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INTRODUCTION

Naval Construction Battalion (NCB) engineers and constructors are responsible for building and modifying advanced bases so that the Navy can carry out its mission. This building activity is usually outside the continental United States and in some cases must be accomplished quickly and without a thorough siting program. While many types of construction are undertaken, particular interest is shown for waterfront construction of permanent and advanced bases that require pilings. Because of the remoteness of many construction sites and the need to build quickly, most of these bases are planned and designed using pre-engineered drawings contained in the Advanced Base Functional Components system (NAVFAC (1982)). However, the foundation design must be site specific and therein lies a potential weakness.

The thin layer of soil that covers the earth is highly variable making pre-engineered foundations difficult to achieve. Foundations that rest on or are embedded in soils require soil surveys, testing, and analyses so that specific designs can be made to accommodate the variety of conditions encountered. The Navy has prepared two manuals to assist the NCBs in designing foundation systems (NAVFAC (1971) and Rocker (Ca 1984)). Application of the design procedures provided by these manuals is successful when the soils encountered behave similar to terrestrial soils. However, in the equatorial regions of the earth, a region of interest to the Navy, the soil is calcareous and engineering experience has shown that calcareous soils behave quite differently than terrestrial soils. In particular, pile foundations in calcareous soils have not always performed well even when designed with higher factors of safety than normally applied to designs in noncalcareous soils. The Naval Facilities Engineering Command has funded a project to investigate the performance of piles in calcareous soils, because of the expected additional construction of pile supported structures in equatorial regions. The intent of this program is to improve the technology related to pile behavior in calcareous soil and enhance the Navy's ability to quickly and efficiently construct pile-supported facilities.

BACKGROUND

Calcareous soils are composed of calcium carbonate. They are predominantly sedimentary and differ from terrigenous soils in several ways (McClelland Engineers, Inc., 1980; Noorany, 1982a; and Datta et al., 1981). Calcareous soils are products of biological activity, having more intra-granular voids, are easier to crush, and are more susceptible to post-depositional physical and chemical alterations. Calcareous deposits can have significant spatial variations and discontinuities such as cavities, solution channels, and highly cemented zones.

Several researchers (Datta et al., 1979, 1980, 1981; Agarwal, 1977; and Demars et al., 1976) have identified parameters that affect the engineering behavior of calcareous soils. These parameters are carbonate content, crushability, degree of cementation, index properties, and geologic processes. These parameters are interrelated and none of them alone provide a unique relationship to engineering behavior.

There is no classification system that adequately characterizes the engineering behavior of calcareous sediments; several have been proposed (Noorany, 1982a; Demars et al., 1976; Demars, 1982; Datta et al., 1981, 1982; Beringen et al., 1982). Datta and others suggested that is it premature to propose engineering classification systems; cited as reasons, among others, is that cementation and susceptibility to crushing cannot be quantified. Beringen and others proposed using cone penetration test (CPT) data to qualitatively measure cementation as a function of cone resistance in conjunction with other classification data. While it is apparent that progress in classification is being made, it is equally apparent that years of data gathering will be required before a comprehensive and reliable classification system will evolve.

A summary of engineering aspects of calcareous soils that are likely to be relevant to pile behavior is provided:

Index Properties

- Calcareous soils have lower densities and higher intra-particle voids than terrigenous soils.

- Calcareous soils contain soft calcium carbonate minerals and their grains are softer than quartz or silica sand.

- The specific gravity, G_s, of calcareous sediments is usually higher than terrigenous soils. For example, the G_s values for the calcareous sands from Florida and Guam (Noorany, 1982b) are about 2.8 or more, whereas the G_s value for quartz is about 2.65.

Compressibility and Crushability

- Calcareous soils are more compressible than terrigenous soils; their compressibility results from grain crushing and the collapse of grain-structure, therefore, volume changes are usually permanent.

- Coarser-grained calcareous sediments show a more significant degree of degradation and grain crushing than finer-grained sediments.

- Grain crushing and the collapse of grain-structure can be induced by applying either confining or shearing stresses.

- Sediments with high carbonate content do not compress to as low a final void ratio as sediments with low carbonate content.

Strength Properties

- Calcareous soils have a higher internal friction between grain-to-grain contact than terrigenous soils (Horne and Deere, 1962).

- Friction angles of calcareous soils decrease with increasing confining pressure. This reduction appears to be the result of grain crushing (Datta et al., 1980). Increasing grain crushing induces decreasing shearing resistance until a limiting value of shear resistance is reached.

- Shearing can cause grain crushing and volumetric contraction.

The summary of engineering behavior of calcareous soils represents the state-of-knowledge at the beginning of this project. It is apparent that the knowledge is meager and not well related to piles.

STATE-OF-THE-ART PILE INSTALLATION PRACTICE

Piles used nearshore and offshore are usually open-ended, steel pipe piles. These piles are popular because they can be easily applied to achieve long length, offer a good strength-to-weight ratio, minimize soil disturbance during installation, and minimize driving resistance. Also, it is more economical to install fewer long piles than more short piles.

Several techniques are currently used for installing piles in calcareous soils. These techniques are listed below in order of preference (frequency of occurrence):

- Driving with impact hammer (driven piles)

- Drilling and grouting (drilled and grouted piles)

- Driving, drilling, and grouting combination

- Drilling an enlarged base, then grouting (belled piles)

- Driving with vibratory hammers

Installing piles with an impact hammer is common because it is simple to use and has been successfully used for a wide variety of soil conditions.

Drilled and grouted piles are often used where rock layers and highly cemented strata are present in the sediment profile and piles cannot be driven to the final design penetration. In these cases, a pile cavity is drilled to final penetration and then a pile is inserted and grouted to form a composite pile. A drilled and grouted pile could be used from the outset. Pile driving has caused significant degradation of skin resistance in calcareous soils (Angemeer et al., 1975). The recently completed pile-supported POL pier at the Navy's Diego Garcia Advance Base in the Indian Ocean used driven, open-end pipe piles in calcareous sediments and experienced severe soil degradations. Construction personnel reported that piles would free drop as much as 6 feet during driving. As a result of these experiences, drilling and grouting the piles may be preferred instead of using an impact hammer to install the piles in calcareous soils.

Belled pile foundations have been used for a tanker terminal project in Saudi Arabia where calcareous sediments are predominant (Burt and Harris, 1980). This method took advantage of the high end bearing resistance in carbonate rocks or highly cemented calcareous soils.

High capacity, low frequency vibratory hammers were used to drive piles into a calcareous sediment in Saudi Arabia (Fugro Ltd., 1982). Installing piles by such a method appears promising, however, little or no data are available to evaluate the effect of this installation method on the load carrying capacity of piles.

Axial Pile Capacity

There are minimal data on the axial load capacity of piles in calcareous soils. The ultimate axial capacity is determined by the sum of skin friction resistance, Q_s, and end (tip) bearing resistance, Q_p, and can be expressed as follows:

$$Q = Q_s + Q_p = f\,A_s + q\,A_p \tag{1}$$

where: f = unit skin friction

A_s = side surface area of pile, which is in contact with sediment

q = unit end bearing capacity

A_p = cross-sectional area at pile tip

Very few experimental studies have been conducted to investigate the behavior of piles in calcareous sediments. Studies that are available tend to concentrate on skin friction. The results of static tensile pile load tests for driven piles and grouted piles in calcareous soils are summarized in Table 1. Angemeer et al. (1973, 1975) indicated that the axial capacity of driven piles in calcareous soils can be lower than that of piles driven in terrigenous soils. They also found that skin friction for driven piles varied from site to site and thought this variation probably reflected differences in crushability, cementation, and density of the calcareous soil. They concluded that calcareous soils are extremely sensitive to crushing as evidenced by grouted piles yielding a frictional capacity on the order of 3 to 5 times that for driven piles. They applied cyclic loads to a grouted pile to measure the threshold pile friction resistance and observed no significant loss in frictional capacity after about 90 cycles. Contradictory observations, though, have been reported in King et al. (1980) when performing small-scale pile segment friction tests in-situ. King's large displacement cyclic tests (i.e., each cycle displacing the pile to the maximum resistance) showed that the frictional resistance reduced substantially. A disparity in threshold frictional resistance, therefore, exists due to the sparsity of data and inconsistent testing procedures. End bearing is even less studied and very little data on end bearing capacity are reported. Figure 1 shows a plot of q determined from field pile tests in chalk and weak cemented calcareous soils versus standard penetration test (SPT) resistance value, N. The figure shows a wide scatter of data, indicating that the SPT, a popular in-situ test device, may not be a good tool for determining the q value for piles in calcareous soils as the CPT.

Pile Design Methods

The design of pile foundations is far more uncertain in calcareous soils than in terrigenous soils. Unfortunately the information available cannot be fully explained by conventional theory and does not aid in making necessary judgments. Where important facilities are planned, it is a normal practice to conduct pile load tests to confirm load predictions. These tests are costly, time-consuming, and impractical for most projects. Instead, large factors of safety are normally introduced to account for design uncertainty. However, this approach leads to costly over-design in many cases and unsafe designs in other cases.

Design practice for determining the ultimate axial capacity of piles in calcareous sediments can be divided into the following categories:

1. Use of conventional theory with modifications to account for certain aspects of calcareous soils.

2. Empirical correlation with penetration resistance during driving.

3. Correlation with in-situ tests.

4. Correlation with full scale pile load tests.

The features of these methods are summarized in Table 2.

Conventional Theory. The conventional theory for predicting axial capacity of piles in terrigenous soils is used with modifications to account for various engineering aspects of calcareous soils. Table 3 shows design parameters presently used for estimating axial capacity of driven piles in calcareous sands; parameters for silica sands are also shown for comparison. The parameters can be used to show that predicted pile capacity for calcareous sands is only about one-third of that for silica sands for similar conditions. Similarly, for piles subjected to tensile loading, the capacity predicted for calcareous soils may be only 20% of that predicted for silica sands. Unfortunately, the design parameters in Table 3 do not relate the degree of cementation to measurable soil properties.

Agarwal et al. (1977) recommended design parameters based on carbonate content that are significantly higher than those in Table 3. The use of these parameters would lead to higher predicted capacities. Datta et al. (1980) recommended coefficients of lateral pressure for calculating skin frictions that would also lead to higher predicted skin friction. These procedures have not found favor because of the uncertainty in designing piles in calcareous soils.

The design of drilled and grouted piles in calcareous sands appears to be so site-specific that recommended design parameters do not appear in the literature. The lack of progress in the development of general design procedures is undoubtedly due to the limited amount of experimental work.

Empirical Correlation With Resistance To Driving. The empirical correlation approach correlates the driving resistance to the axial capacity of piles in accordance with an empirical formula. A variety of these formulas are available, among them is the Engineering News Record formula:

$$W_H H = R (S + 0.1) \qquad (2)$$

where: W_H = weight of the driving ram
H = distance of the ram travel
R = resistance of the pile to driving
S = distance the pile tip penetrates the soil

Each of the formulas equate the driving energy with the resistance the soil offers to pile penetration. This approach was used in the pier constructed at Diego Garcia. Based on Equation (2), the determined shear strength property from selected soil samples and a pile load test, a blows per foot criteria for pile driving procedures and the estimated pile embedment depth was established to reach the designed pile capacity. The estimated depth was about 65 feet. Based on the unpublished pile driving record, many of the piles required two to three times this depth to meet the blow count criteria. The results of the work at Diego Garcia, therefore, indicated that the driving resistance is not a good indication for predicting pile capacity in calcareous soils.

Correlation With In-Situ Penetration Tests. Standard penetration resistance has been used for calculating pile capacity in chalk. As indicated in Figure 1, this correlation yields a wide scatter of results. In addition SPT results are not reproducible and as a result this method must be used with caution.

Full-Scale Pile-Load Test. The in-situ, full-scale pile-load test is the best method for taking site-specific parameters into consideration. However, calcareous sediments vary significantly in composition and engineering behavior between nearby locations and is difficult to extrapolate results. Thus, the number of pile load tests required for calcareous soils may be larger than for terrigenous soils. This can be very costly. Research and development in understanding soil-pile interaction might reduce the number of tests required.

LABORATORY TESTS OF SOIL-PILE INTERACTION BEHAVIOR

Purpose and Approach

The preceding background has shown the lack of knowledge of calcareous soils, their engineering behavior, and designing piles for use in calcareous soils.

The problem confronting the Navy is that the design of piles in calcareous soils cannot be done with confidence and this poses a threat to the successful completion of expedient operations. Planning for these operations becomes extremely difficult when, for example, it is not known how much piling to send with an elevated causeway that must be constructed in order to support an amphibious operation.

A program for improving pile design capability in calcareous soils was presented by Bailard and McCarel (1981). They recommended a four-point program that included: laboratory and field studies to determine the geotechnical behavior of piles driven in calcareous sediments; development of improved pile technology for calcareous soils; and, development of geotechnical design procedures for piles used in calcareous soils. The corner stone of the program was a comprehensive research program aimed at gaining a better understanding of the geotechnical behavior of piles during inserting and loading in calcareous soils, and how this behavior differs from that in terrigenous soils. The goal of this program was to discover what specific failure mechanisms cause the widely divergent behavior. Particular attention was given to the relative importance of cementation, grain crushability, carbonate content, and crushed grain fraction in governing the behavior. Because of the difficulty and expense of conducting full-scale field experiments, the bulk of this work has been conducted in the laboratory. Later, verification of these results will be obtained by partial and full-scale field tests using instrumented piles.

The purpose of this report is to document the results of the laboratory investigations designed to determine the factors influencing pile behavior in calcareous soils and to examine alternative pile concepts for calcareous soils. A general investigation designed to bound the problem and provide guidance for the other investigations was performed by the Naval Civil Engineering Laboratory (NCEL). A study of frictional behavior, particularly in regard to crushed friction was performed by Dr. Iraj Noorany (Noorany, 1982b). The study of grain crushing and the study of alternative pile concepts were performed by Earth Technology Corporation (ERTEC, 1983a and b).

General Investigation

The background information clearly points out that an understanding of the calcareous soil-pile interaction and the ability to provide adequate pile design and installation guidelines or procedures is not available to the engineering community. NCEL conducted a series of laboratory experiments to gain a better understanding of the general behavior of the calcareous soil-pile interaction, and to bound the geotechnical parameters controlling this interaction. The investigators determined that a visual representation of the calcareous sand media during pile driving would meet these goals. By varying the soil media's consistency (i.e., density and degree of induration), the different behaviors of the calcareous sand during model pile embedment could be compared to a controlled sand (silica sand) having similar consistencies. The behavior difference and the influencing factors were more clearly defined.

Program

The model selection criteria used for this experiment required that the model be manageable in the laboratory and be free of as many laboratory induced effects as possible yet provide the desired results. A technique had to be developed that would allow a view of the change occurring in a soil mass as a pile was driven into the soil without influencing its behavior. The solution to the problem was found by using radiography. Radiography is expensive to use because of the equipment and the special accommodations required to conduct the experiment and provide against x-ray exposure. The method, therefore, is not commonly used. Nevertheless, a GE 250, 5 Ma, 210 kV power apparatus normally used for nondestructive testing was available at the Aeronautical Mechanical Prototype Support Branch at the Pacific Missile Testing Center, Point Mugu, Calif.

Radiography provides an image of an object subjected to x-rays. An exposed film, sensitive to x-rays, shows the internal structure of the object and highlights areas of high density. High density material will either absorb the x-ray or slow down their transmission time. Since the film is exposed to an x-ray source for a predetermined length of time, the high density areas projected onto the film is contrasted relative to the surrounding material. After exposure no residue radiation is present since lead shielded walls surrounding the objective area absorbs all x-ray scatter instantly.

Applying the radiography principal to the calcareous sediment model study helped to select the laboratory components. They included a plywood box (6 by 12 by 24 inches deep), lead shot pellets, and a steel model pile (24 inches long by 1-1/2 inches diam). Other ancilliary equipment such as assorted driving weights, templates for lead shot placement, and a pile guide for vertical penetration were provided (see Figure 2).

Test Materials

Two test materials were used for this study: (1) calcareous sand and (2) silica sand. Calcareous sand is not abundant in the continental United States and the most likely area to find the material is in Florida. In the past, abundant supplies of the sand were available on beaches located in the Florida Keys as well as southernly beaches on the mainland. However, many of these beaches have been expanded with imported silica sand, thereby, contaminating the source. Also, silica sand is used on eroding beaches, which is another cause of contamination. Fortunately, a clean source of calcareous sand was located at the Key West Naval Air Station. An abandoned fort, located on the base, had rooms filled with 100% calcareous sand. Several cubic yards of the sand were shipped to

Port Hueneme after it had been examined and found to be pure. This sand has a specific gravity of 2.72 and a carbonate content of 86.17% with 1.45% organics. The grain size analysis is shown in Figure 3.

Silica sand was obtained from a commercial source. This sand was from Monterey and is referred to as Monterey sand. For the purpose of this report, the sand will be referred to simply as silica sand. The specific gravity for this sand is 2.65. The grain size analysis is shown in Figure 4.

Model Preparation Procedures

The program's philosophy was to bound the geotechnical parameters of the soil-pile interaction. The objective was to view the behavior of the two sands during pile driving. For each soil, two densities (a loose and tamped soil) and seven levels of cementation (0%, 0.25%, 0.50%, 1%, 2%, 4%, 8%) were used. Furthermore, a limited suite of samples were tested consisting of a 50-50 mixture of the two sands with the 0%, 1%, and 2% cementation levels.

Three parameters: (1) density, (2) cementation level, and (3) carbonate content, were viewed as important factors controlling the behavior of calcareous sand. First, density variations were used since engineering properties of terrestrial sands behave differently at different void ratios (density). For example, a dense sand undergoing shear loading will expand at failure while a loose sand will usually compress into a more compact state. Since this is a prominent phenomenon occurring with sand, the application of density variants appeared appropriate.

Second, the varying cementation levels were appropriate because calcareous sand deposits exist in uncemented and weak-to-medium cemented states. The use of the cement variants will dispel questions of how the soil-pile interaction changes with different stages of induration.

Last, as was stressed earlier (Demars et al., 1976), the possibility exists that carbonate content could be used as an index property. Thus, for purposes of bounding the effects of this property, the carbonate content was varied at three levels: 0%, 50%, and 100%.

The soil preparation procedure required that enough moisture was available in the mixture to hydrate the Type III cement. The calcareous sand was brought up to 19% moisture content (weight of water divided by weight of solids times 100) and the silica sand to 10% (these moisture contents remained the same for all the combination of soil samples). The reason for the disparity of moisture contents is because the calcareous grains have a high affinity for absorbing the interstatial water before hydration is allowed to commence, thus, additional moisture was needed. The amount of cement added to the mixture was measured as a percent of total sample weight. After mixing thoroughly, the soil sample was placed into the container as explained below and cured in a 100% humidity controlled environment for 7 days.

A raining technique, common to soil laboratory testing, was used for the high void ratio (volume of voids/volume of solids) soil placement. The prepared soil was placed in a retaining hopper above the container and released, and was atomized as it passed through a sieve screen. For this study, a disbursing box and a 1/2-inch Tyler screen were used. The soil was placed in 1-inch depth increments (i.e., lifts).

The container holding the soil was 6 by 12 by 24 inches high. The depth of soil was 18 inches leaving 6 inches available at the container top for pile positioning equipment.

The low void ratio soil placement procedure also used a disbursing box and a 1/2-inch Tyler sieve screen. In order to achieve a 1-inch higher density lift, additional soil was added and tamped by a 3- by 3-inch square wooden hammer. The following void ratios were obtained for the suite of soil models:

Model	High Void Ratio	Low Void Ratio
Calcareous Sand	2.0 ± 0.1	1.4 ± 0.1
Calcareous/Silica Sand	1.43 ± 0.01	0.94 ± 0.02
Silica Sand	0.75 ± 0.1	0.6 ± 0.1

A total of 18 inches of soil required 18 lifts. In between each even lift, 13 lead shot pellets were placed on the surface of the lift. Three pellets were placed, 1/2 inch apart, directly on the center line of the pile driving path with the center pellet on the center line of the pile. The remaining 10 pellets were placed symmetrically on each side of the pile in a geometric progression manner starting with 1/4 inch from the pile surface and stopping at 4 inches. The finished model, had eight layers of 13 lead shot pellets for a total of 104 discrete points. The size of the pellets was compatible with the median soil grain size determined from the grain size distribution curve (Figure 3).

Test Procedures

Thirty-four sand models were constructed and tested to determine the driving energy required to penetrate the sand and to monitor the vertical and horizontal movement of each lead shot pellet by radiography. Cured sand models were taken to the Aeronautical-Mechanical Prototype Support Branch at Point Mugu and set up in front of the x-ray equipment. A metal grid, for monitoring relative movement, was tacked to the back of the wooden container (model) and a pile guide secured over the soil. An x-ray* was taken showing the initial position of the lead pellets. The pile driving mechanism was placed in the guide and a selected weight fell 30 inches and impacted the top of the pile. A second x-ray was taken after the pile was driven 2 inches (Figure 5). This process was repeated until the pile had been driven 14 inches. In a few cases, the high cementation sand models were too hard for the piles to be driven 14 inches. The hardened material caused severe cracking of the soil mass and container failure from the driving energy. When this occurred, pile driving was stopped. After the pile reached full penetration, a pullout test was done but the load was not measured.

*The word "x-ray" implies that film, placed behind the model, was exposed to the x-ray source that recorded the position of the grid, lead shot pellets, and the pile.

Results

The objective of the NCEL laboratory program was qualitative and attempted to provide a visual representation of the soil-pile interaction behavior. To that extent, the objective was met. The only parameter measured was the driving energy, which was correlated with the parameters of void ratio, cementation, and carbonate content.

Regarding the selected range of cementation levels, the findings indicated that the calcareous sand practically reached an indurated state where the bulk soil sample cracked and behaved as cemented chunks. The influence of the state of induration on the material behavior was not, therefore, outside the testing boundary of slight to medium cementation. Any effects due to this parameter would be seen. For silica sand, this stage was reached at the 2% cementation level, therefore, tests on the 4% and 8% cementation for silica sand were not conducted because of this finding.

Figures 6 through 13 show complete suites of x-rays for selected soil models. All points in each x-ray were computer digitized to reduce the data for comparing the various behaviors. Table 4 summarizes the total driving energy for each soil model tested. Figures 14 through 16 show the variation between cementation, carbonation, and void ratio. Based on the model study at NCEL the following observations were made:

1. The amount of calcareous sand in a sample appeared to control the amount of resistance the soil mobilizes during pile driving for dense soil samples. Figure 14 shows that for the dense models containing 100% silica sand, the slope of the input energy curve was larger than samples containing 100% calcareous sand. The slope of input energy for similar dense models containing 50% silica sand and 50% calcareous sand resembled the 100% calcareous sand model slope. This became evident when the loose density curves were analyzed in Figure 14. All the curves show that input energy increased with cementation, however, each one shows a different slope. Closer examination shows that the total energy slopes of the loose density mixtures appear linear and as the composition of the model changed so did the rate of energy. That is, as opposed to dense soil model, the amount of calcareous sand does not control the amount of resistance the soil mobilizes during pile driving.

2. The pullout resistance parameter was not measured in this test. It was noted during the test that the calcareous sand's pullout resistance did not appear to increase with increased driving energy, nor did it increase with increased cementation. Contrarily, the pullout resistance for silica sand increased substantially with increased cementation, and at the higher cement contents the pile was very difficult to remove from the sand model. This suggests that calcareous sand's driving energy is independent of pullout tension.

3. Figures 15 and 16 show the horizontal and vertical movement of the lead shot pellets relative to their distance from the pile surface. Several general observations can be made:

a. The motion of pellets in the calcareous sand model was larger than in the silica sand models. Since the silica sand showed more driving resistance, perhaps the calcareous sand required more movement to mobilize its strength.

b. The total motion in the cemented sand models was smaller than in noncemented models. This suggests that cement bonding, although small, was tying grains together causing the mass to respond more as a unit.

c. The motion of the lead shot was greater in the low void ratio (dense) then in the high void ratio sand models. This was expected because high void ratio sand compacts in order to mobilize its shear strength whereas low void ratio sand transfers the shear load further into the sand medium enabling the medium to mobilize more strength.

d. The downward vertical movement of the high void ratio calcareous sand exceeded that of the silica sand. In the low void ratio sand models, the majority of vertical movement was in an upward direction suggesting that more particles were involved with producing a greater mobilized strength. Furthermore, the increased upward vertical movements of the low void ratio silica sand model indicated that this sand responded more effectively and with higher strength to resist loading.

FRICTIONAL BEHAVIOR TESTS

A study (Noorany, 1982a) investigated the internal friction and soil-metal friction of two calcareous sands, and evaluated the influence of particle crushing on these properties.

