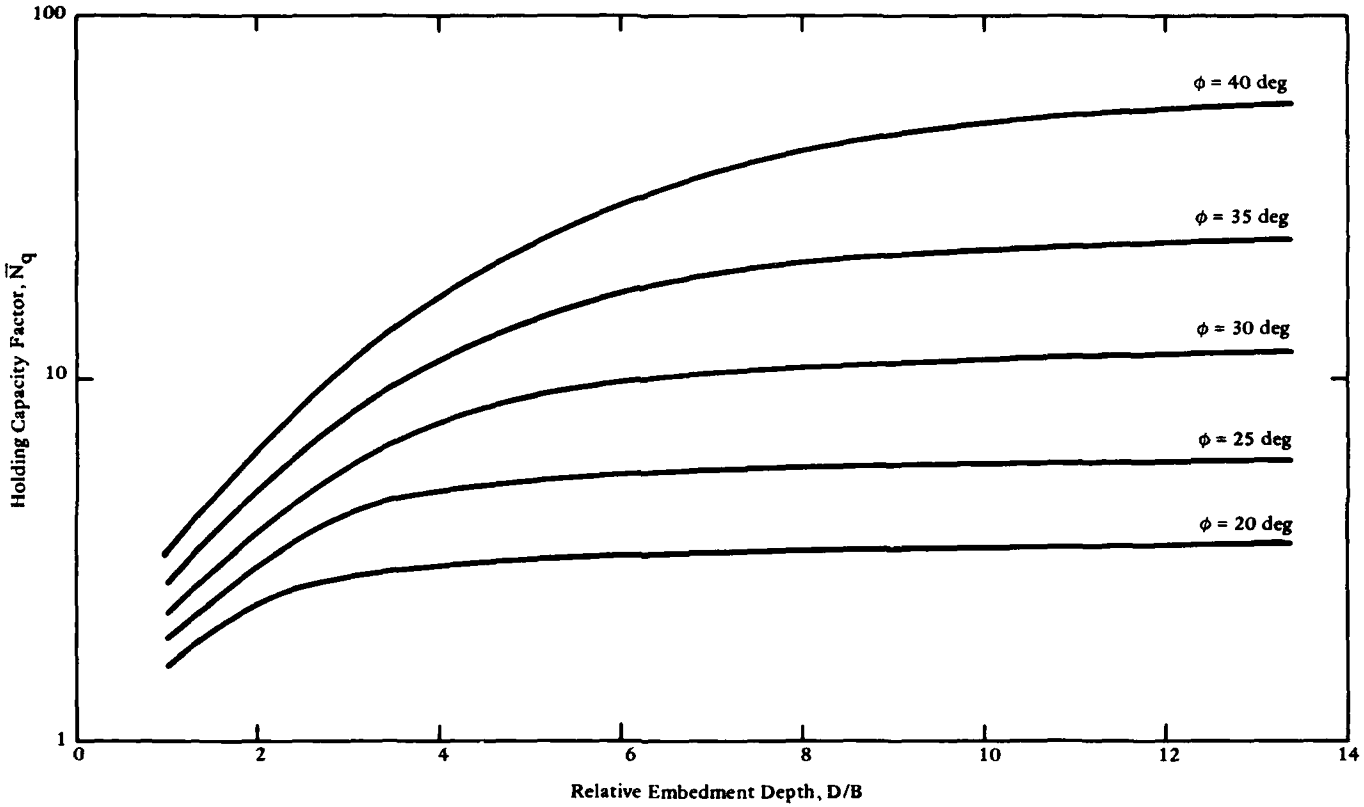

Figure 22. Holding capacity factors for drained analysis.

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