The study did obtain a clearer view of the frictional behavior of calcareous sands with particular emphasis on the effect of particle crushing on the internal friction angle, ϕ, and on the soil-metal friction angle, δ.

Test Plan

First, the friction angle of each sand was measured by triaxial compression tests in loose as well as dense conditions. Next, each calcareous sand was crushed and tested again to determine the effect crushing had on the soil friction angle, ϕ, in loose and dense states.

The soil-metal friction was measured for both calcareous sands in loose as well as dense conditions. The effect of particle crushing on soil-metal friction angle was also investigated.

For comparison, a silica sand was also tested. The results of the tests were analyzed to evaluate the influence of soil crushing on shear behavior and frictional resistance of calcareous sands.

Test Materials

Two calcareous sands, from Guam and Florida, and a Ottawa sand, were tested. The calcareous sand from Guam was uniformly graded coraline material with a D_{50} = 0.45 mm and a uniformity coefficient,

$C_u = D_{60}/D_{10} = 1.8$. The natural grain size distribution curve for the sand is shown in Figure 17. The sand contained both rounded and elongated particles. Microscopic examination indicated that most of the particles were porous and had a rough texture. Even the very small particles were porous, and some were even hollow. The specific gravity of solids measured 2.80. The grain size distribution curves for the crushed sand, compared with the natural sand, are also shown in Figure 17.

The calcareous sand from Florida was the same material used in the previous laboratory study. It was a uniform calcareous sand with a $D_{50} = 0.4$ mm and a uniformity coefficient, $C_u = D_{60}/D_{10} = 2.8$. This sand was finer than the Guam sand and contained about 2% by weight silt-sized particles (finer than sieve No. 200). The grain size distribution curve for the sand is shown in Figure 18. The specific gravity of solids measured 2.72. The sand contained flat pieces of broken shells as well as bulky particles. Under the microscope, the texture of the particles appeared rough. The sand was also tested after crushing. The grain size distribution of the crushed sands, compared with that of the natural sand, are also shown on Figure 18.

The Ottawa sand is a uniformly graded silica sand with a $D_{50} = 0.45$ mm and a uniformity coefficient, $C_u = D_{60}/D_{10} = 2.0$. The grain size distribution curve for this sand is shown in Figure 19. The sand consisted of bulky particles that under the microscope appeared to have smooth, polished surfaces. The specific gravity of solids for this soil measured 2.61. The purpose for using the Ottawa sand was to compare the results of the calcareous sand to a typical non-calcareous sand.

Test Procedures

The internal friction angle of each sand was measured by means of triaxial compression tests. Test specimens had a cross-sectional area, A_o, of 10 cm^2 and were prepared in both loose and dense conditions. The loose samples were made by a raining technique where the dry sand is poured through a funnel into a cylindrical sample mold. The distance between the funnel and the accumulating sand was kept constant. Dense samples were prepared by placing the sand in five layers and compacting each layer with a small, hand-held tamper. The isotropic compression behavior was studied when confining stresses were applied to the samples. This was done by observing volume changes under the compression loads.

All triaxial compression tests were constant-rate-of-strain tests at constant lateral pressure until stress-strain curve passed a peak value. The rate of deformation for all tests was 0.03 inch per minute. The axial load was measured by means of an electronic load cell, and the axial deflection was measured by a strain indicator.

The majority of triaxial compression tests were performed on dry sand without volume change measurements. However, for each of the calcareous sands used in this study, a test was made under saturated conditions and the sample volume change was measured during drained shear (i.e., no excess pore pressure build-up). This was done to observe the dilatancy characteristics of the sands.

The friction coefficient of each sand against metal was measured directly by sliding a rigid flat steel plate on the surface of sand. The plate was 3-1/2 inches by 3-1/2 inches, and was placed on a bed of sand 1 inch deep. After the desired normal stress was applied, the plate was subjected to shear loads in small increments until sliding occurred.

Results

The results of the triaxial shear tests are presented in Table 5. The results show that the friction angle of the calcareous sands was about 10 degrees higher than that of the Ottawa sand. This was true for both sands in loose and dense conditions. The range of variation of friction angle of these calcareous sands from loose-to-dense condition was narrow. The increase in friction angle from loose-to-dense condition was only 3 to 4 degrees for both sands. The high friction angles of the calcareous sands tested were not due to dilational behavior during shearing. In the loose condition, both sands showed volume decrease, yet had friction angles in the range of 44 to 46 degrees.

Some particle crushing occurred during shearing of these calcareous sands in the triaxial compression tests (Figure 20). When additional tests were made on the crushed sand, the amount of further crushing during shear was substantially reduced.

The friction angle of these calcareous sands decreased as the confining pressure was increased. At a confining pressure of 8,000 lb/ft^2 the amount of reduction in ϕ value for calcareous sands was 2 to 4 degrees. Even partially crushed sand showed some reduction in the ϕ value when tested under high confining pressures.

Tests on the crushed and recompacted calcareous sands indicated that the crushed sand was not weaker than the natural sand. When the friction angle of natural sand at a given void ratio was compared with the friction angle of partially crushed or ground calcareous sand at the same void ratio, it was found that they were about the same value (Table 5).

The results of isotropic compression tests indicate that the calcareous sands tested are more compressible than the Ottawa sand (Figure 21). This could be due to the presence of intraparticle voids, and crushing of the sharp edges and corners of highly irregular particles.

The results of the soil-steel friction tests are presented in Table 6. These data indicate that for the two calcareous sands, the soil-steel friction angle appears independent of soil density, and increased slightly after crushing. For the Ottawa sand the friction angle is independent of soil density. No tests were done with crushed Ottawa sand.

CRUSHABILITY

The behavior work performed by NCEL provided a clearer understanding of the soil-pile interaction, and a better understanding of the crushing phenomena. However, answers to questions such as the location and extent of crushing relating to various cementation levels and void ratios, and what effect crushing has on driving energy and pullout tension would provide a better understanding of the overall soil-pile interaction and the shear transfer mechanism operating in calcareous sands. To study these questions, NCEL awarded a contract to ERTEC Western, Inc., Long Beach, Calif.

ERTEC Program

In an effort to correlate the findings from this study with those from previous studies, ERTEC's work was developed around varying void ratios and cementations, and driving a model pile into two different sands. The objective of this investigation was to qualitatively examine the following items related to piles in calcareous sands:

- The effect of pile driving on the degree of grain crushing.

- The effect of grain crushing on the frictional resistance characteristic of piles in calcareous sands.

- The effect of cementation on the crushability and frictional resistance characteristics.

The test setup and equipment used for this investigation were specifically designed and built to meet the project goals. Figure 22 is an illustration of the complete test setup. The test setup consisted of the following systems:

- Model pile and pile driving stem

- Sample retention system (red barrel)

- Sample preparation system

- Pullout load test system

- Instrumentation and data acquisition system

The model pile and the driving assembly was the same as that used in the NCEL program, i.e., a 1-1/2-inch diam pile driven into the sand by free weights falling through a guide rod and impacting on top of the pile. The sample was placed in a 30-inch diam by 30-inch high, hollow steel cylinder test drum with end plates bolted to both ends of the cylinder. The test drum was equipped with a pressurization system that could apply up to 100 psi pressure to simulate overburden and lateral stress conditions. For this study, an overall confining pressure of 20 psi was applied in every test, simulating a pile driven about 50 to 60 feet below the seafloor. The pullout load test included a hydraulic loading system and load frame. The hydraulic system consisted of a hydraulic actuator, a hydraulic pump, and a flow rate adjuster. This system was used to pull out the pile at a slow rate.

Test Procedures

A total of 20 laboratory tests were performed on the two different sands prepared at different densities, moisture contents, and cement contents. Each test was done in the following sequence:

1. Prepare and cure each sample.

2. Place the sample in the test drum and apply a confining pressure of 20 psi.

3. Drive the model pile into the sample.

4. Perform a pullout test.

5. Subsample specimens at various depths and distances from the soil-pile interface.

Two methods of sample preparation, tamping, and raining were used in this investigation. Both methods were based on the procedure used in the previous behavior test to achieve an in-place low and high void ratio density.

All prepared samples were cured in a 100% humidity room for 3 days before they were placed in the test drum. This curing period was determined on the basis of unconfined compression tests that indicated that 3- and 7-day strengths of the samples were about the same. After curing, the test sample was placed on top of a prepared bed of silica sand in the test drum. The space between the test sample and the drum wall was then filled with silica sand compacted in layers with a density similar to the test sample to maintain compliance. The drum was then covered and all pressure lines connected. A hydrostatic confining pressure of 20 psi was applied to the sample.

The pile driving assembly was attached after the sample was in place and the confining pressure applied. Pile driving was done by manually dropping a 15-pound deadweight 12 inches above the pile head. Pile driving continued until the model pile penetrated 12 inches. Pullout tests were performed with the hydraulic pulling system. The pile was connected to the vertical ram of the hydraulic actuator and pulled vertically at a speed of about 0.04 inch per second. The pullout test continued until a maximum pullout force was reached. The pullout resistance force and pile displacement were continuously monitored on an X-Y-Y-recorder.

It was hypothesized that the degree of grain crushing, if any, would be more severe at or near the pile and decrease significantly as the distance away from the pile increased. It was necessary, therefore, to obtain soil specimens at or near the pile wall during this test. Nine soil specimens were taken from each test sample at three horizontal distance intervals of 0 to 1/4, 1 to 2, and 2 to 3 inches from the pile perimeter and at three depth intervals between 0 to 4, 4 to 8, and 8 to 12 inches from the soil surface in each horizontal distance interval.

Specimens from the cemented samples were analyzed for grain size. In order to do this, the cementing agent and sand grains would have to be separated to assess the changes in grain size distribution. The regular separation method of using physical or mechanical forces was not acceptable since it could break and crush the soft calcareous sand grains. Chemicals could not be used because they could dissolve the calcareous sand grains as well as the cement agent. Instead, the calcareous sand and the cement agent was separated by hand. Extreme care was taken to prevent crushing the sand grains. After the specimens were separated, they were soaked in distilled water for 24 hours. Sieve analyses were then performed on the soil specimens in accordance with ASTM D422-63 procedures (ASTM, 1981).

Results

Of the 20 tests performed, 5 were nonscheduled and 15 were scheduled. The nonscheduled tests were "shakedown tests" to calibrate and modify various test procedures. Relevant data and characteristics of all test samples, as well as certain test results, are summarized in Table 7. The shakedown test data were included in Table 7 for completeness and to qualitatively supplement the results of the scheduled tests.

An examination of the pile driving energy data indicated that:

1. As expected, higher driving energy was required to install the pile into higher density sand samples.

2. For higher density samples of calcareous and silica sand, the following observations were made:

   - The driving energy required to install the pile increased with increased cement content.

   - At each prescribed cement content, pile driving resistances were on the same order of magnitude for higher density silica and calcareous sand samples.

3. For lower density samples of either silica or calcareous sand, an increase in cement content did not increase pile driving resistance.

The results of maximum pullout resistance and corresponding pile displacement data are also summarized in Table 7. A summary plot of the maximum pullout force versus the number of blows required to advance the pile 12 inches is provided in Figure 23. Based on these results, several observations were made:

1. Although the pile driving resistances for the higher density silica sand and calcareous sand samples are similar, the pullout resistance of the silica sand was two to five times greater than of the pullout resistance of the calcareous sand. The difference of the pullout resistance appeared to increase with the increases of cement contents and pile driving resistance.

2. In lower density calcareous sand, pile pullout resistance appeared to increase with the increase of pile driving resistance.

3. In silica sand, pile pullout resistance increased with the increase of pile driving resistance.

Based on these observations, it appeared that pile driving resistance may not be a rational parameter for use in pullout capacity (or frictional pile capacity) predictions for piles in calcareous sands.

As described earlier, subsamples of the soil samples were taken at various depths and distances from the pile wall after each pullout test. The fines content versus distance from the pile wall (soil-pile interface) are presented in Table 8. These results show that most of the grain crushing took place at or near the pile wall. The degree of grain crushing decreased away from the pile wall. The grain size analyses for the specimens show that:

1. At or near the pile wall, more grain crushing occurred along the bottom 8 inches of the pile than the top 4 inches.

2. Pile driving crushes more grains of the higher density calcareous sand samples while its effect on silica sand and lower density calcareous sand samples was less pronounced.

The observation in item 1 appears questionable since one would think that grain crushing would be more pronounced near the soil surface where more shear stress cycles are imposed by the pile driving. This could be caused by the physical limitation of the red barrel and pressurization system. The top 4 inches of the pile were subjected to less complete confinement than the bottom 8 inches of the pile, thus, it can be hypothesized that grain crushing also depends on the applied confining stress.

ALTERNATIVE PILE CONCEPTS

In the past, the most popular piles used in calcareous sand have been the open-ended pipe piles. The reasons for this were discussed in the BACKGROUND section. However, it has been hypothesized that driven pipe-piles create problems in calcareous sand, particularly in regard to developing skin resistance. These hypotheses have been verified by ERTEC, 1983a. The recognized problems do not rule out the use of pipe piles, but other piling concepts might be more appropriate for some cases.

ERTEC was funded to investigate the possibility of using alternative piling systems that could be used in calcareous soils. Conceptual systems were developed on the basis of the current understanding of the pile behavior in calcareous sands. Because the behavior of piles in calcareous sands is complex and the understanding meager, it was necessary to postulate various behavioral aspects on the basis of engineering understanding and judgment. Attempts were made to extend beyond the state-of-the-art, thus, some of the developed systems involve substantial risks and require development work in order to prove their viability.

In developing the improved piling systems for calcareous sediment applications, the following principles were considered:

1. Recognized the various special behavioral aspects of piles in calcareous soils based on our knowledge.

2. Postulated the mechanisms behind these aspects in accordance with good engineering judgment and assumptions if the knowledge was nonexistent.

3. Improved various behavioral aspects that yield low load carrying capacity, and maintained or enhanced various behavioral aspects that yield relatively high load carrying capacity similar to those in comparable terrigenous soils.

4. Developed practical systems that were practical, achievable and involved minimal risk and development.

5. Applied feasible, proven techniques, that were successful elsewhere.

Installing and loading piles in calcareous sediments will introduce varying degrees of grain crushing and volumetric contractions near the pile. The extent of grain crushing and volumetric contraction depends on the piling system, installation method, characteristics of calcareous sediments, state of stress, loading characteristics, and other factors. After examining the key behavioral aspects, it was clear that any improved piling system must incorporate one or a combination of the following features:

1. Must increase the effective lateral stress on pile shaft.

2. Must force the pile to transfer load to the zone of soils where degradation due to grain crushing is minimal.

3. Should have an enlarged base area.

4. Should eliminate or reduce the effect of grain crushing and associated volumetric contraction as well as soil arching.

5. Should increase contact area between pile and sediments.

Numerous variations of piling systems exist that can incorporate one or a combination of the above features. Most of the conventional piling systems (except driven piles) do incorporate some of these features. These features are costly and are out of the scope of this conceptual development work.

The following improved piling systems were conceptually developed following the approaches and principles described in previous sections:

- Backfilled piles (BP)
- Vibratory installed backfilled piles (VBP)
- Pressurized piles (PP)
- Backfilled and pressurized piles (BPP)
- Piles with enlarged tips (PET)
- Modified drilled and grouted piles (MDGP)
- Keyed-in piles (KIP)
- Drilled and screwed piles (DSP)
- Other potential piling systems

These concepts are different from conventional piling systems to a varying extent and were evaluated on the basis of the limited understanding presently possessed, various postulations regarding calcareous soil-pile interaction, and engineering judgment. These concepts and their effectiveness of improving load carrying capacity in calcareous sediments still need to be proven. In addition, most of the procedures and equipment required for deploying these "improved" piling systems need further development. Some of the piling systems are presented with various alternates depending on site conditions and applications.

Backfilled Piles (BP)

As shown in Figures 24 through 26, this system installs the pile in an oversized hole and then backfills the annulus between the pile and the hole with granular material. The granular material can be densified by internal underwater vibrators or by vibration force provided from the pile top or by other means. This procedure is very similar to the drilled and grouted piles except granular material is used instead of grout.

Using granular material as backfill offers the following advantages:

1. Increases the effective lateral stress on the pile shaft, thus, increasing the skin friction resistance.

2. Eliminates grain crushing and soil arching effect in the calcareous sands.

3. Fills the cavities with granular material.

4. Eliminates the need for grout.

The major disadvantages are: (1) Granular material must be quarried and placed underwater, and (2) drilling equipment must be used to drill the oversized hole. There are several alternates that are capable of drilling an oversized hole. They are described as follows:

Alternate A - Conventional Drilling. This alternate is shown in Figure 24. The oversized hole can be drilled by conventional drilling techniques. For noncemented or lightly cemented calcareous soils, drilling mud may be required to stabilize the drill hole. In this case, a granular material, such as sand slurry, can be pumped in under pressure to force out the drilling mud.

Further research and development work is necessary for this alternate. This includes determining:

- The types of granular materials suitable for use as backfill.
- The extent of mud contamination and its effect on the granular backfill and pile capacity.
- Procedures and equipment needed to place and compact the granular material.

- The best size of the backfilled annulus to minimize the effects of soil arching on the lateral stress.

- The pile behavior under static and cyclic loadings.

From a construction viewpoint this system is applicable with cemented or solidified calcareous sediments, where the drilled hole will stay open without using drilling mud. Installing piles under these conditions would be relatively simple and straightforward.

Alternate B - Cased Drill Hole Using a Withdrawal Tube. Contamination by drilling mud can reduce the load carrying capacity significantly. To avoid this complication, an alternative, which eliminates the need for drilling mud, is needed. This alternate (Figure 25) involves attaching a drill bit to the tip of a withdrawable tube slightly smaller than the oversized drilled hole. The drill bit and tube are advanced until the correct depth is reached. The drill bit is then withdrawn while holding the tube stationary followed by inserting the pile using a centralizer. Granular material is placed (or pumped in) into the annulus between the pile and the wall of the oversized hole. The tube is slowly withdrawn during backfilling and used to compact the granular material at regular intervals. Installation is completed when the tube is fully withdrawn.

Alternate C - Driving a Withdrawal Tube With an Expendable End Plate. This alternate is similar to Alternate B except the oversized hole is created by driving a withdrawal tube with an expendable end plate attachment (Figure 26). After the tube is driven to the correct depth the pile is inserted. Backfilling and compacting are done by following the same procedures as those described for Alternate B.

The development work required for Alternates B and C are similar to those described for Alternate A except for the problem of contamination.

Vibratory-Installed Backfilled Piles (VBP)

This system can also be used to avoid using drilling mud in noncemented and lightly cemented calcareous sediments. As shown in Figure 27, this system installs of an inverted and slightly tapered pile using vibratory hammers. The gap between the tapered piles and hole created by the pile tip can be filled with granular material from a supply reservoir or pumping in granular (sand) slurry. Vibratory hammers are used to drive the piles and compact the backfill materials at the same time.

Again, the procedures and equipment to install this system require further development. However, the effort is expected to be minimal. Further development work as described for the BP system (except for the drilling mud problem) is also needed.

Pressurized Piles (PP)

This system uses artificial imposition of a high lateral stresses on the pile shaft. This can be done in several ways. One method would be to use split-designed piles that can be expanded through hydraulic or mechanical systems. Explosives can also be used to force the pile shaft to expand outward and key into the sediments and cavities. Another method would be to design the pile with weak segments that are expanded by excessive drilling forces.

Alternate A. For this system, the structural integrity of piles during installation, pressurization, and loading has to be carefully considered in the design.

The above PP alternates are unconventional. Development work is needed in pile design, installation procedures, and confirmation of their load carrying capacity. The first alternate involves installing an expandable pile consisting of two overlapping half sections as shown in Figure 28. The pile could be installed by using either impact hammers or drilling and inserting. Hydraulic pressure or mechanical systems can be used to force the pile to expand outward. There are a number of systems that can be designed for this purpose. The criteria for selecting the correct system should be based on cost and complexity of the applications. Two sample mechanical systems are shown in Figure 28; one system expands the pile by applying a vertical downward force and the second system by applying a torsional force.

Alternate B. The second alternate (Figure 29) involves using explosive charges at regular intervals and locations where cavities are adjacent to the pile shaft. A regular, tubular steel pile (either open-ended or close-ended) can be used. After the pile is installed, the explosive is detonated. (A special pile cap may be required to maximize the blasting effect.) Exploding the pile will expand the wall and cause it to fill the voids or cavities. This system will increase the lateral pressure on the pile shaft as well as create spurring contacts with the cavity walls when the pile is subjected to axial compression or uplift (i.e., increase the skin friction resistance).

Alternate C. This alternate involves installing specially designed and fabricated, open-ended piles that have segments of weak sections (by slotting the piles or using thinner wall thickness) as shown in Figure 30. The piles can be installed by impact hammers or drilling and inserting procedures. If impact hammers are used, it is important to ensure that the impact energy is strong to install the pile to the proper depth, but will not damage the weaker segments (section) of the pile. After the pile has been installed, a higher impact energy is used to buckle the weaker segments and force them to fill any voids or cavities.

Backfilled and Pressurized Piles (BPP)

This system combines the backfilling procedures discussed above with the pressurized pile technique. The oversize hole would be drilled and backfilled. The expandable pile would then expand against the newly placed backfill.

Piles With Enlarged Tips (PET)

This system is similar to the belled piles used in the industry to achieve an increased end-bearing resistance or uplift capacity. However, belled piles for nearshore and offshore applications are very costly and time consuming. The PET developed for this study basically follows the proven techniques used on land. Two alternates were developed and are described.

Alternate A - Piles Installed With a Withdrawable Tube. Figure 31 is a conceptual drawing of this system. This system is very similar to the Franki piles, the Alpha piles or the pedestal piles (Tomlinson, 1977). To use this system:

- Lower the tube to the sediment surface.
- Drive the tube down to the correct depth by using an internal drop hammer.
- Hold the tube stationary and pour in gravel or concrete cement to form a plug at the bottom of the tube.
- Use the hammer on the plug to form a bulb (expanded) end.
- Use the hammer to drive the pile into the bulb end.
- Use the hammer to compress the concrete, grout, or granular backfill while removing the tube.

The pile is installed when the tube and hammer are removed. This system can be easily installed in most calcareous sands except in highly solidified calcareous rock (such as limestone). Since similar systems have been successfully used elsewhere, it is anticipated that this system can be readily used for calcareous application with a minimal amount of development work. The development work should concentrate on understanding the pile-sediment mechanisms and the verification of load carrying capacity increase.

Alternate B - Modified Belled-Piles (MBP). This system differs from the conventional belled piles (Figure 32). A regular open-ended steel pile can be used with this system. To install a pile:

- Use an impact or vibratory hammer to drive the pile to the correct depth.
- Insert an internal expandable drill bit and underream a belled end.
- Remove the drill bit.
- Pour concrete or grout, through the pile, into the belled end.

- Use a hammer (or another means) to force the end of pile into the concrete or grout.

- Pour in more concrete or grout to complete the pile installation.

The equipment used to remove the internal plug and to form a belled end is available and can be easily adapted for this system.

Modified Drilled and Grouted Piles (MDGP)

As shown in Figure 33, this system is a slightly modified version of the conventional drilled and grouted piles. The pile wall in this system is perforated and an oversized drill bit is attached at the tip of the pile. The pile and drill bit are moved forward at the same time to the correct depth. (No drilling mud is used, thus avoiding mud contamination.) The drill bit is removed and grout from inside the pile is forced out through the holes to fill the annulus. On some occasions the hole could collapse during drilling and prevent the pile being inserted. If this happens either a vibratory or impact force may have to be used to insert the piles.

Keyed-in Piles (KIP)

As shown in Figure 34, this system uses a specially designed pile with a mechanical keying-in system equipped with lateral extension branches that are pushed out through the piles and into the surrounding sediment after the pile has been driven or installed by vibratory hammers. Either hydraulic or mechanical systems can be used to push out the extension branches. Axial loading will force the extended branches to form a different and more complicated failure pattern in the soil than the conventional piles. Additional load carrying capacity in either axial compression or pullout is expected for this system.

More development work is needed before this system can be used. Several aspects require further evaluation; they include, but are not limited to, the following items:

- Understand the failure mechanism governing the load carrying capacity of this piling system in a variety of calcareous sediments.

- Determine what effects the keying-in system will have on the axial behavior of this system.

- Design and implementation of keying-in and pushing-in systems.

Drilled and Screwed Piles (DSP)

This system is very similar to the Fundex piles used in Europe (Tomlinson, 1977). This system (Figure 35) uses an expandable helically-screwed drill point held by a bayonet joint at the lower end of a tube. The tube is rotated by a hydraulic motor or rotary table at the same time it is forced down by hydraulic rams. After reaching the correct depth, the pile is inserted and either grout, granular material, or concrete is poured into the annulus. The tube is then withdrawn. While withdrawing, the tube can also be used to compact the backfill material.

The techniques and equipment for this system have been deployed elsewhere. It is expected that the effort required to develop necessary installation equipment and techniques for this system will be minimal. However, the extent of improvement in its load carrying capacity require further evaluation and confirmation.

DISCUSSION

General Pile Behavior

The general behavior of piles in calcareous sands can be hypothesized based on observations and engineering judgment. The driving energy increased with increased cementation (Figure 14). Figure 36 is a plot of the slopes of the curves from Figure 14. These six curves were combined into two groups - a high and low void ratio. From Figure 36, the high void ratio curve shows that as the silica content for a composition increases, so did the energy required to drive the pile, indicating linearity. The low void ratio curve did not indicate this with the amount of data available. For the low void ratio, the driving energy did not increase until the material composition was close to 100% silica sand. This implies that the behavior of calcareous and silica sand compositions is influenced by the presence of calcareous sand.

More data will be needed to verify the behavior influence that calcareous sand plays in calcareous and silica sand compositions. Applying some fundamental characteristics of shear strength in sand soils may provide a preliminary explanation as to why calcareous sand can control the behavior. From soil mechanics, we know that for strong particles in an initially loose array that the array becomes denser during shear (Peck et al., 1974). This occurs because during the application of shear the particles move and assume a tighter array. If the array is tight, the particles' grain-to-grain contact force will increase to a point where grain overriding begins. At this point the array will experience volume expansion provided the grains are strong and do not crush. The soil mobilizes greater strength through grain overriding than by densification.

In compositions of silica and calcareous sand, strong particles (silica sand) and weak particles (calcareous sand) are blended together. During pile driving, the shearing strength of the composition is mobilized to resist the pile. For the dense compositions, the grain overriding, which normally occurs for mobilizing the strength, may not occur due to the presence of the weaker grains. Where the strong silica sand grains can support high contact pressures and transfer excessive pressures to neighboring grains, this redundant loading array system breaks down when the weak calcareous grains crush and leave a void and no transfer of shear strength. With insufficient silica grain available to provide an adequate transfer system, the composition behaves like a calcareous sand.

Frictional Behavior

Despite the fact that calcareous sands are composed of crushable grains and have higher overall void ratios than noncalcareous sands, at comparable state of compaction they exhibit considerably higher friction angles (about 10 degrees higher) both in loose and dense conditions. This result can be generalized to the fact that the friction angles of calcareous sands are significantly higher than the values of most noncalcareous terrestrial sands. The high values for loose sand are particularly surprising. Other information (Datta et al., 1979 and 1980; Beringen et al., 1982) confirms this result.

High friction angles are often associated with dilational behavior (volume increase during shear), but the high friction angles of the calcareous soils tested were not due to dilational behavior during shearing. In loose state, both soils (Guam and Florida calcareous sands) exhibited volume decrease, yet they had friction angles in the range of 44 to 46 degrees. Surface roughness at grain contact points, and the reinforcing effects of elongated and/or flat particles in the soil matrix might be factors contributing to high frictional resistance of calcareous sands.

The variety of friction angles of calcareous soils from loose to dense condition is surprisingly narrow. For both soils tested, the increase in friction angle form loose to dense condition was only 3 to 4 degrees.

Some particle crushing occurred during shearing of these calcareous sands in the triaxial compression tests. When tests were made on crushed soil, further crushing during shear was reduced.

Grain crushing also seemed to play a role in decreasing the friction angle of calcareous sands observed with increases in confining pressure. The reduction in friction angle (2 to 4 degrees) appeared to be the result of this phenomenon, and is more intense at higher confining pressures (Datta et al., 1979 and 1980). Even partially crushed soil showed some reduction in the angle of internal friction when tested under high confining pressures.

Although particle crushing that occurs during shearing caused some reduction in frictional resistance, it cannot be concluded that individual crushed particles have lower frictional resistance. Tests on deliberately crushed and recompacted calcareous sands indicated that the crushed soil was not weaker than the natural soil. This is probably because of two compensating factors: (1) a tendency for soil strength to decrease because at a given void ratio the net interparticle void space in the crushed sand is higher than in natural soil; and (2) a tendency for soil strength to increase because partially crushed soil grains will break up less during shearing.

The results of the isotropic compression tests indicated that the calcareous sands tested were more compressible than the noncalcareous sand (Figure 21). This could be due to the presence of intraparticle voids, and crushing of highly irregular particles.

The results of the soil-steel friction tests are presented in Table 6. These data indicate that for the calcareous sands, the soil-steel friction angle appears to be independent of soil density, and seemed to increase slightly after being crushed. For the Ottawa sand, the friction angle also seemed independent of soil density. No tests were done on the crushed silica sand. This result is in agreement with the work done by Beringen, et al. (1982).

The contribution of skin resistance developed on the surface of a pile to total pile capacity was given in Equation (1) as $f A_s$, where f is the unit skin friction and A_s is the side surface area of the pile in contact with the soil. As shown in Figure 37, f can be expressed as:

$$f = \sigma'_n \tan \delta - K \sigma'_v \tan \delta$$

where: σ'_n = effective lateral stress acting on the pile surface

σ'_v = effective vertical stress (overburden pressure)

K = coefficient of lateral earth pressure

δ = soil-pile friction angle

The results of these tests demonstrated that calcareous soils have very high friction angles. They also have soil-steel friction angles that are higher and are comparable to silica sands. Furthermore, grain crushing caused by pile driving does not reduce the soil-steel friction angle. It seems, therefore, that the reason for the low capacities of piles in calcareous soils might be the low effective lateral stress, σ'_n, acting perpendicular to the pile surface.

It is hypothesized that when a pile is driven into a noncalcareous sand, pile vibration helps densify the sand around the pile which increases lateral pressure on the pile. In calcareous soils, the soil matrix does not densify around the pile under vibration, and the earth pressure coefficient, K, is small. Consequently, the developed skin resistance is much lower than expected. Soil cementation may further influence the soil-pile interaction (Agarwal et al., 1977). Laboratory model pile tests and full-scale field tests can help clarify the influence of lateral pressure and soil cementation on the behavior of piles in calcareous sands.

Crushability During Pile Driving

A study was conducted to investigate the relationship of grain crushing to pile-soil driving friction as a function of cementation, density, and carbonate content of a soil. The results clearly indicate that the behavior of piles in calcareous sand is complex. The results establish certain definitive behavior patterns with respect to crushability, degree of cementation, density, driving resistance, and pullout resistance. It is clear that the frictional characteristics of a pile in calcareous sand depend on the interrelated effects of these items and none of these items alone can adequately explain the behavior.

The frictional characteristics of a pile in calcareous sands are functions of the friction parameter between the pile and soil, and the lateral soil stress on the pile. As discussed earlier, grain crushing might reduce the coefficient of friction between the soil and pile

(Noorany, 1982a). But this reduction alone cannot fully explain the extremely low pullout resistance values of piles in calcareous sand. It is reasonable to postulate that grain crushing also reduces the lateral soil stress on the piles. However, no reasonable design methodology can be established without well-documented, realistic pile-load test data (laboratory and field data). These data should include measurements of axial shear transfer and lateral stress at or near the pile wall. The results of the studies thus far are not sufficient to develop a pile design method. The results do indicate that crushing takes place near the pile wall and has an adverse affect on frictional resistance. The results also clearly indicate that driving resistance is a poor indicator of load capacity. It should be recalled that this (blow count criteria) had been the state-of-the-art design procedure used for the Diego Garcia pier.

It is apparent that friction capacity of hammer-driven piles in cemented calcareous sands is poor. In order to achieve adequate capacity pile loads will have to be either transferred to the pile tip or transferred by friction, probably by using a pile installation method that can develop friction capacity.

Alternate Pile Concepts

The conceptual piling systems are different from conventional piling systems to varying extents. These systems and their effectiveness in improving load carrying capacity in calcareous sediments still need to be verified. In addition, most of the procedures and equipment required for deploying these "improved" piling systems need further development.

After further development and confirmation tests (either in the laboratory by centrifuge tests or in the field), some of these developed systems may be more applicable to certain types of calcareous sediments than others. It is anticipated that various systems might work well in certain calcareous sediments but not well in others. The ultimate goal is to achieve the ideal scenario that the pile makeup will consist of various standardized segments and features arranged to maximize its load carrying capacity in any specific type of calcareous sediments.

These systems have not been evaluated for the Navy's missions and capabilities. Some of the systems might be discarded for operational reasons. Certainly the complexities of installing these piling systems make the simplicity of hammer-driven pipe piles attractive.

CONCLUSIONS

In general the state of knowledge of the engineering behavior of calcareous sediments is meager and the understanding of pile behavior in calcareous sediments is even less. However, improvements are being made. The work reported herein adds to the knowledge on frictional behavior, the processes occurring near driven piles, and the frictional resistance of driven piles in calcareous sands.

The complexity of the depositional process, grain-structure arrangements, discontinuities, and post-depositional alterations dictate that a significant spatial variation in composition and behavior of calcareous sediments can exist within a very small distance. The most significant parameters affecting the behavior of calcareous soils seem to include (1) carbonate content, (2) grain crushability and associated volumetric changes, (3) degree of cementation, (4) index properties, and (5) geologic process. Several relevant behavior aspects of calcareous soils are summarized as follows:

- Soil is softer and more compressible after grain crushing and volumetric change induced by confining or shearing stresses.
- Friction angles decrease with increasing confining pressure.
- Friction angles are higher than or similar to the values for terrigenous soils, before being crushed.

Long, open-ended, steel-pipe piles are used for nearshore and offshore applications. Installation techniques include: (1) driving by impact hammer, (2) drilling and grouting, and (3) driving by vibratory hammers. A review of the state-of-the-art and the alternate pile systems suggests that the open-ended, pipe piles is still a popular solution to foundation problems in calcareous soils.

Present design methods rely heavily on either empirical approaches or expensive field pile load tests. Further work and a better understanding of pile-sediment interaction mechanism of piles in a calcareous sediment is necessary to develop improved design methods.

FINDINGS

1. Calcareous sands exhibit higher friction angles than silica sands.

2. The high friction angles of calcareous sands are not the result of a dilatent behavior.

3. The friction angles of calcareous sands decrease with increasing confining stress.

4. The friction angles of calcareous sands and the angles between calcareous sand and steel are not changed by grain crushing.

5. The friction angles between calcareous sands and steel are comparable to silica sand and steel.

6. Calcareous sands are more compressible than silica sands.

RECOMMENDATIONS

1. Continued laboratory work is needed for developing a rational design methodology for installing piles in calcareous sands using the large drum described above. The work needs to determine contributions to total pile resistance from end bearing and side friction and the shear transfer

mechanism for input to the new design methodology. An additional portion of the work should include obtaining cone penetrometer data for correlation with static pile load capacity. Analysis of the cone data may lead to improved in-situ soil survey methods for calcareous sand deposits. The end product of this work should be a predictive methodology for driven piles in calcareous sand in the form of guidelines and formulas based on in-situ soil survey data of specific sites.

2. Scaled up tests using centrifuge testing is recommended to initiate validation of the new design methodology and to evaluate conceptual alternative pile systems. This method of testing avoids similitude problems associated with testing small size models under earth's gravity, i.e., the stress and strain in the centrifuge model are identical to those in the prototype. Since soil has stress-dependent behavior and the ratio of body forces to gravity forces has a significant influence on both the mechanism and magnitude of failure stress, a centrifuge test program will be a good interim step from the laboratory to the expensive full scale testing. Results of this program will: (1) improve the confidence level of the new design methodology, (2) provide information for designing a more effective pile load test program, and (3) reduce the cost of the full scale pile load test program.

3. A full scale pile load test program is recommended to complete validation and provide the specifications and guidelines. This program may require two sites depending on the centrifuge test results to confirm valid design methodologies: a non-cemented calcareous sand deposit without voids, and a weakly cemented deposits with large voids filled with loose calcareous sediments.

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Table 1. Results of Pile Load Tests in Calcareous Sands (after Datta et al. 1980)

Site	Strata	Number of Tests	f_{av} (kg/cm²)	q (kg/cm²)	Remarks
Bass Strait Australia (Ref. Angemeer et al.; 1973)	Sand silty, showing nonuniform degree of cementation; skeletal debris, crushes easily	5	0.11 to 0.16	58.6 to 97.6	Cylindrical steel pile, driven; embedded length = 45 to 82 m below mudline
North West Shelf, Australia (Ref. Angemeer et al.; 1975)	Sandy silt to silty sand, showing varying degree of cementation, shell fragments present	1	0.38	---	Cylindrical steel pile section, driven 11 m in a 73 m deep oversized sleeved hole
		---	1.0	---	Drilled and grouted piles
Arabian Gulf Saudi Arabia (Ref. Fuller; 1979)	Sand, cemented, some shells and coral fragments	3	Greater than 0.32	*	Concrete precast cylindrical pile, placed in an oversized predrilled hole and driven 2 m below predrilled depth. Total pile length = 12.2 to 15.3 m below mudline. Pile failure did not occur.

where: f_{av} = unit friction

q = unit end bearing

* = $N_q \geq 40$ where $q = (q_o)(N_q)$; q_o = effective overburden pressure and N_q = bearing capacity factor

Table 2. Current Practice to Predict Ultimate Capacity of Single Piles in Calcareous Sediments

Methods	Features	References	Remarks
Conventional theory with modifications	$f = K\, q_o \tan \delta$ $q = q_o N_q$ with imposed limiting f and q values and/or with higher factors of safety used in design	Datta et al. (1980)	1. Limiting f and q values are empirical in nature 2. Limited experimental data base 3. Does not account for variability
Empirical correlation with penetration resistance to driving	Pile capacity is corrected to require energy to penetrate pile or pile head forcetime history measurements during driving	Lyon Associate (1976, 1978)	1. Resistance to driving is not a good parameter in predicting skin friction resistance and uplift resistance 2. Empirical in nature 3. Disregard the true nature of grain crushing and variability in calcareous sediment 4. The potential of over or under design exists
Correlation with in-situ tests	Correlation with SPT results	Datta et al. (1980)	1. Installation effects are not differentiated 2. Lack of data bases for correlation 3. Empirical in nature 4. Site specific in nature 5. The potential of over and under design exists
Correlation work on field pile load tests	Interpretation of field pile load tests and application or results to design	Angemeer et al. (1973, 1975)	1. Data are site specific 2. Higher confidence level of use in design 3. Costly - a significant number of tests is required for piles in calcareous sediments

where: f = unit skin friction resistance

K = coefficient of lateral earth pressure

δ = soil-pile friction angle

q_o = effective overburden

N_q = f (ϕ), bearing capacity factor

q = unit end bearing resistance

ϕ = internal friction angle

Table 3. Recommended Values of Design Parameters for Calcareous and Noncalcareous Sands (after Datta et al., 1980)

Recommended By	Year	Noncalcareous Sands (clean, medium dense-to-dense)				Calcareous Sands											
						Degree of Cementation (not specified)				Uncemented or Lightly Cemented				Well Cemented			
		δ	f	N_q	q	δ	f	N_q	q	δ	f	N_q	q	δ	f	N_q	q
McClelland	1974	30	1.0	40	100	30*	0.2	40	50	--	--	--	--	--	--	--	--
API	1977	30	l.c.	40	l.c.	l.c.	l.c.	l.c.	l.c.	--	--	--	--	--	--	--	--
Consultant A	1978	30	2.0	40	100	25	0.2	20	65	--	--	--	--	--	--	--	--
Consultant B	1979	30	1.0	30	100	--	--	--	--	30*	0.3	40*	30	30*	0.6	40*	60

*implied although not specifically stated

where: δ = friction angle between pile and soil, degrees

f = unit skin friction, kg/cm^2

N_q = bearing capacity factor

q = unit end bearing, kg/cm^2

l.c. = as per local conditions

Table 4. Summary of NCEL Test Program and Results

Type of Sand Model	Mode of In-Place Sand	Cementation (%)	Void Ratio[a]	Moisture Content for Sand Placement (%)	Total Energy for 12-in. Penetration (ft-lb)	Total Energy for 14-in. Penetration (ft-lb)
Calcareous	High void ratio	0	2.10	19	102	140
Calcareous	High void ratio	1/4	1.96	19	112	142
Calcareous	High void ratio	1/2	1.93	19	190	255
Calcareous	High void ratio	1	2.07	19	322	440
Calcareous	High void ratio	2	1.90	19	728	978
Calcareous	High void ratio	4	1.86	19	1,120	1,410
Calcareous	High void ratio	8	2.00	19	2,020	2,620
Calcareous	Low void ratio	0	1.56	19	1,448	1,688
Calcareous	Low void ratio	1/4	1.45	19	1,560	2,000
Calcareous	Low void ratio	1/2	1.43	19	2,140	2,890
Calcareous	Low void ratio	1	1.41	19	1,620	2,120
Calcareous	Low void ratio	2	1.38	19	2,700	3,440
Calcareous	Low void ratio	4	1.40	19	5,140	6,520
Calcareous	Low void ratio	8	1.37	19	6,760	7,920
Silica	High void ratio	0	0.70	10	178	230
Silica	High void ratio	1/4	0.80	10	578	762
Silica	High void ratio	1/2	0.72	10	435	610
Silica	High void ratio	1	0.81	10	970	1,325
Silica	High void ratio	2	0.77	10	2,480	2,990
Silica	High void ratio	4	0.77	10	3,180[b]	3,580[b]
Silica	High void ratio	8	0.74	10	3,920[b]	3,920[b]
Silica	Low void ratio	0	0.60	10	1,310	1,840
Silica	Low void ratio	1/4	0.60	10	1,130	1,550
Silica	Low void ratio	1/2	0.60	10	1,350	1,840
Silica	Low void ratio	1	0.59	10	2,840	3,500
Silica	Low void ratio	2	0.59	10	3,580	4,220
Silica	Low void ratio	4	--	--	--	--[c]
Silica	Low void ratio	8	--	--	--	--[c]
Calcareous/Silica	High void ratio	0	1.44	16.1	170	228
Calcareous/Silica	High void ratio	1	1.43	16.1	475	615
Calcareous/Silica	High void ratio	2	1.41	16.1	1,170	1,510
Calcareous/Silica	Low void ratio	0	0.96	16.1	2,080	2,780
Calcareous/Silica	Low void ratio	1	0.96	16.1	3,250	3,930
Calcareous/Silica	Low void ratio	2	0.93	16.1	3,600	4,580

[a]Volume voids:volume solids

[b]Data for 8-inch penetration only. Box began separating at this point.

[c]Sample not mixed or tested because cement level solidified composition.

Table 5. Measured Internal Friction Angles for Calcareous and Ottawa Sands

Soil Type	Dry Unit Weight (pcf)	Void Ratio	Range of Confining Stress (kg/cm^2)	Range of Friction Angle (deg)
Loose Calcareous Sand	74.0	1.36	0.5 to 4	43 to 46
Dense Calcareous Sand	80.0	1.18	1 to 4	45 to 49
Loose, Crushed Calcareous Sand	--	1.32	0.6 to 4	42 to 46
Dense, Crushed Calcareous Sand	--	1.12	1.2 to 4	45 to 48
Loose Calcareous Sand	--	1.44	0.5 to 4	43 to 44
Medium Dense Calcareous Sand	--	1.30	1 to 3	43 to 45
Dense Calcareous Sand	--	1.19	1 to 4	43 to 47
Medium Dense, Crushed Calcareous Sand	--	1.30	1 to 4	44.5
Dense Crushed Calcareous Sand	--	1.06	1 to 4	46 to 49
Loose Ottawa Sand	85.5	0.905	0.5 to 4	35
Dense Ottawa Sand	94.1	0.73	1 to 4	40

Table 6. Soil-Steel Friction Angles for Ottawa, and Calcareous Sands

Sand	Soil Condition	Void Ratio	Soil-Steel Friction Angle (deg)
Ottawa	Loose	0.905	21
	Dense	0.73	20
Guam	Loose	1.36	18
	Dense	1.18	18
	Loose crushed	1.32	21
	Dense crushed	1.12	22
Florida	Loose	1.44	20
	Dense	1.19	20
	Loose crushed	1.30	23
	Dense crushed	1.06	23

Table 7. Summary of the Test Program and Results

Test Sample No.	Soil Type	Dry Density, gm/cc (%)	Moisture Content gm/cc (%)	Sample Preparation Method	Cement Content (%)	Curing Period (days)	Blow Count	Penetration (in.)	Maximum Pullout Force (lb)	Displacement (in.)
C-1	Calcareous	1.17	19	Low void ratio	0.0	0	182	12.0	330	2.0
C-2	Calcareous	1.17	19	Low void ratio	0.5	4	323	12.0	280	2.0
C-3	Calcareous	1.17	19	Low void ratio	1.0	3	364	12.0	270	2.0
C-4	Calcareous	1.17	19	Low void ratio	2.0	6	345	12.0	312	1.0
C-5	Calcareous	0.94	19	High void ratio	0.0	0	29	12.2	105	0.45
C-6	Calcareous	0.94	19	High void ratio	0.5	3	42	12.2	170	0.70
C-7	Calcareous	0.94	19	High void ratio	1.0	3	42	12.0	116	0.80
C-8	Calcareous	0.94	19	High void ratio	2.0	3	45	12.1	176	0.80
S-1	Silica	1.61	10	Low void ratio	0.0	0	95	12.0	640	1.0
S-2	Silica	1.61	10	Low void ratio	0.5	4	242	12.0	1,200	1.9
S-3	Silica	1.61	10	Low void ratio	1.0	3	245	12.0	1,210	1.2
S-4	Silica	1.61	10	Low void ratio	2.0	3	353	12.0	1,330	1.2
S-5	Silica	1.51	10	High void ratio	0.0	0	116	12.0	600	1.0
S-6	Silica	1.51	10	High void ratio	0.5	3	144	12.0	680	1.6
S-8	Silica	1.51	10	High void ratio	2.0	3	137	12.0	490	0.4
SC-2	Calcareous	1.07	19	Low void ratio	0.5	4	50	12.1	115	1.2
SC-3	Calcareous	1.07	19	Low void ratio	1.0	3	110	12.0	340	1.6
SS-3	Silica	1.61	10	Low void ratio	1.0	4	223	12.0	1,460	0.8
SC-8	Calcareous	0.94	10	High void ratio	2.0	3	31	12.1	60	0.40
SC-1	Calcareous	1.07	19	High void ratio	0.0	0	47	12.0	145	0.30

NOTES:
1. Test Sample Nos. C-1 through C-8 and S-1 through S-8 are scheduled tests.
2. Test Sample Nos. SC-1, SC-2, SC-3, SC-8, and SS-3 are nonscheduled tests.

Table 8. Changes in Fines Content

Test Sample No.	Type of Soil	Dry Density (g/cm^3)	Water Content (%)	Cement Content (%)	Average Fines Content[a]	Change in Fines Content, 0-4 inch[b] (%)	Change in Fines Content, 4-8 inch[b] (%)	Change in Fines Content, 8-12 inch[b] (%)	Average Change in Fines Content 0-12 inch (%)
C-1	Calcareous	1.17	19	0.0	2.3	2.6	8.2	9.4	6.7
C-2	Calcareous	1.17	19	0.5	2.8	4.2	11.6	12.0	9.3
C-3	Calcareous	1.17	19	1.0	2.6	5.7	12.2	11.1	9.7
C-4	Calcareous	1.17	19	2.0	3.7	7.9	13.2	11.3	10.8
C-5	Calcareous	0.94	19	0.0	2.6	0.5	1.6	0.9	1.0
C-6	Calcareous	0.94	19	0.5	2.5	0.6	2.2	2.0	1.6
C-7	Calcareous	0.94	19	1.0	2.6	0.8	1.1	1.5	1.1
C-8	Calcareous	0.94	19	2.0	2.9	0.3	1.6	2.5	1.5
S-1	Silica	1.61	10	0.0	0.0	0.0	0.3	0.5	0.3
S-2	Silica	1.61	10	0.5	0.5	0.1	1.4	1.5	1.0
S-3	Silica	1.61	10	1.0	0.8	0.4	1.4	1.6	1.1
S-4	Silica	1.61	10	2.0	1.5	0.4	1.8	2.4	1.5
S-5	Silica	1.51	10	0.0	0.0	0.2	0.5	0.6	0.5
S-6	Silica	1.51	10	0.5	0.4	0.2	0.8	0.8	0.6
S-8	Silica	1.51	10	2.0	1.3	0.2	0.8	0.8	0.6

[a]These specimens were taken from an area 2 to 3 inches from the pile wall.

[b]These specimens were taken at or near the pile wall.

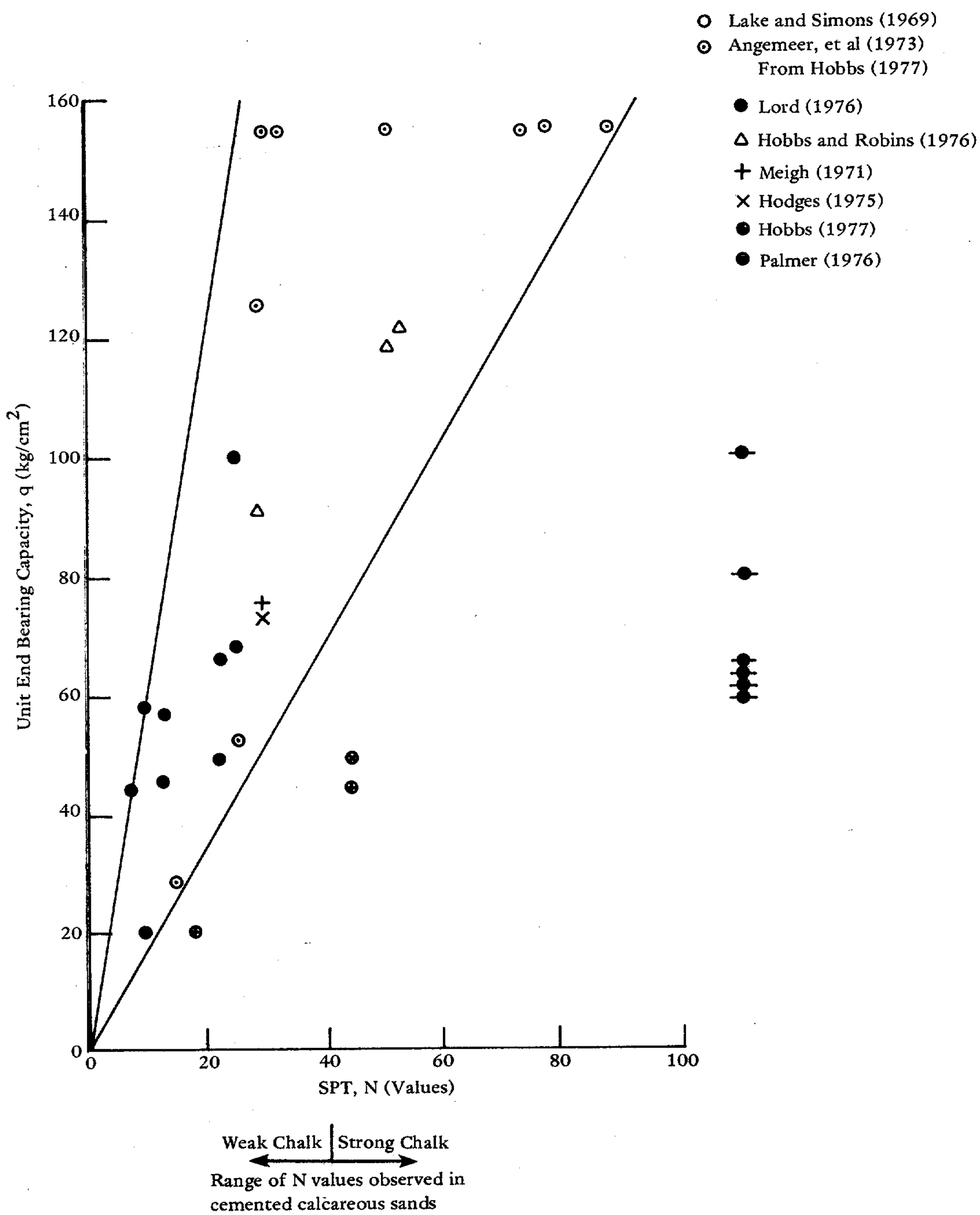


Figure 1. Relation between unit end bearing capacity and standard penetration resistance in chalk (after Datta et al., 1980).

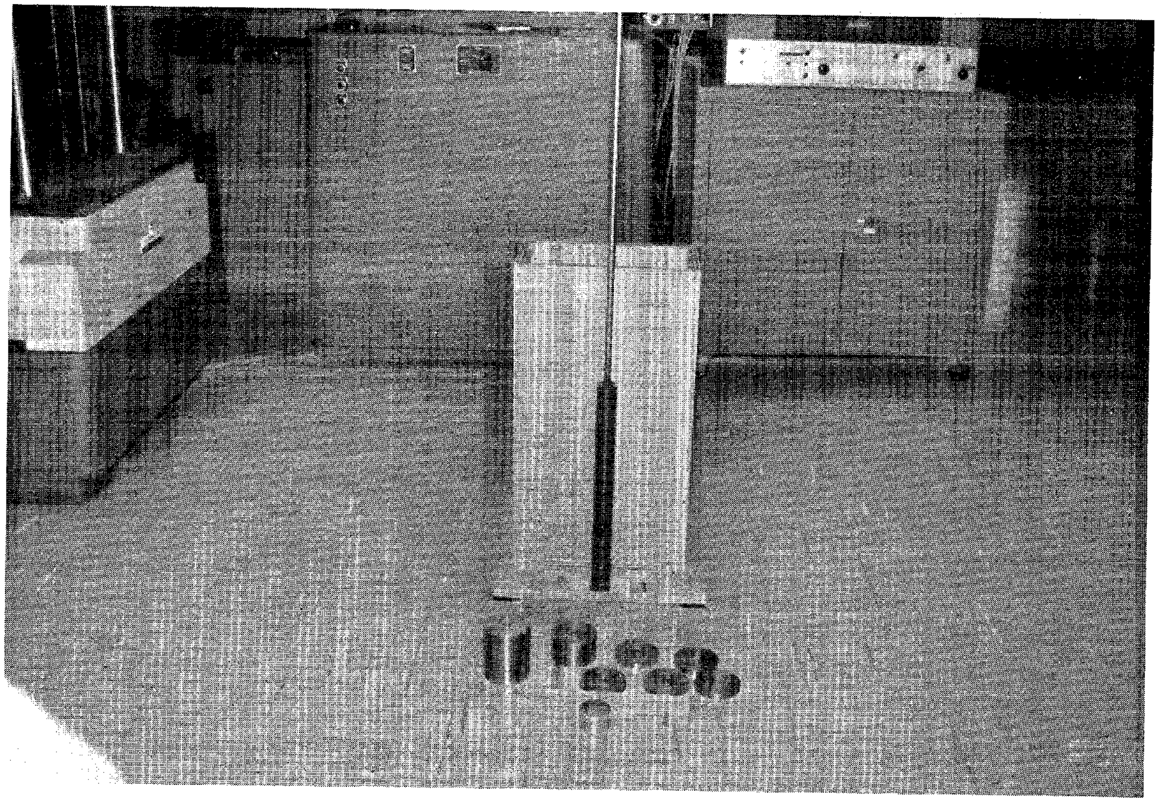


Figure 2. Equipment used in calcareous sediment behavior study.

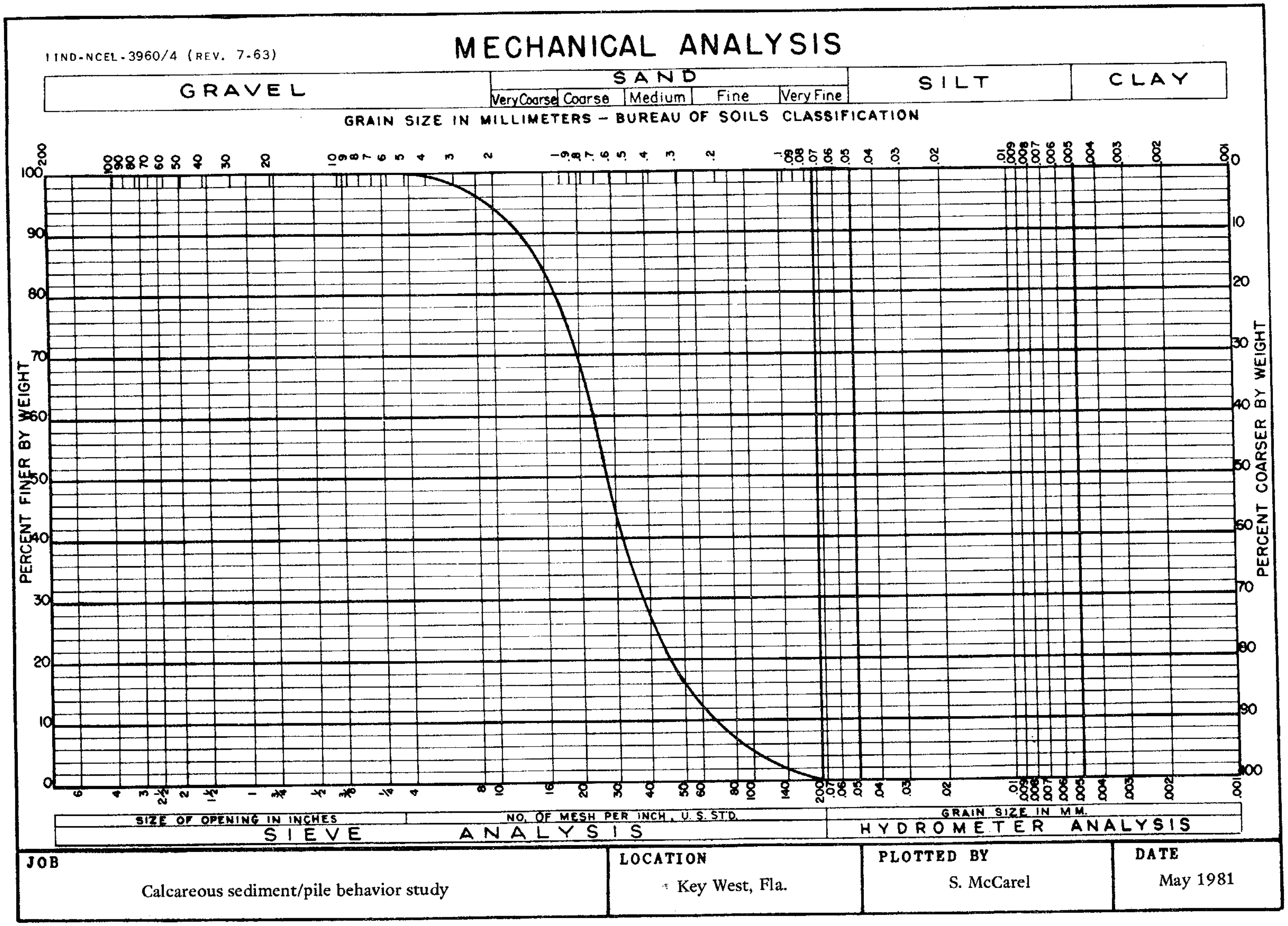


Figure 3. Grain size distribution of Florida calcareous sand.

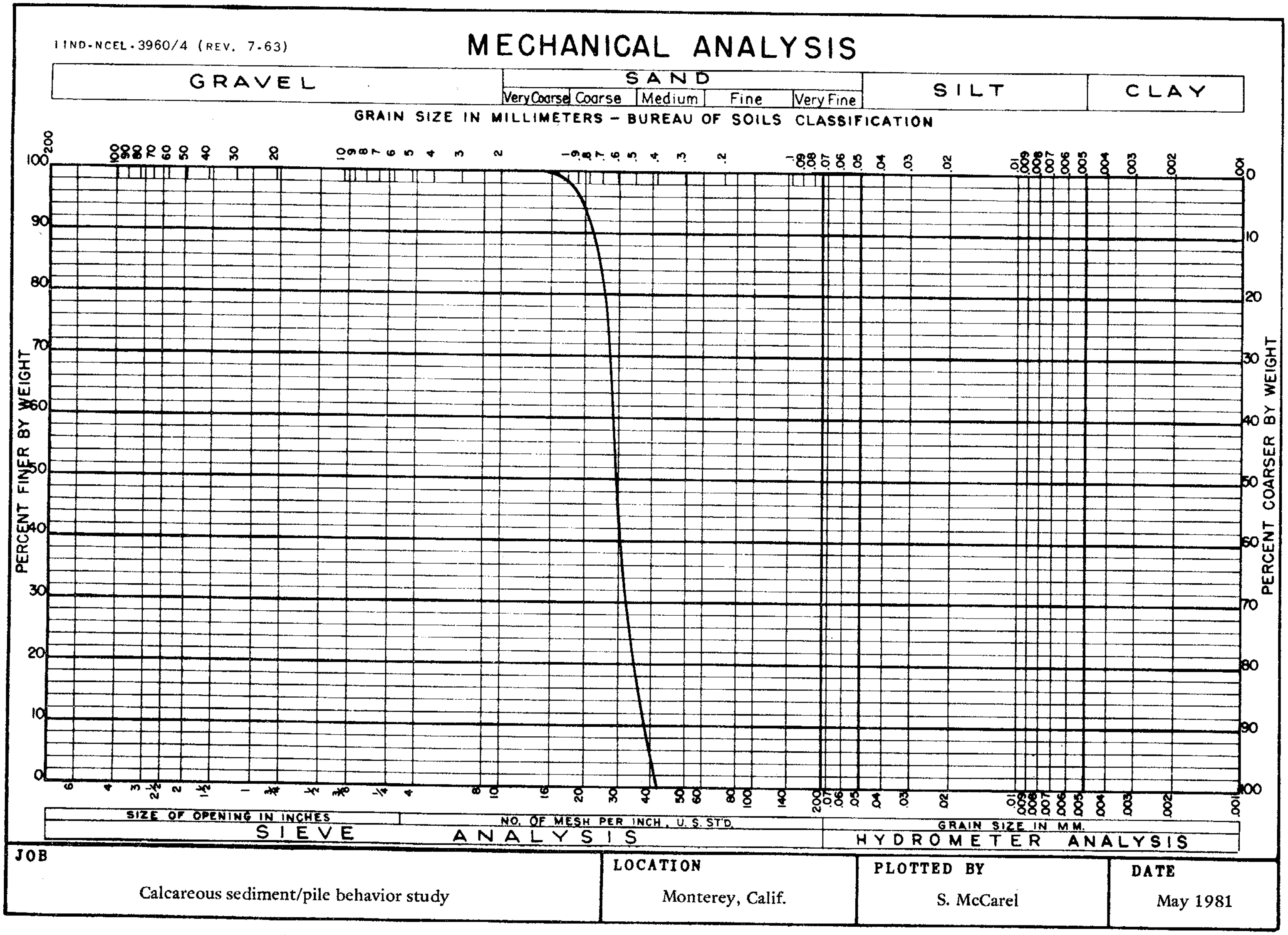


Figure 4. Grain size distribution of Monterey silica sand.

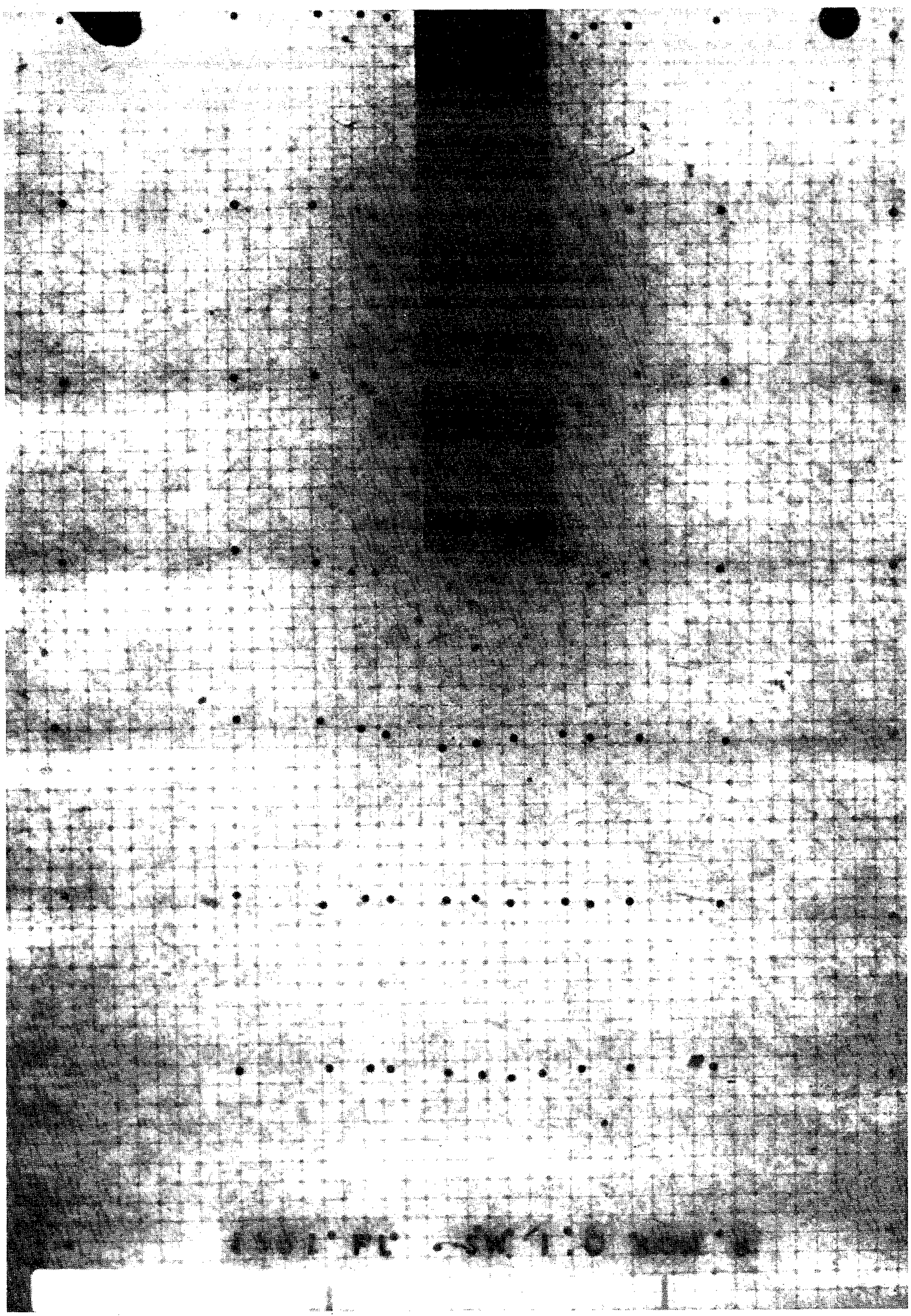


Figure 5. X-ray of the grid, lead shot pellets and model pile.

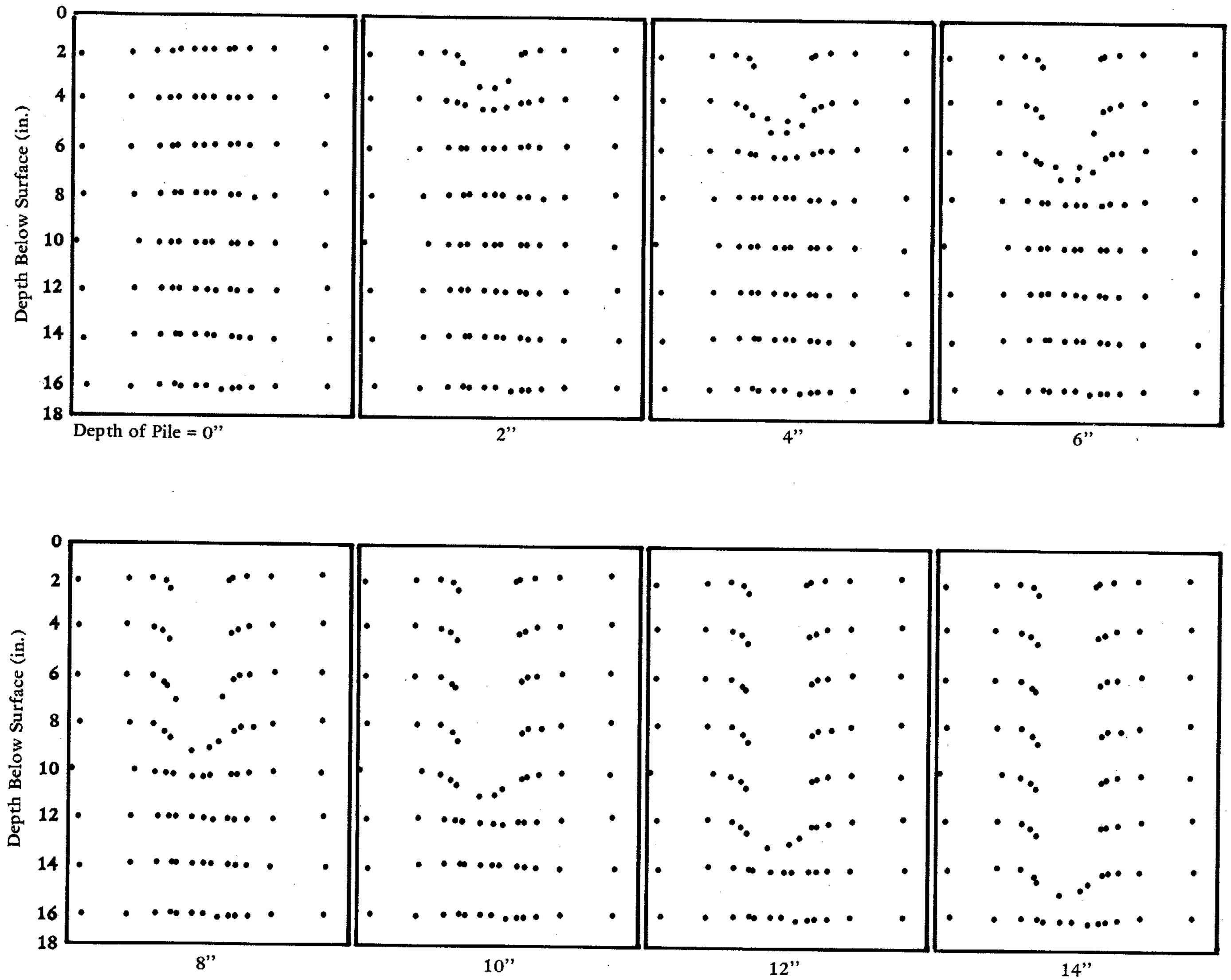


Figure 6. Complete x-ray suite for calcareous sand, high void ratio, and 0% cement.

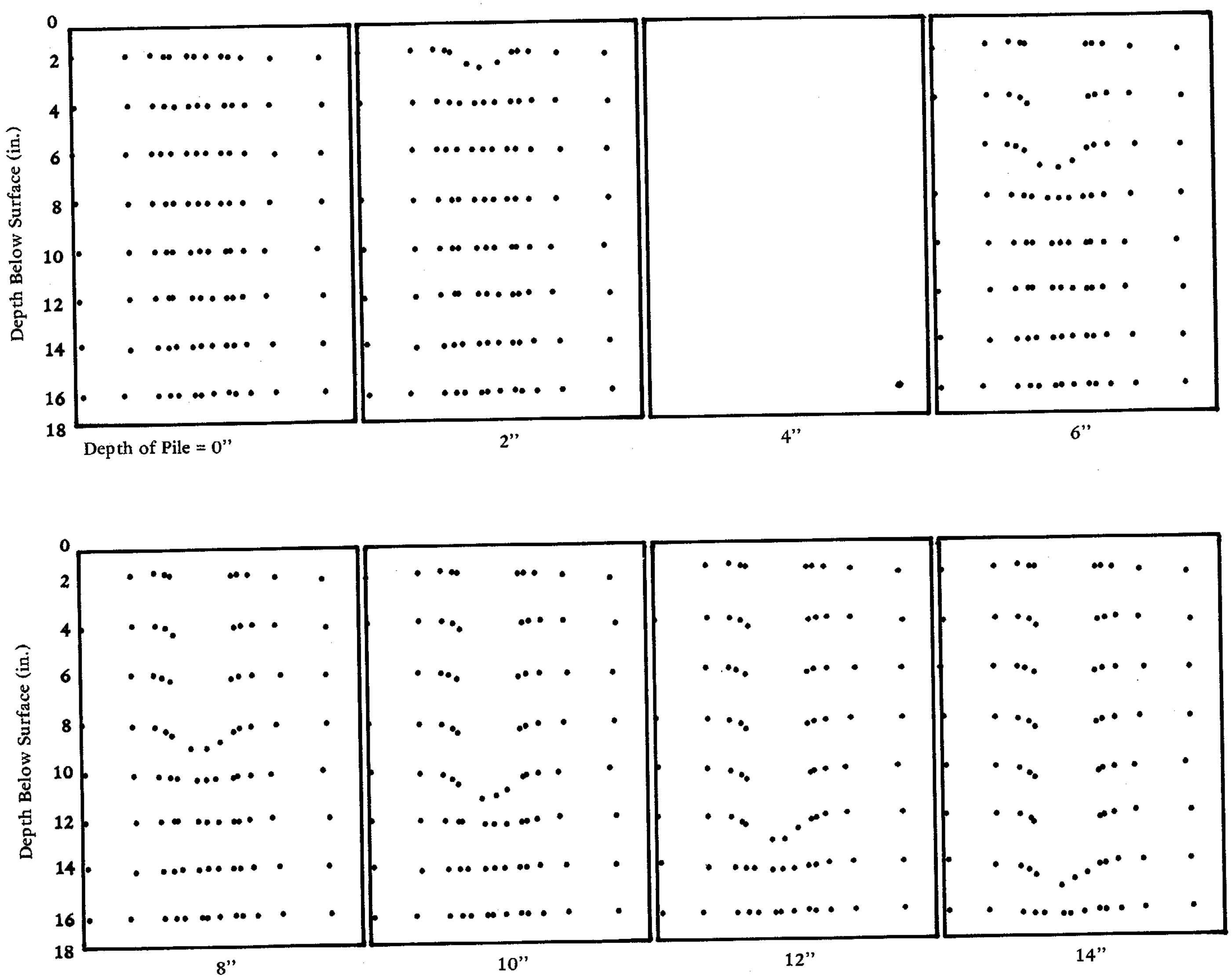


Figure 7. Complete x-ray suite for calcareous sand, low void ratio, and 0% cement.

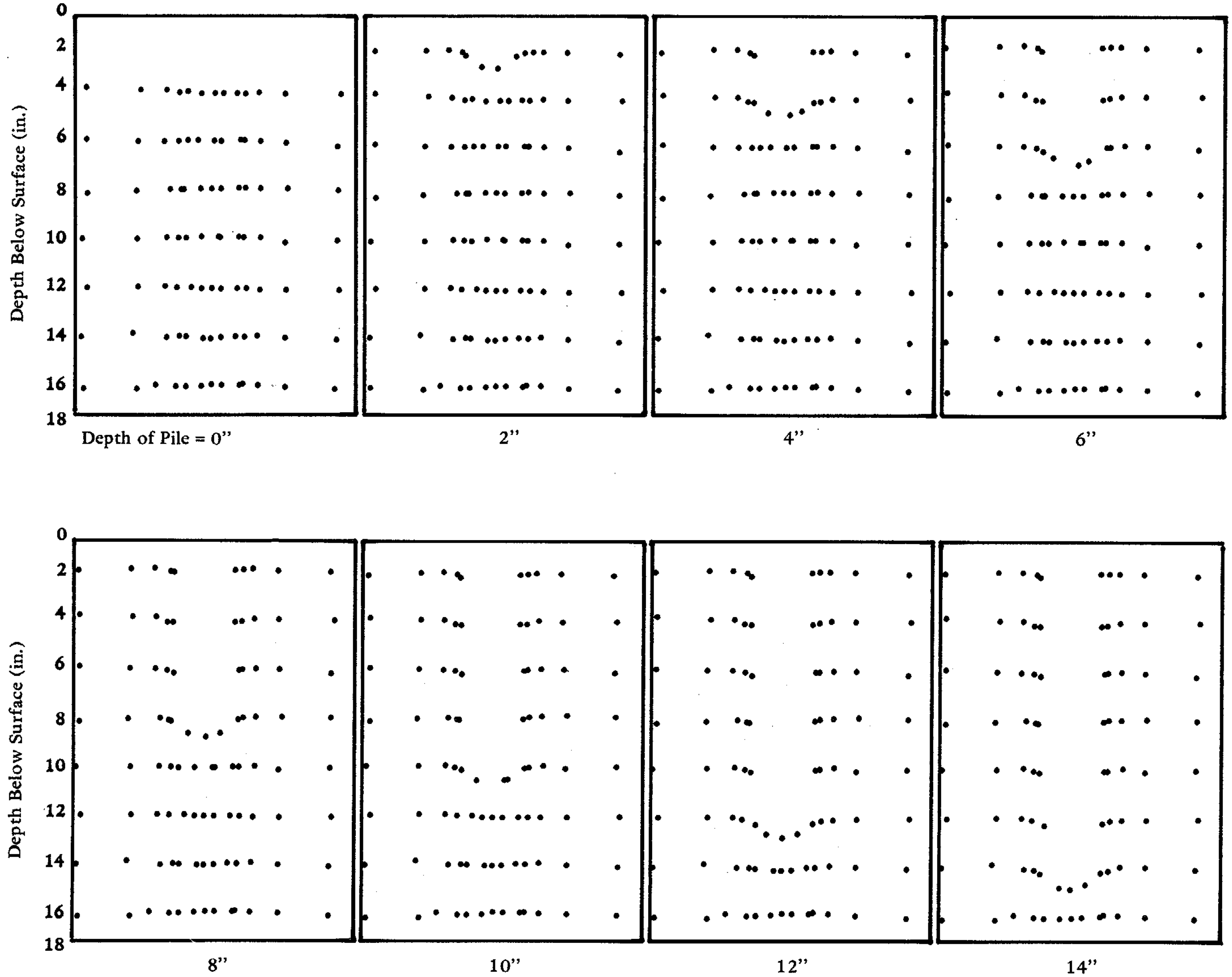


Figure 8. Complete x-ray suite for silica sand, high void ratio, and 0% cement.

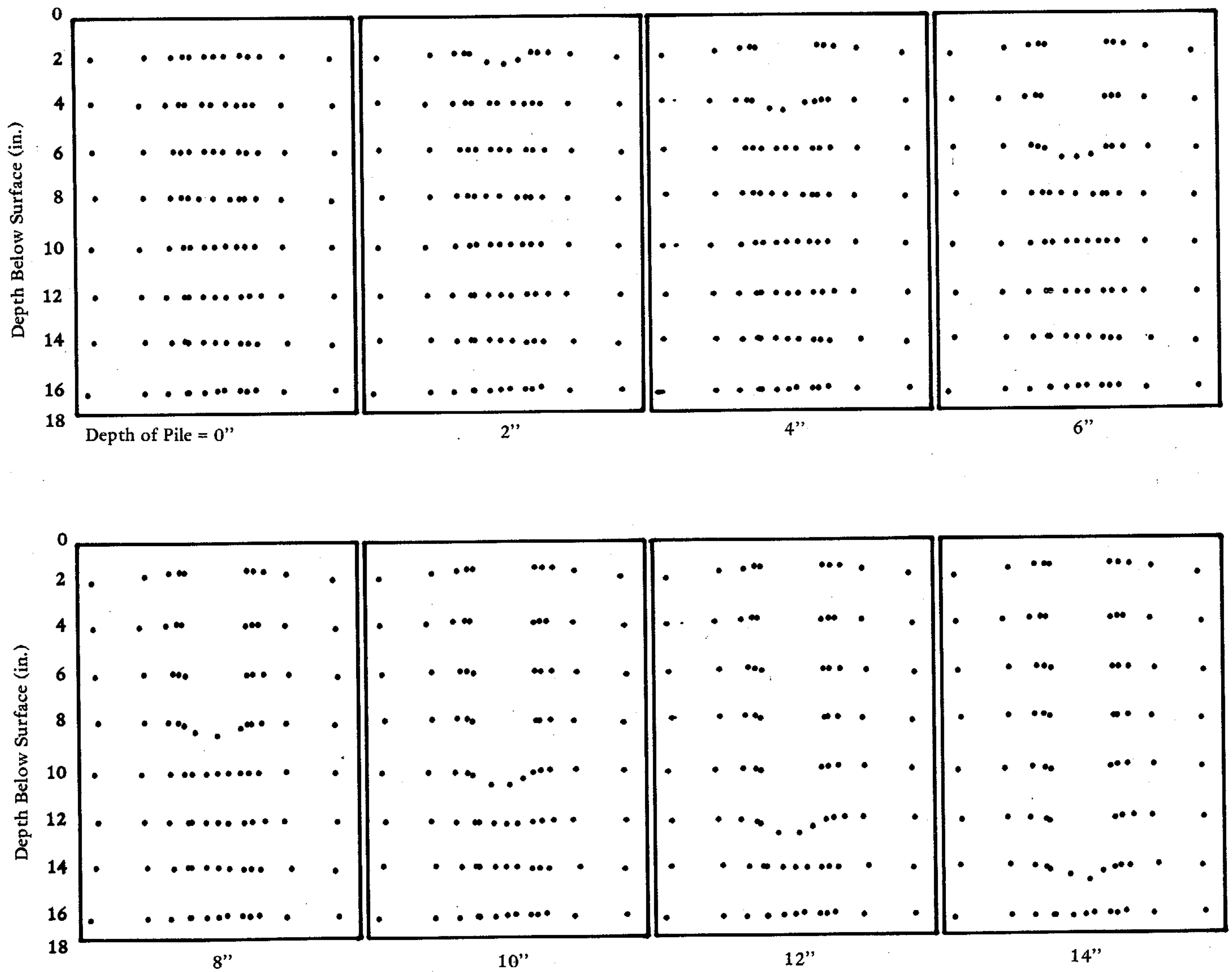


Figure 9. Complete x-ray suite for silica sand, low void ratio, and 0% cement.

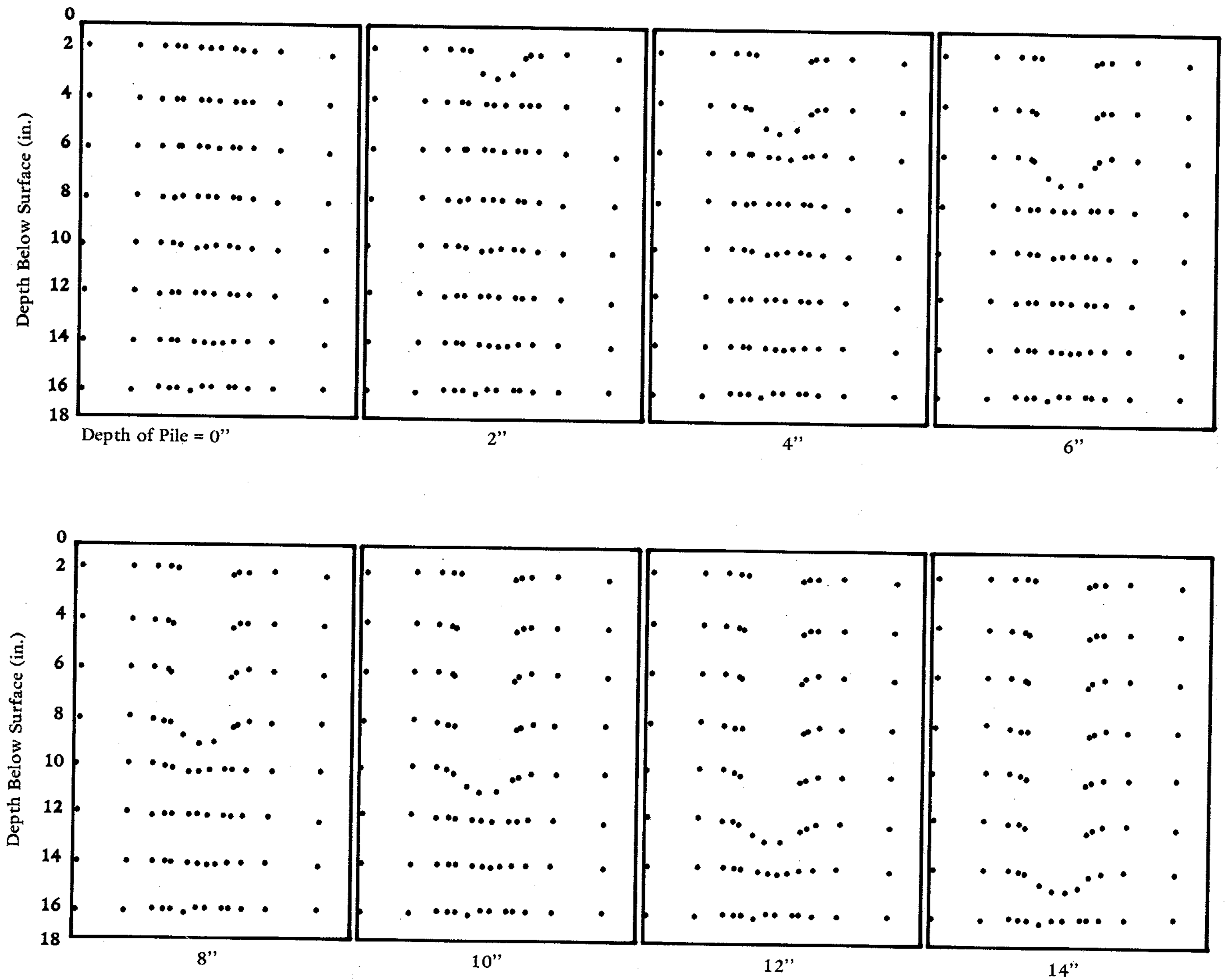


Figure 10. Complete x-ray suite for calcareous sand, high void ratio, and 1% cement.

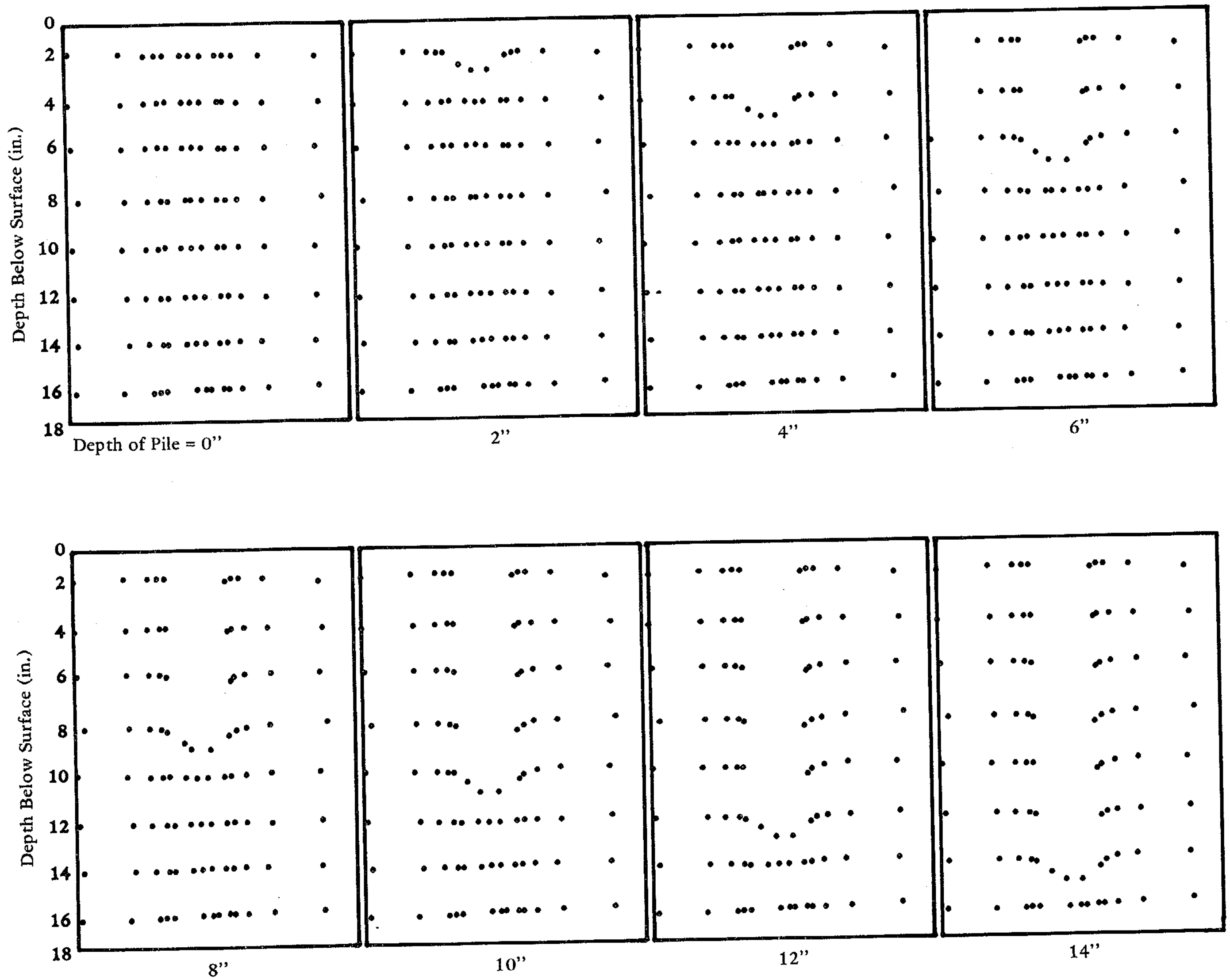


Figure 11. Complete x-ray suite for calcareous sand, low void ratio, and 1% cement.

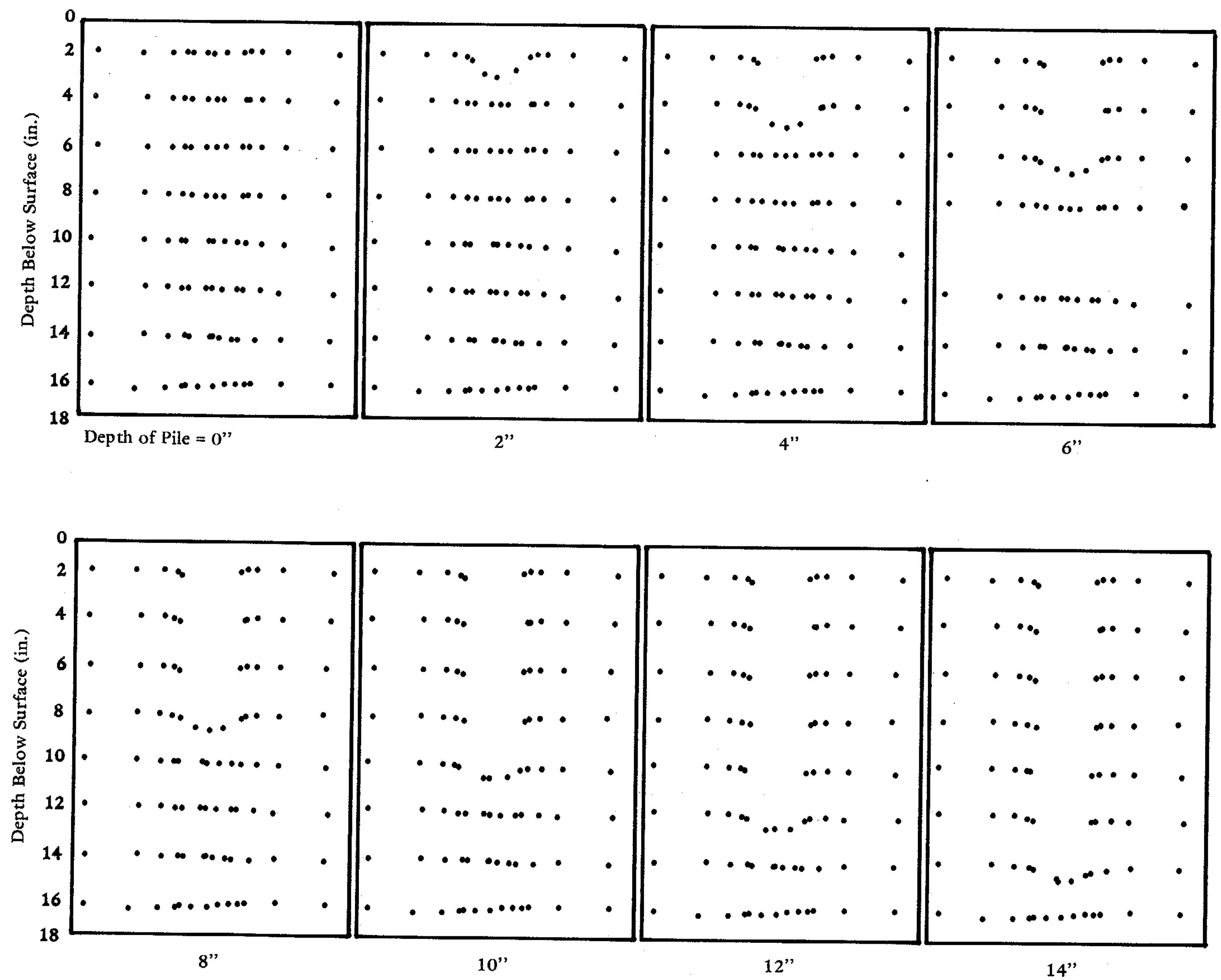


Figure 12. Complete x-ray suite for silica sand, high void ratio, and 1/2% cement.

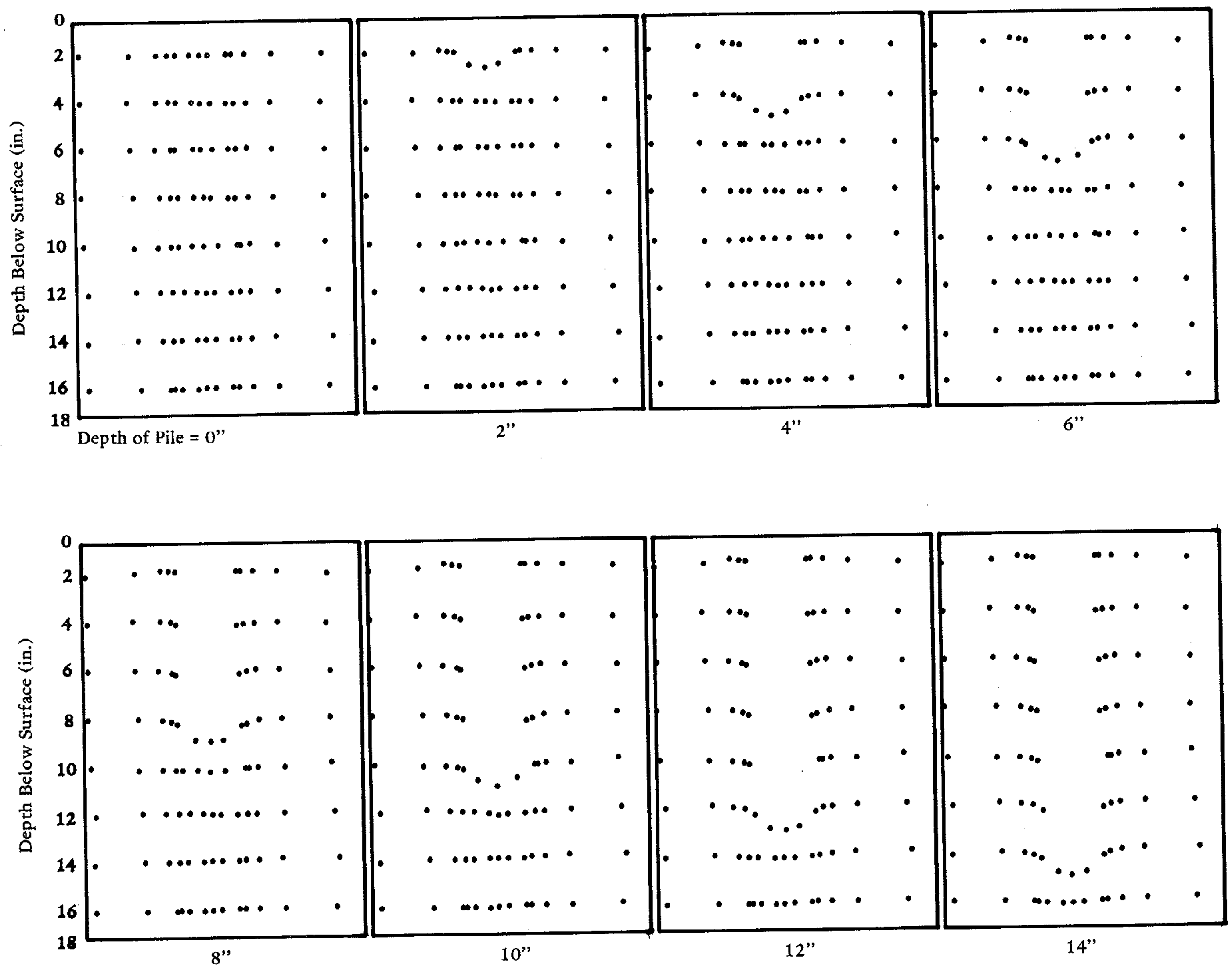


Figure 13. Complete x-ray suite for silica sand, low void ratio, and 1/2% cement.

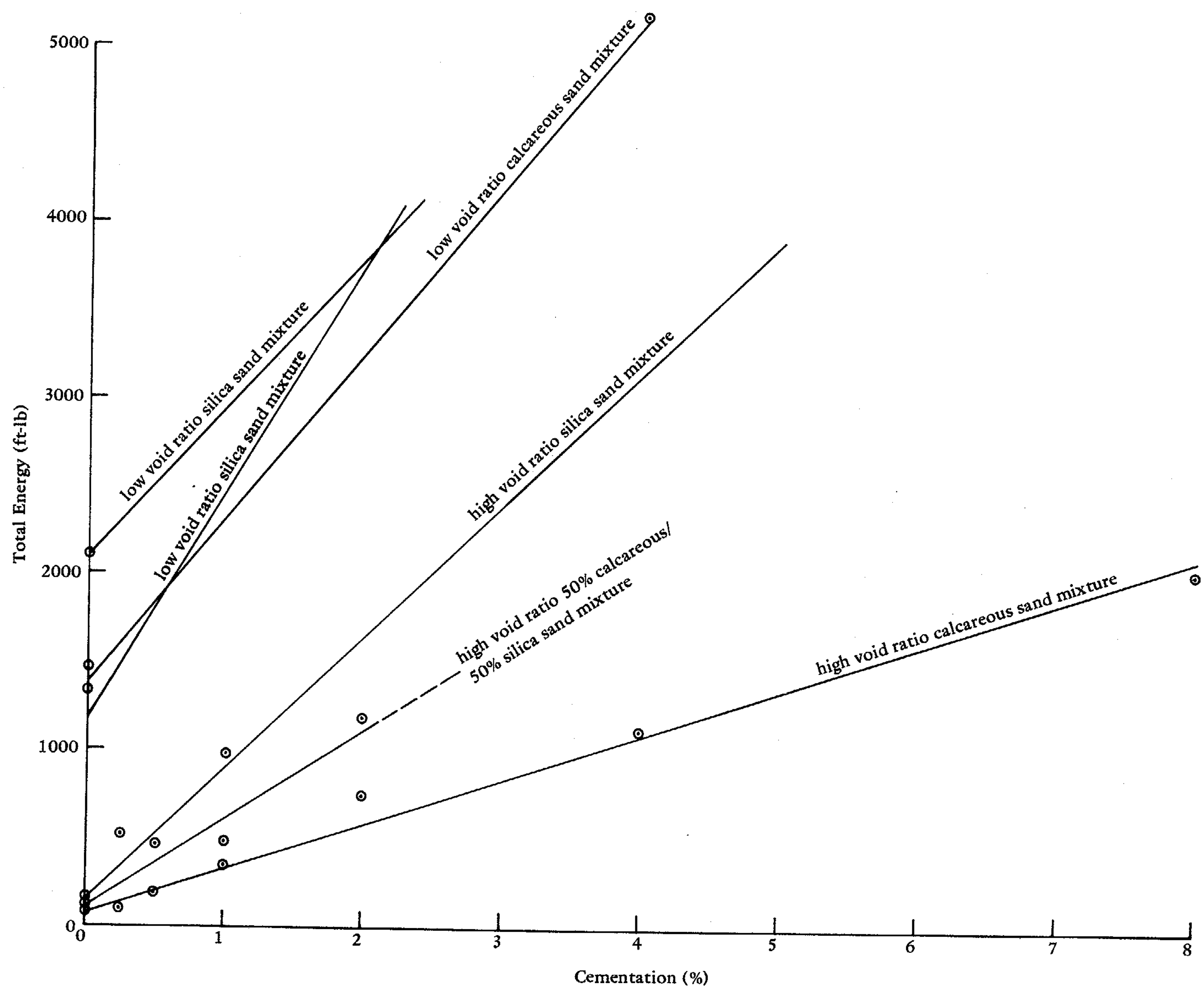
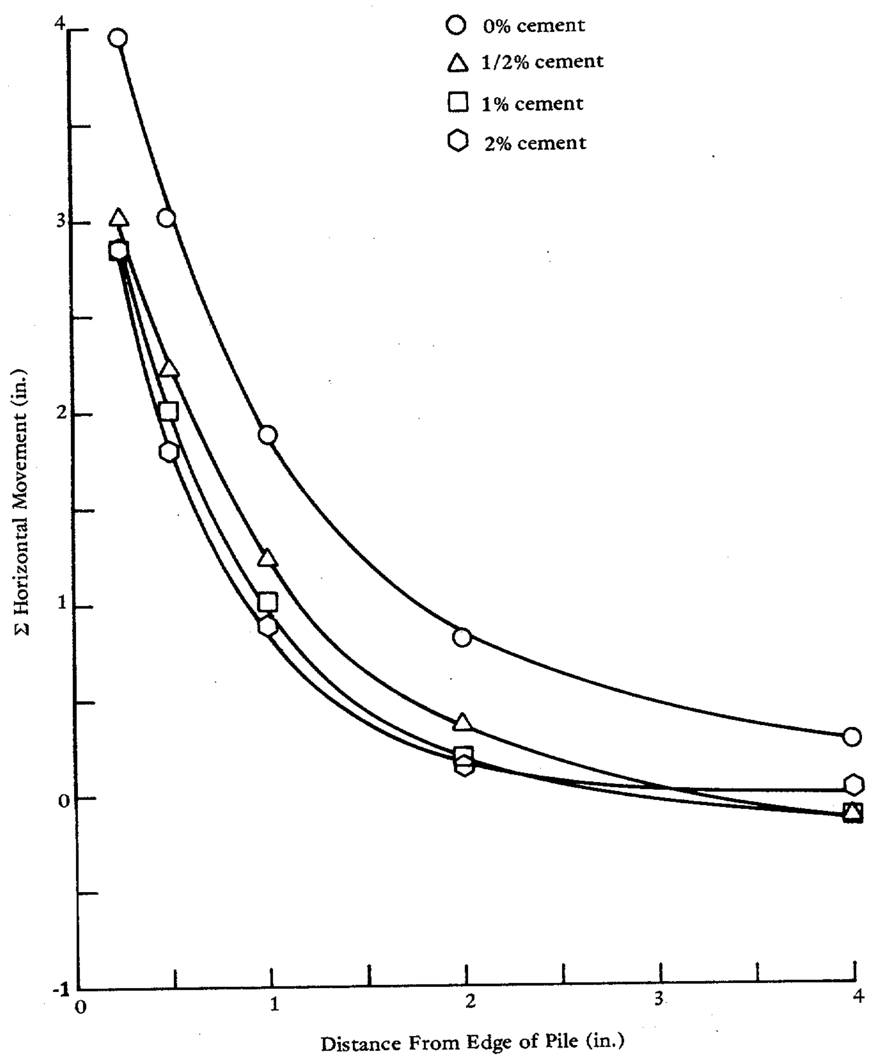
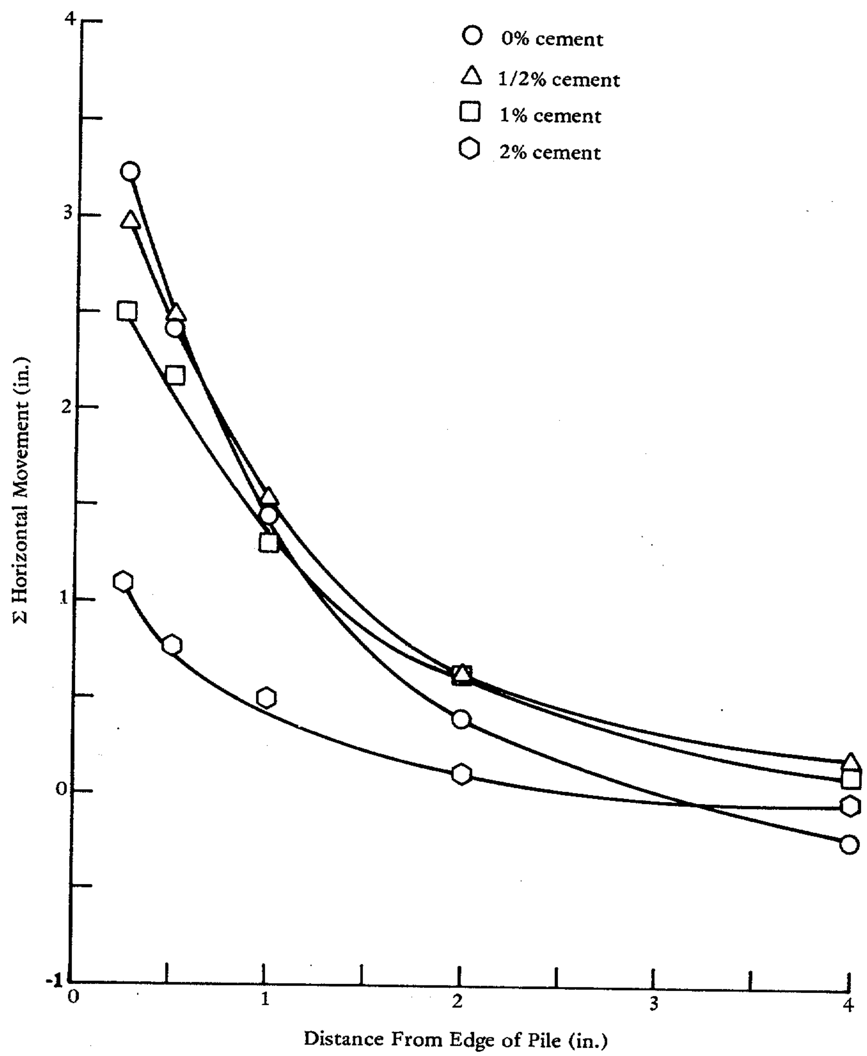


Figure 14. Cementation versus total energy (NCEL program).



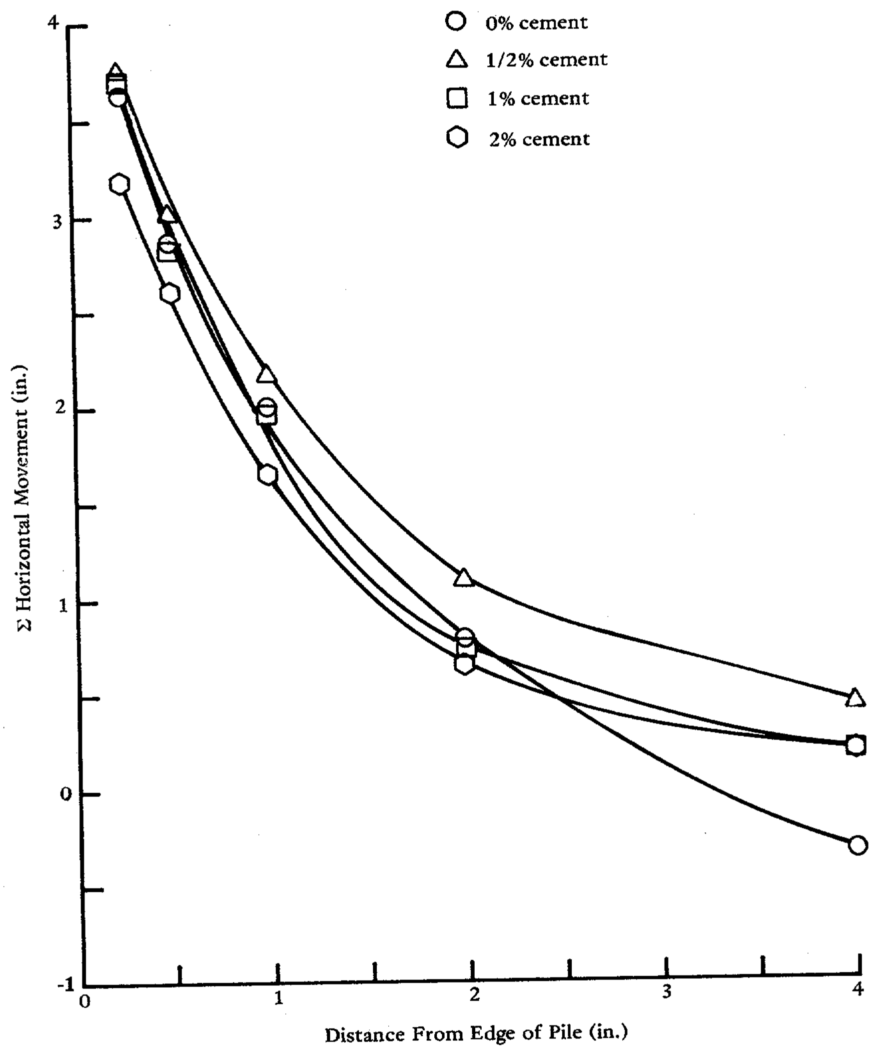
(a) Loose Calcareous Sand

Figure 15. Horizontal soil displacements after pile driving in calcareous and silica sands.



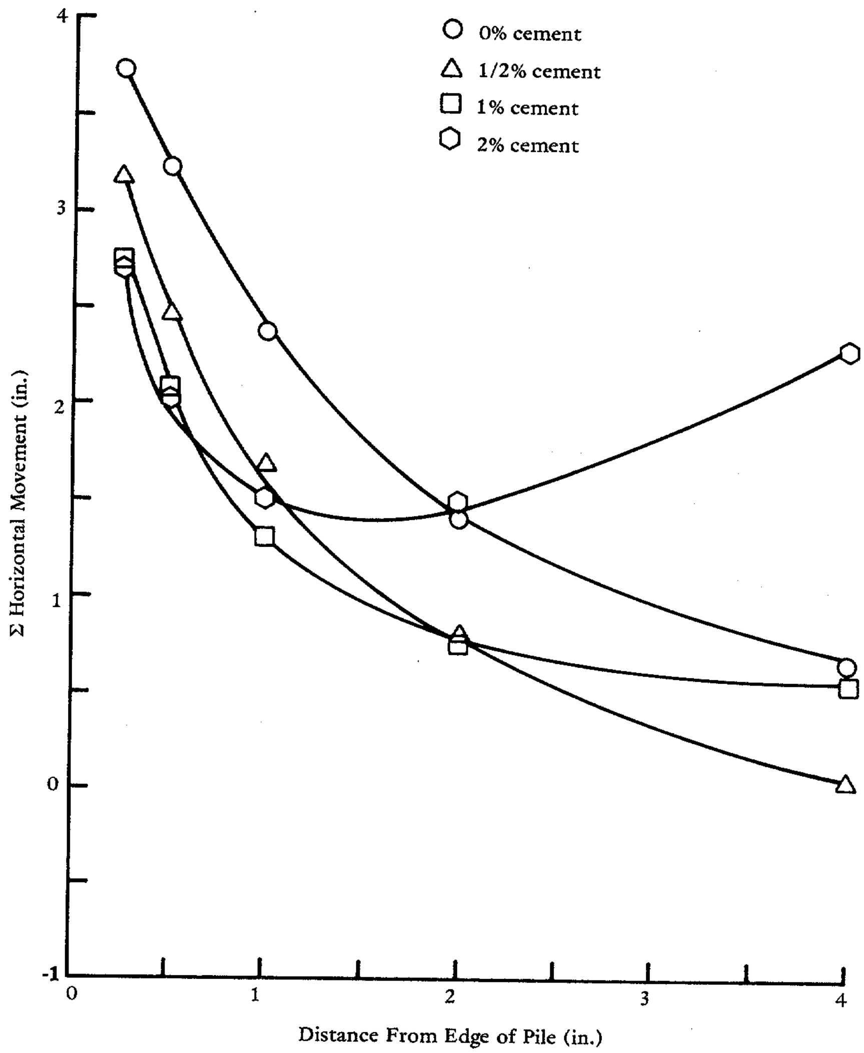
(b) Loose Silica Sand

Figure 15. Continued.



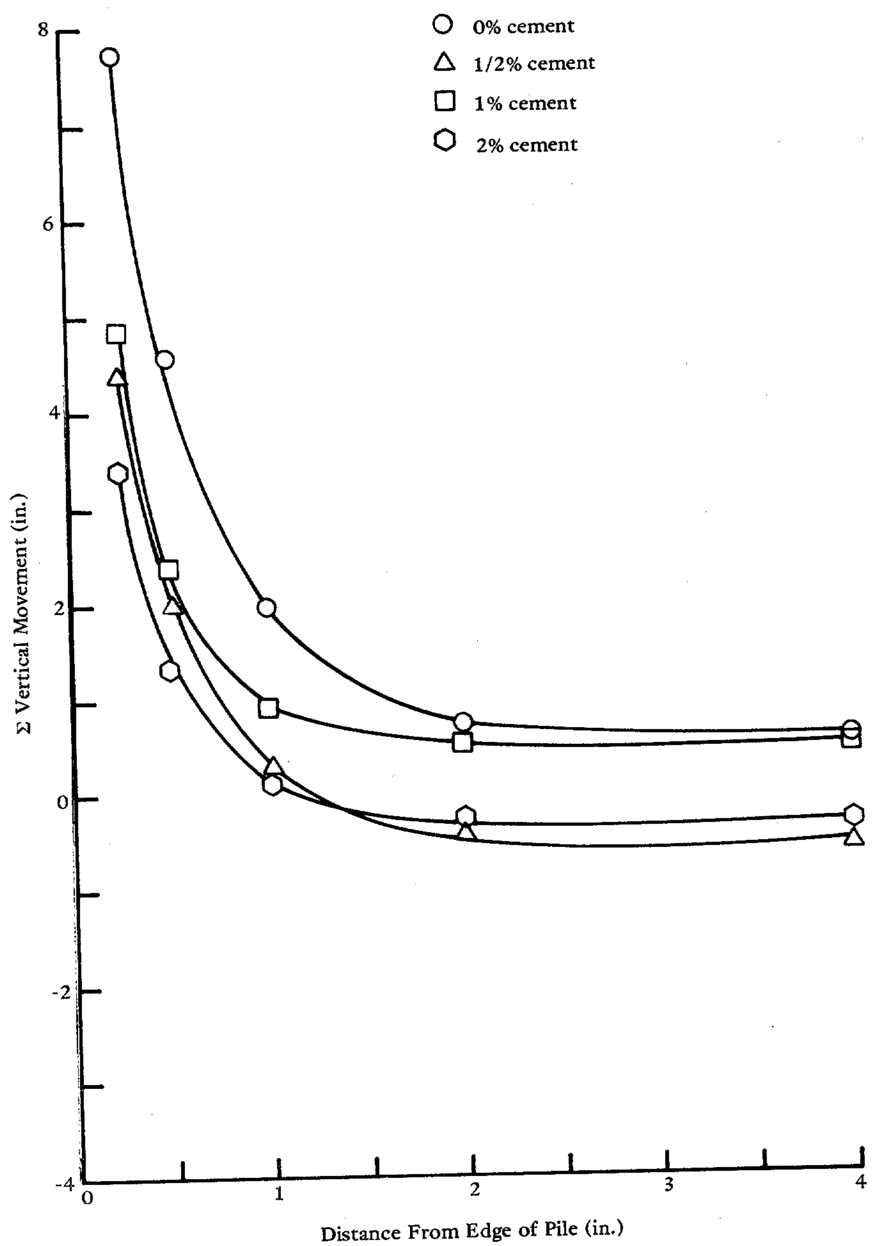
(c) Dense Calcareous Sand

Figure 15. Continued.



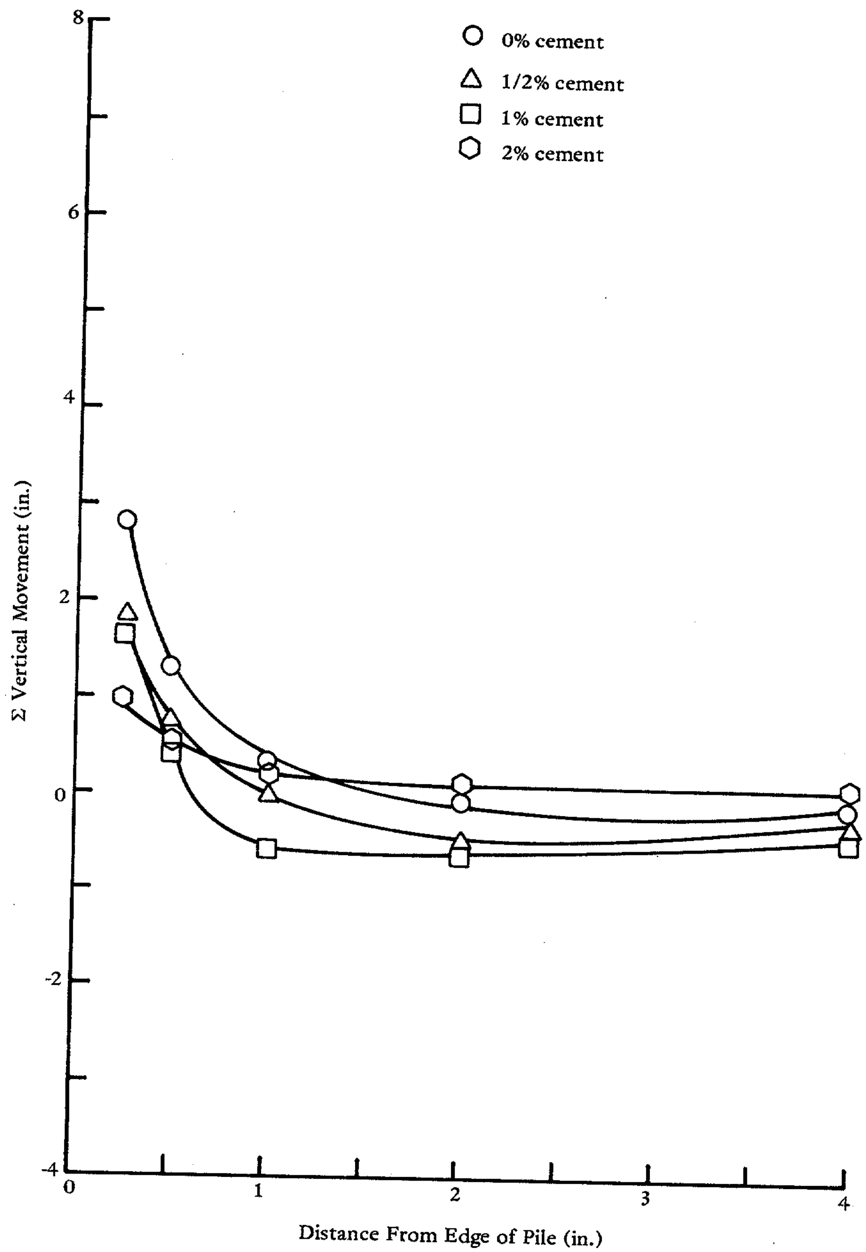
(d) Dense Silica Sand

Figure 15. Continued.



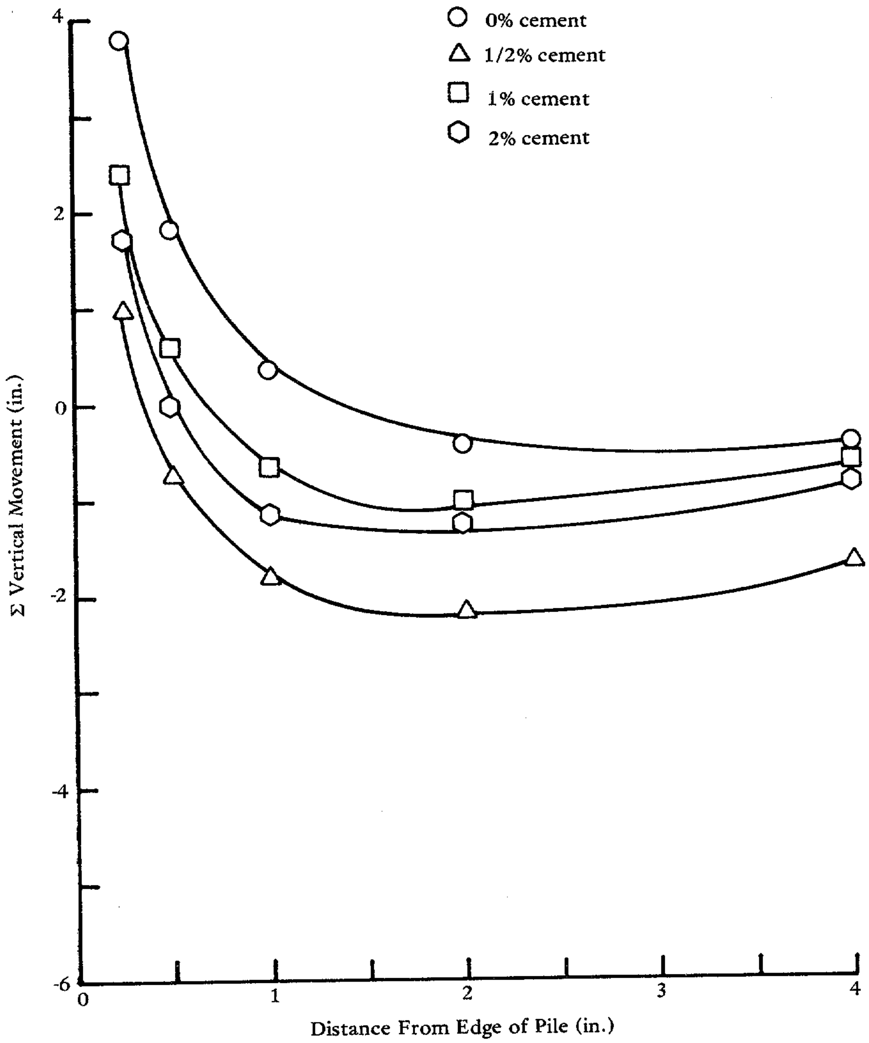
(a) Loose Calcareous Sand

Figure 16. Vertical soil displacement after pile driving in calcareous and silica sand.



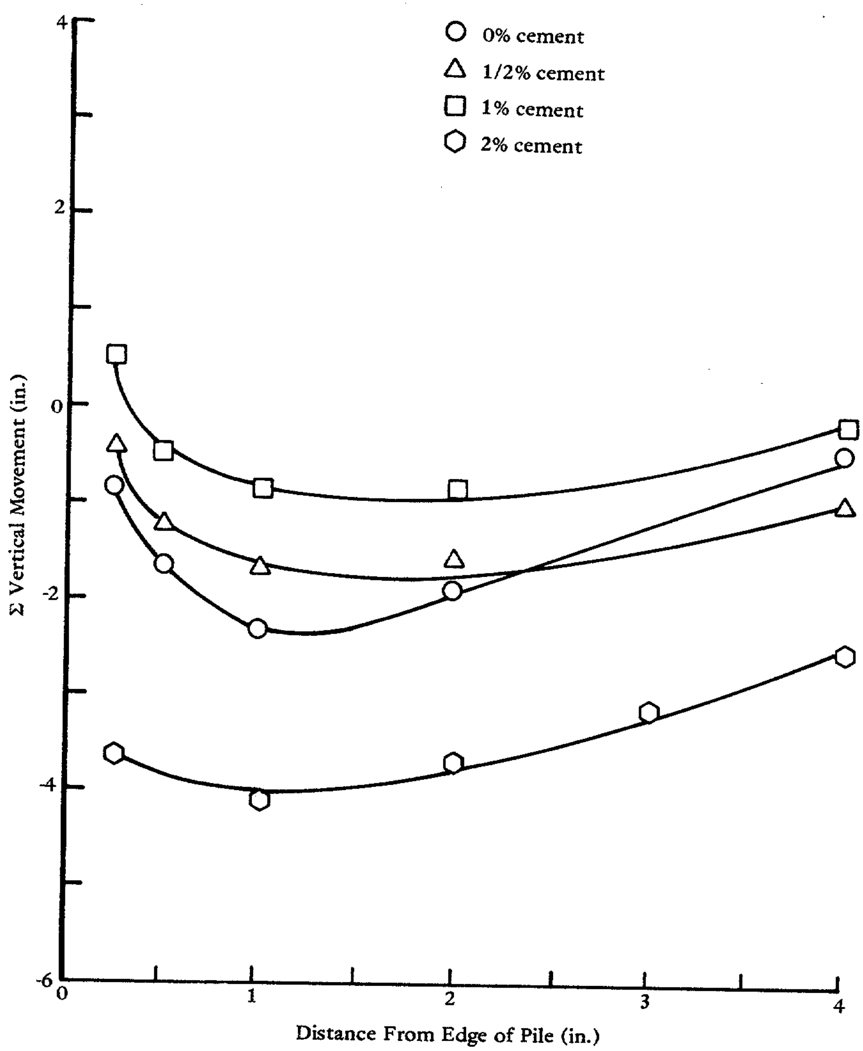
(b) Loose Silica Sand

Figure 16. Continued.



(c) Dense Calcareous Sand

Figure 16. Continued.



(d) Dense Silica Sand

Figure 16. Continued.

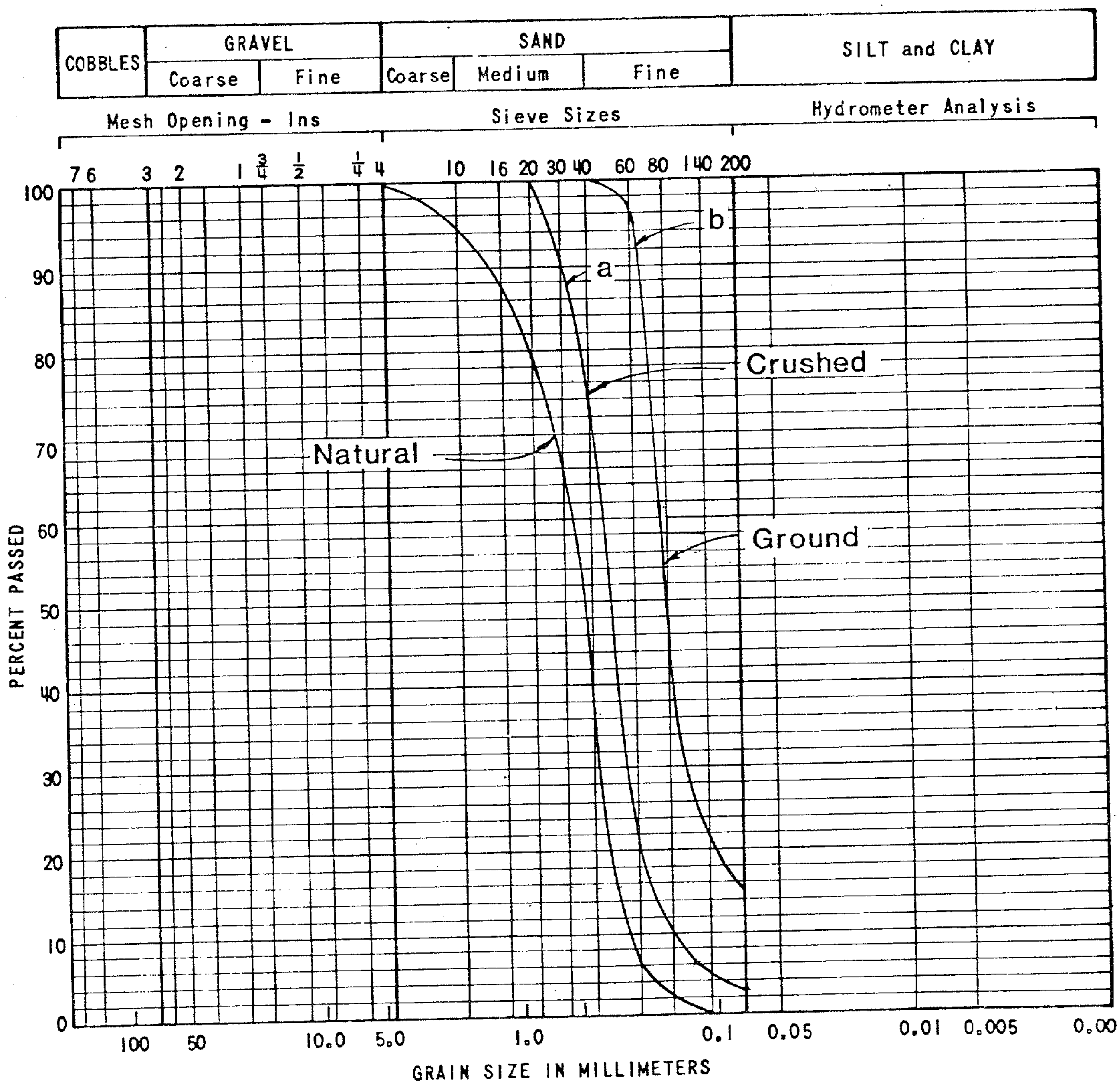


Figure 17. Grain size distribution curves for crushed and natural calcareous sand from Guam.

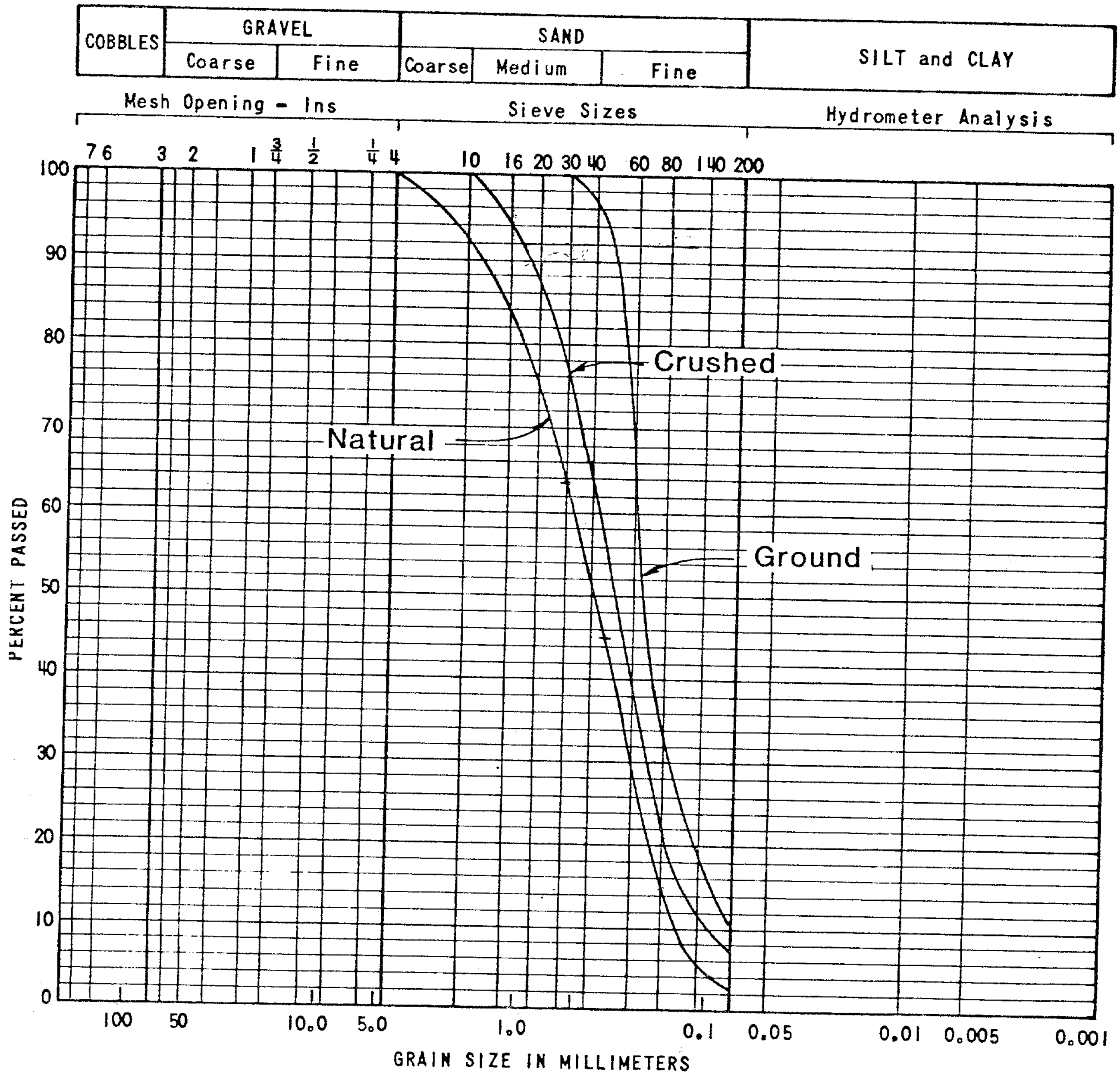


Figure 18. Grain size distribution curves for crushed and natural calcareous sand from Florida.

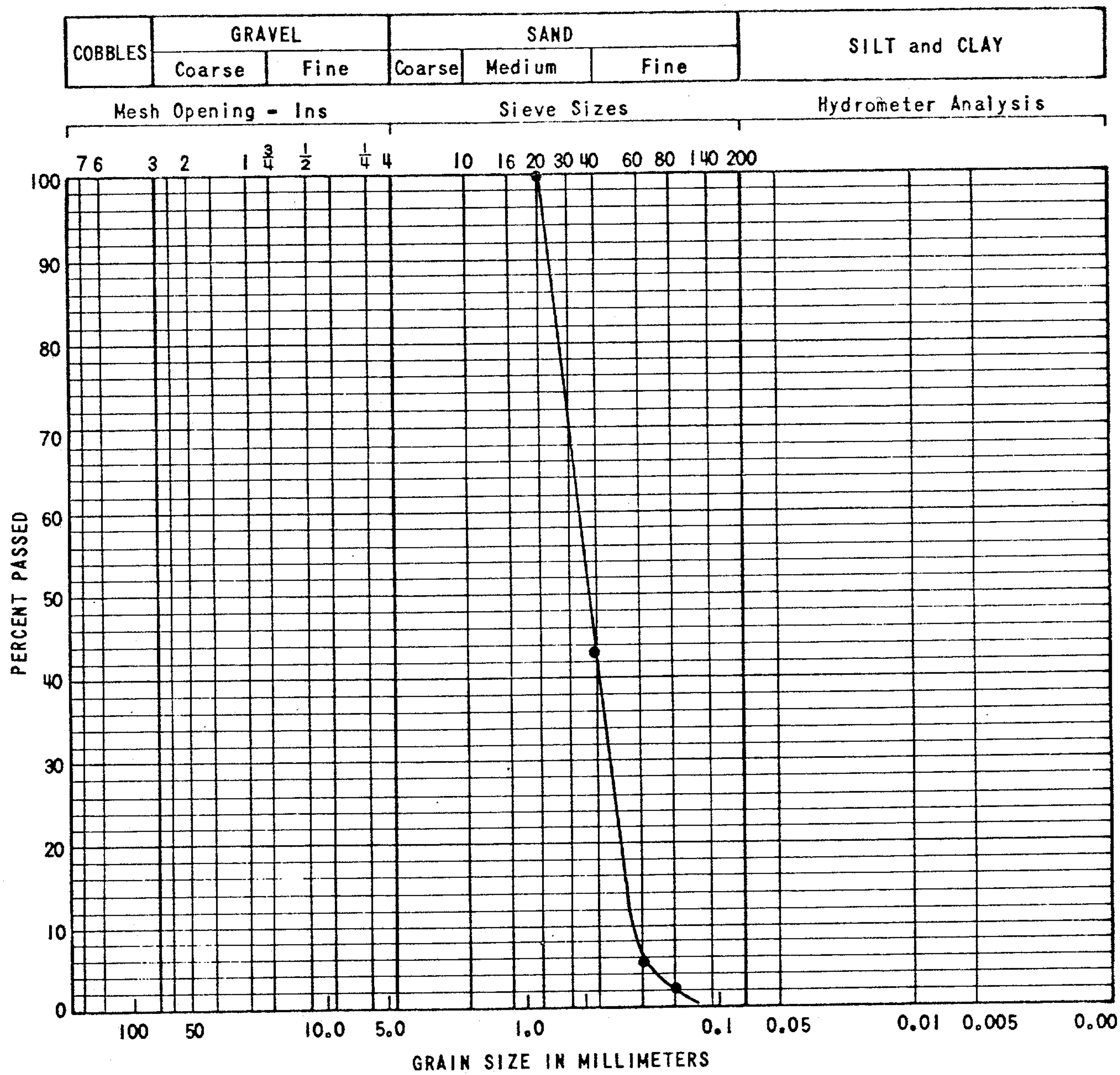


Figure 19. Grain size distribution curve of Ottawa sand.

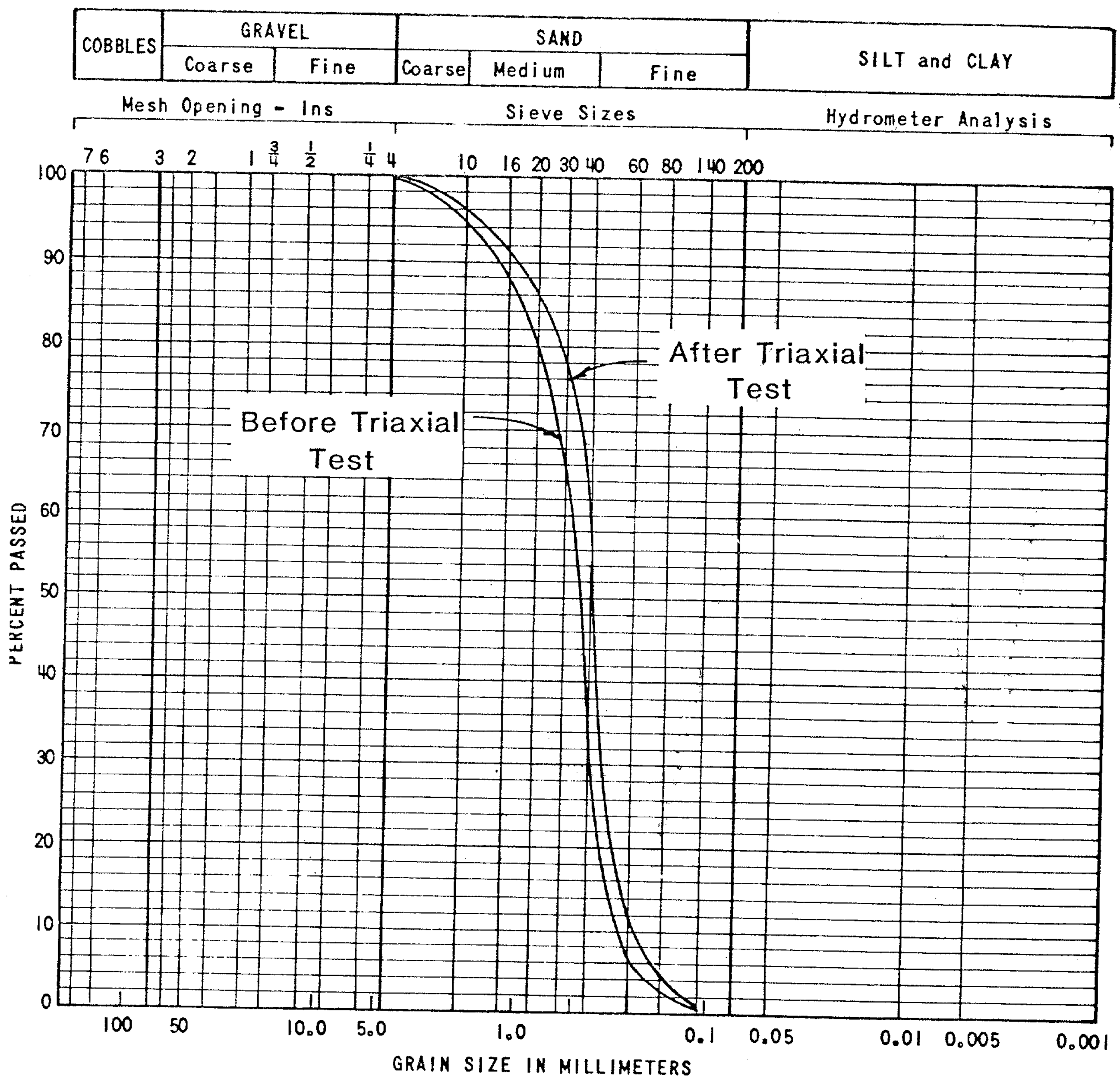


Figure 20. Grain size distribution curves of calcareous sand before and after triaxial compression test.

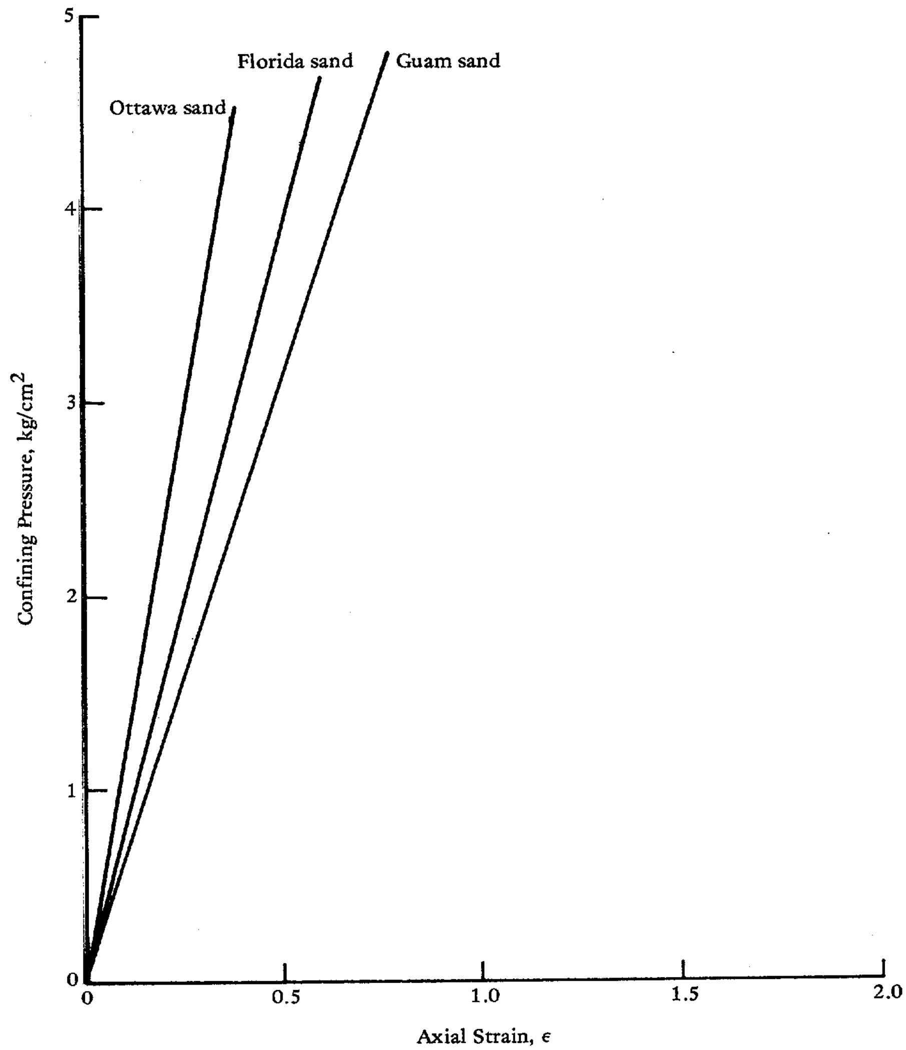


Figure 21. Stress-strain diagram for isotropic compression of silica and calcareous sands.

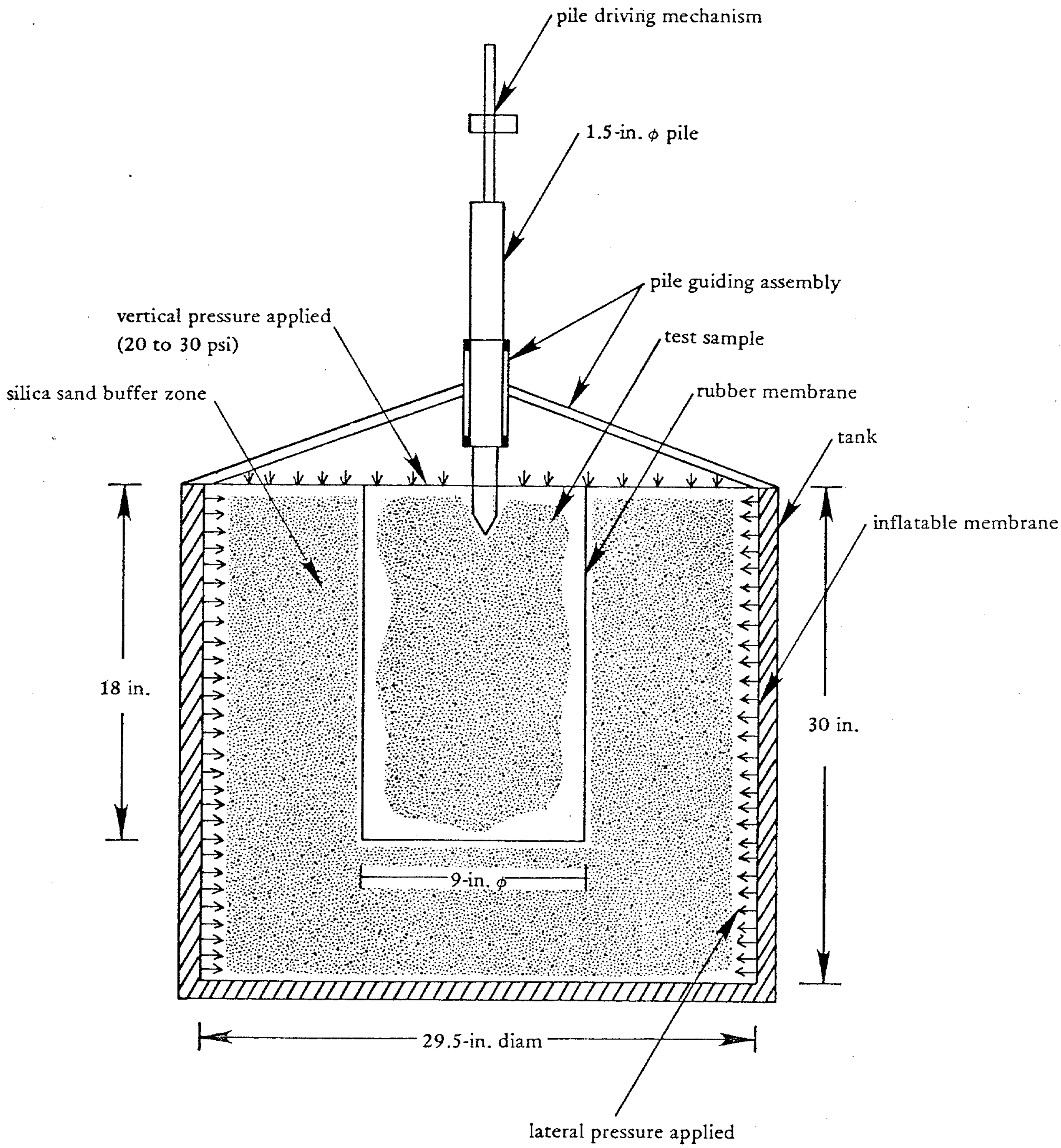
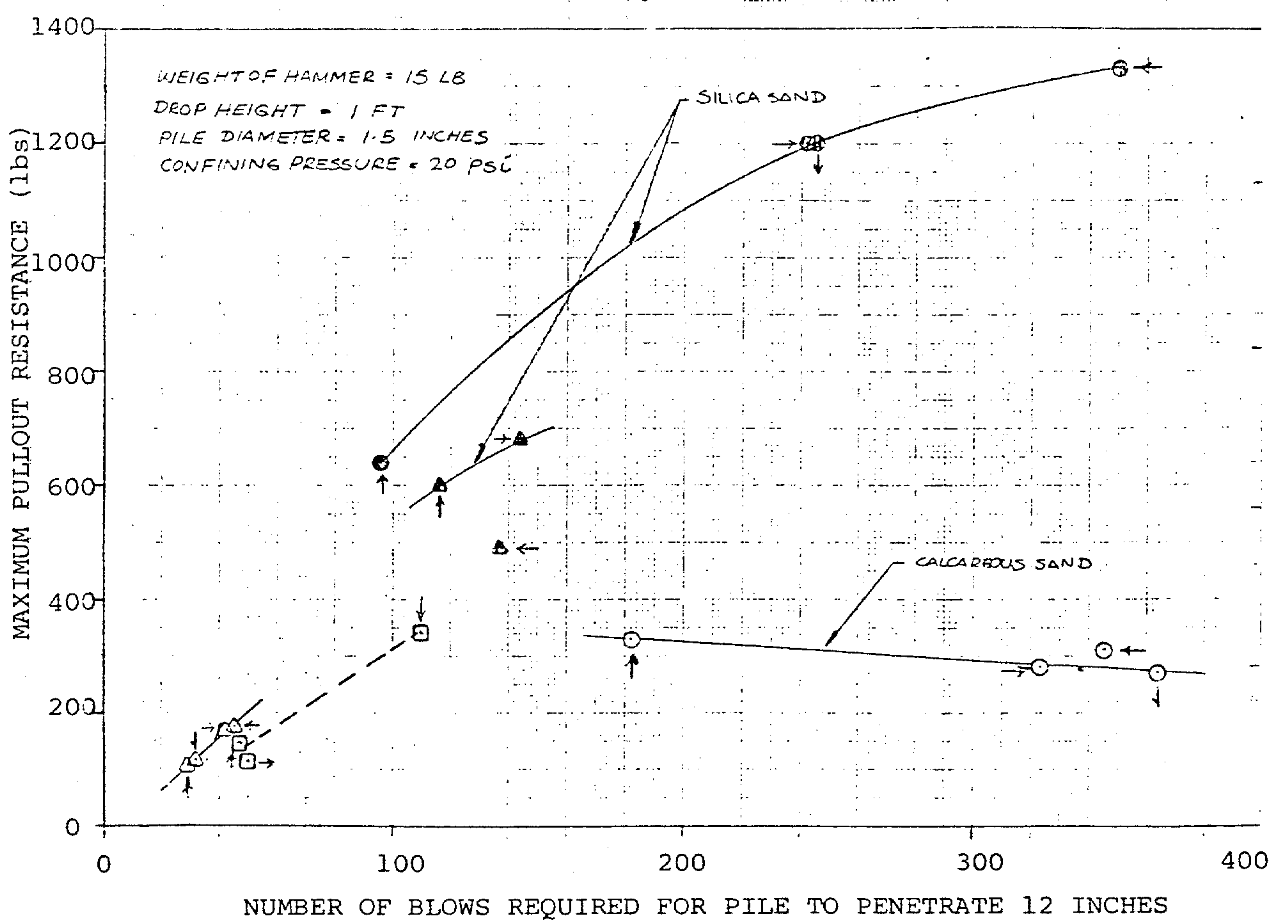


Figure 22. Diagram showing pile load test on the calcareous and silica sand.



SYMBOL	SAND TYPE	DRY DENSITY(g/cm)	PREPARATION METHOD
⊙	Calcareous	1.17	Low void ratio
⊡	Calcareous	1.07	Low void ratio
△	Calcareous	0.94	High void ratio
●	Silica	1.61	Low void ratio
▲	Silica	1.51	High void ratio

NOTE: Arrow directions indicate cement contents:

↑ = 0%, → = 0.5%
↓ = 1.0%, ← = 2.0%

Figure 23. Maximum pullout resistance versus pile driving resistance.

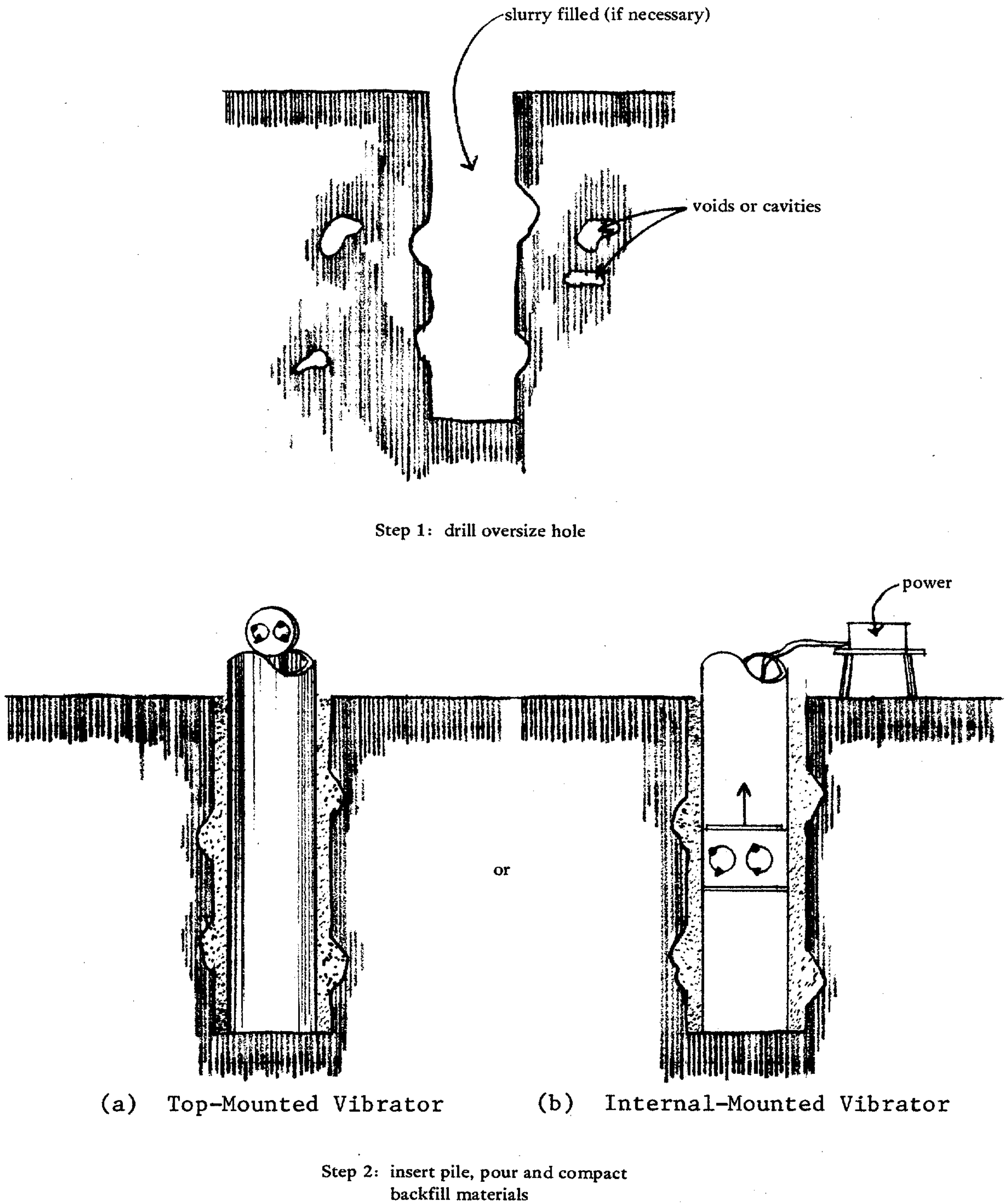
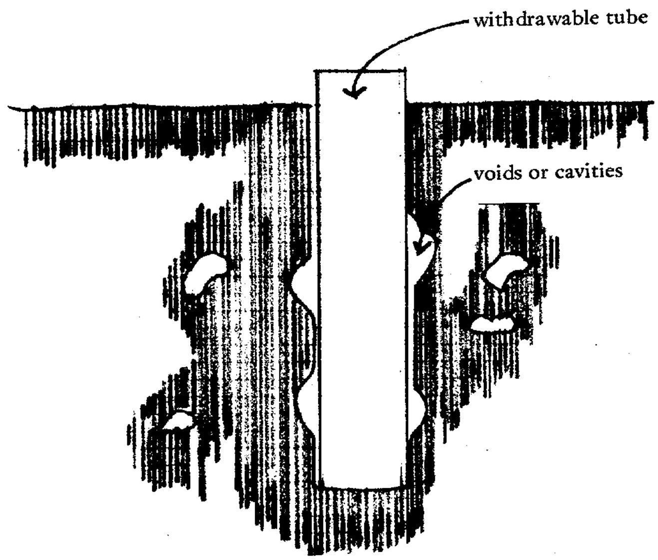
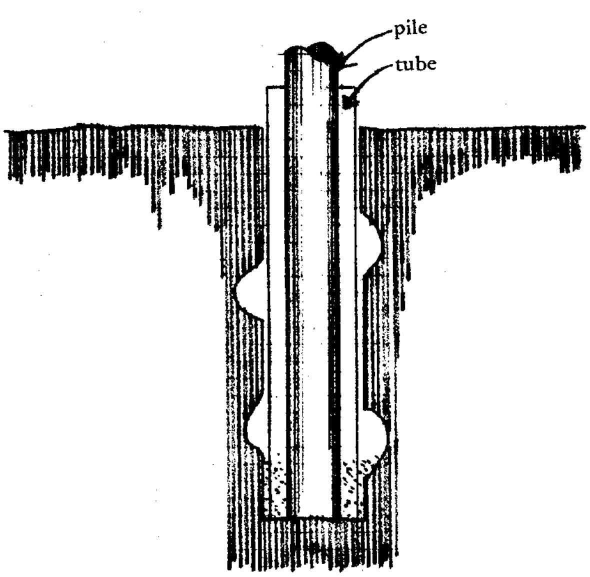


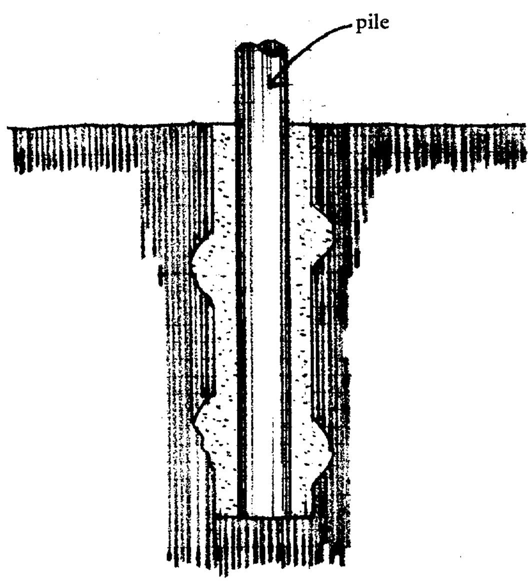
Figure 24. Conceptual drawing of a backfilled pile - Alternate A.



Step 1: drill oversize hole with withdrawable tube



Step 2: insert pile, backfill and compact while slowly removing tube



Step 3: installed pile

Figure 25. Conceptual drawing of a backfilled pile - Alternate B.

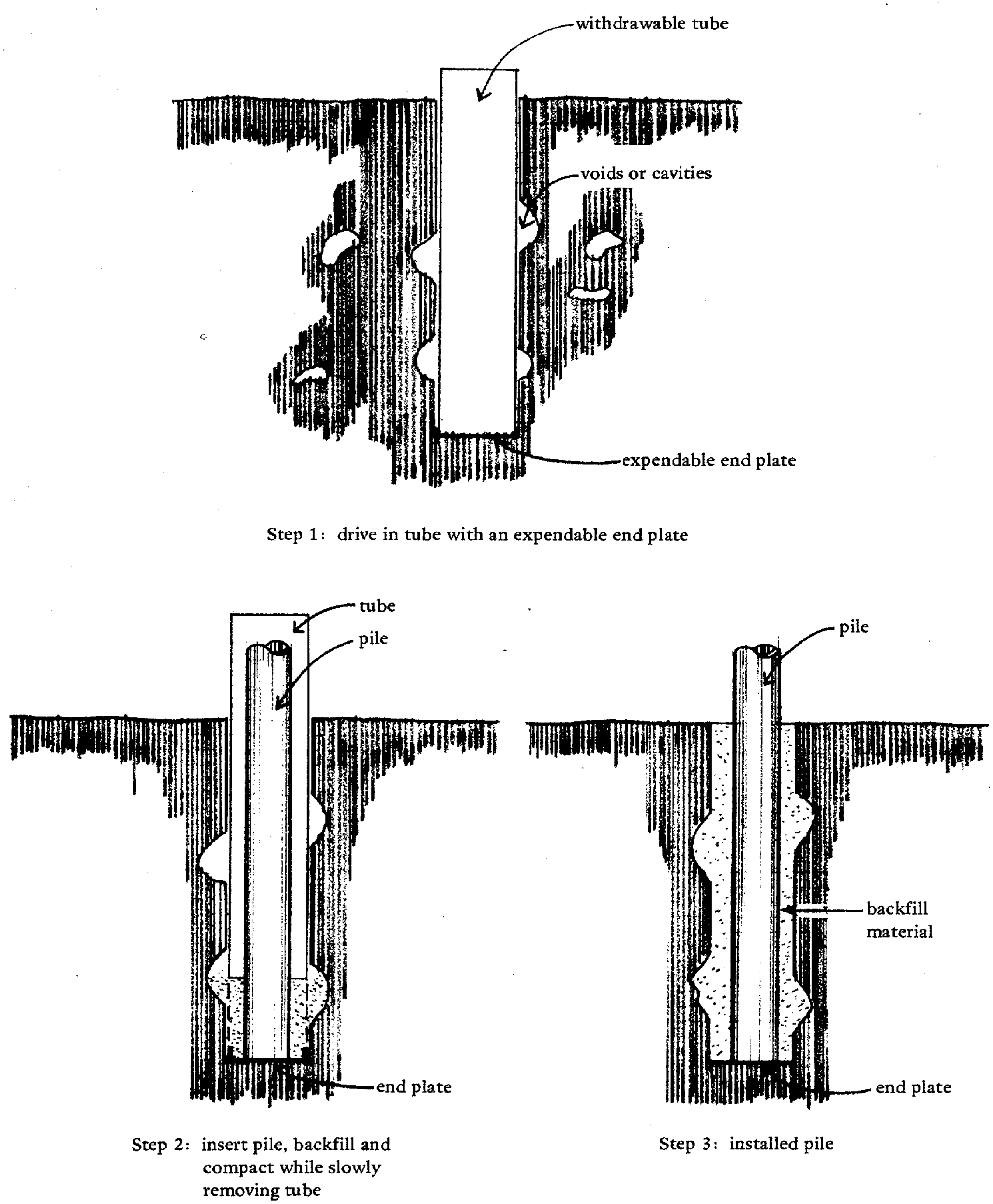
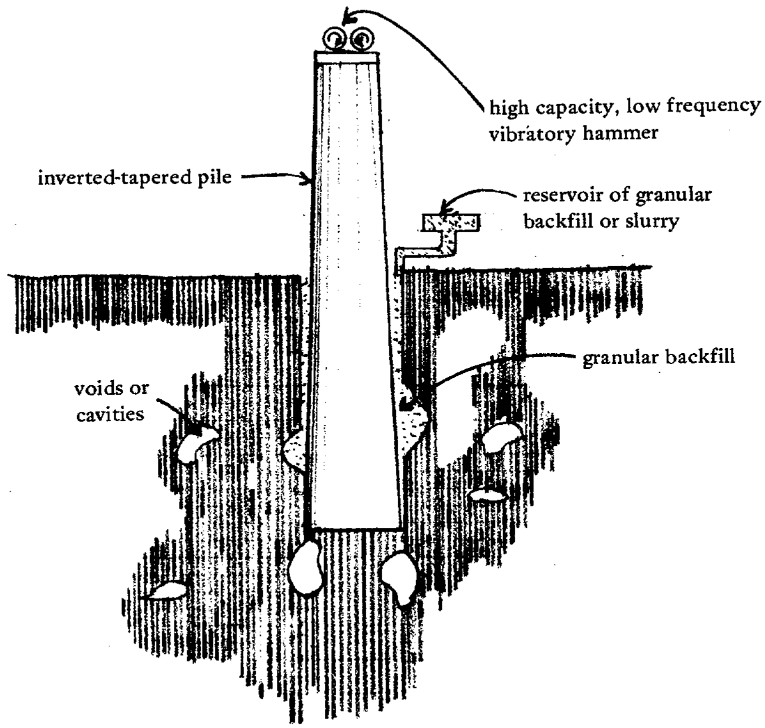
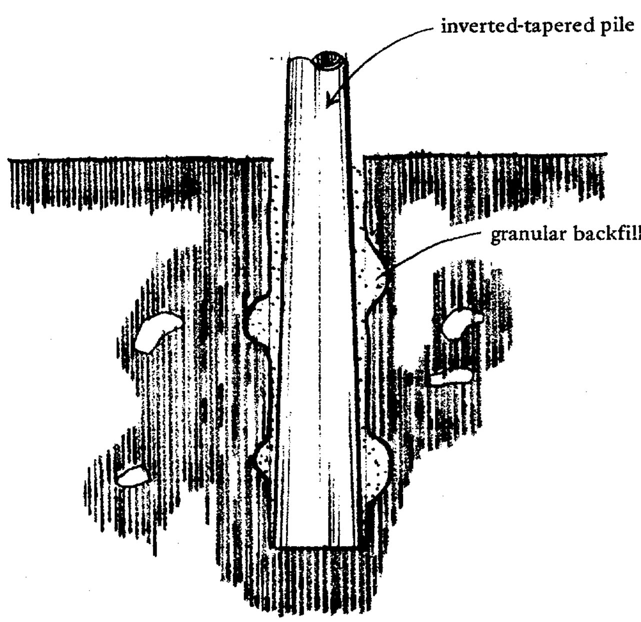


Figure 26. Conceptual drawing of a backfilled pile - Alternate C.

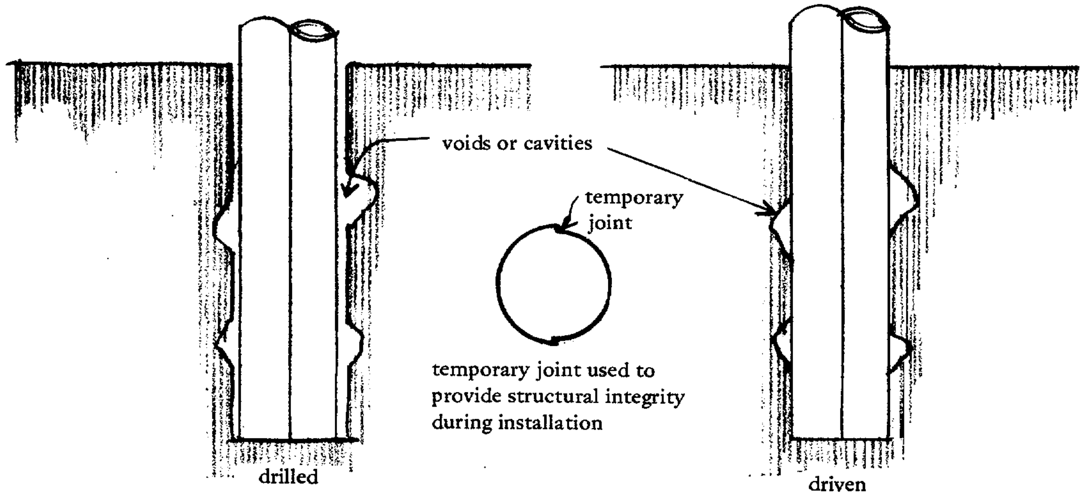


Step 1: install inverted tapered pile, backfill and compact with vibratory hammer

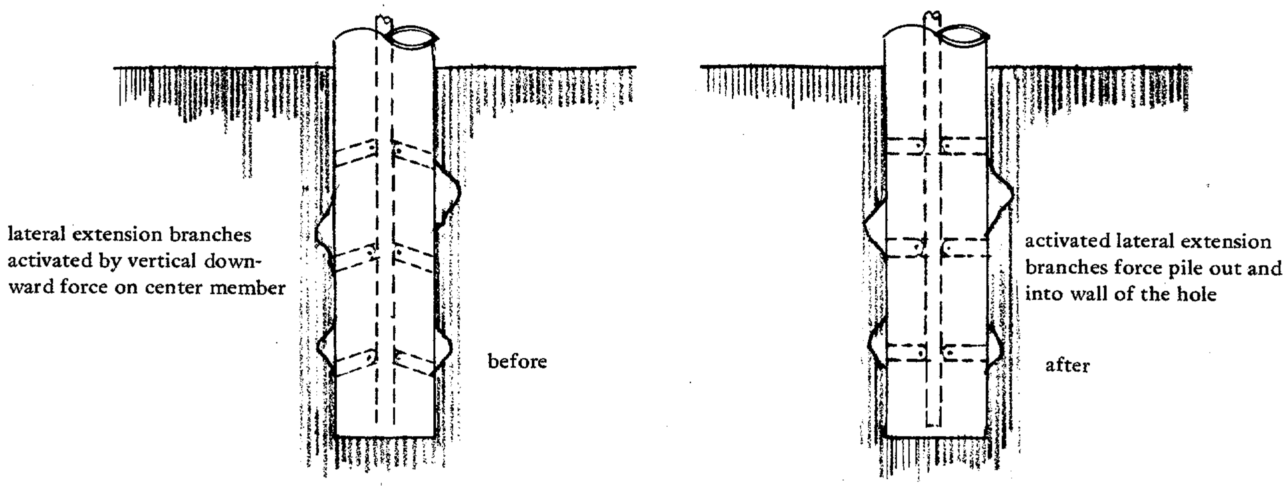


Step 2: installed pile

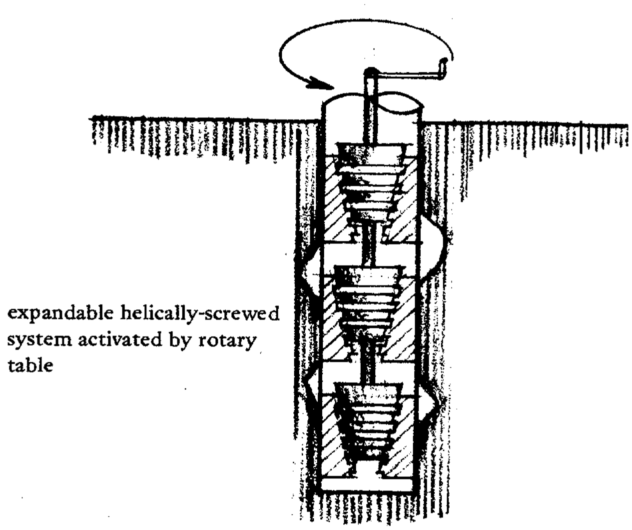
Figure 27. Conceptual drawing of vibratory-installed backfilled piles.



(a) Mechanical System



(b) Lateral Extension Branches System



(c) Helically-Screwed System

Figure 28. Conceptual drawing of a pressurized pile - Alternate A.

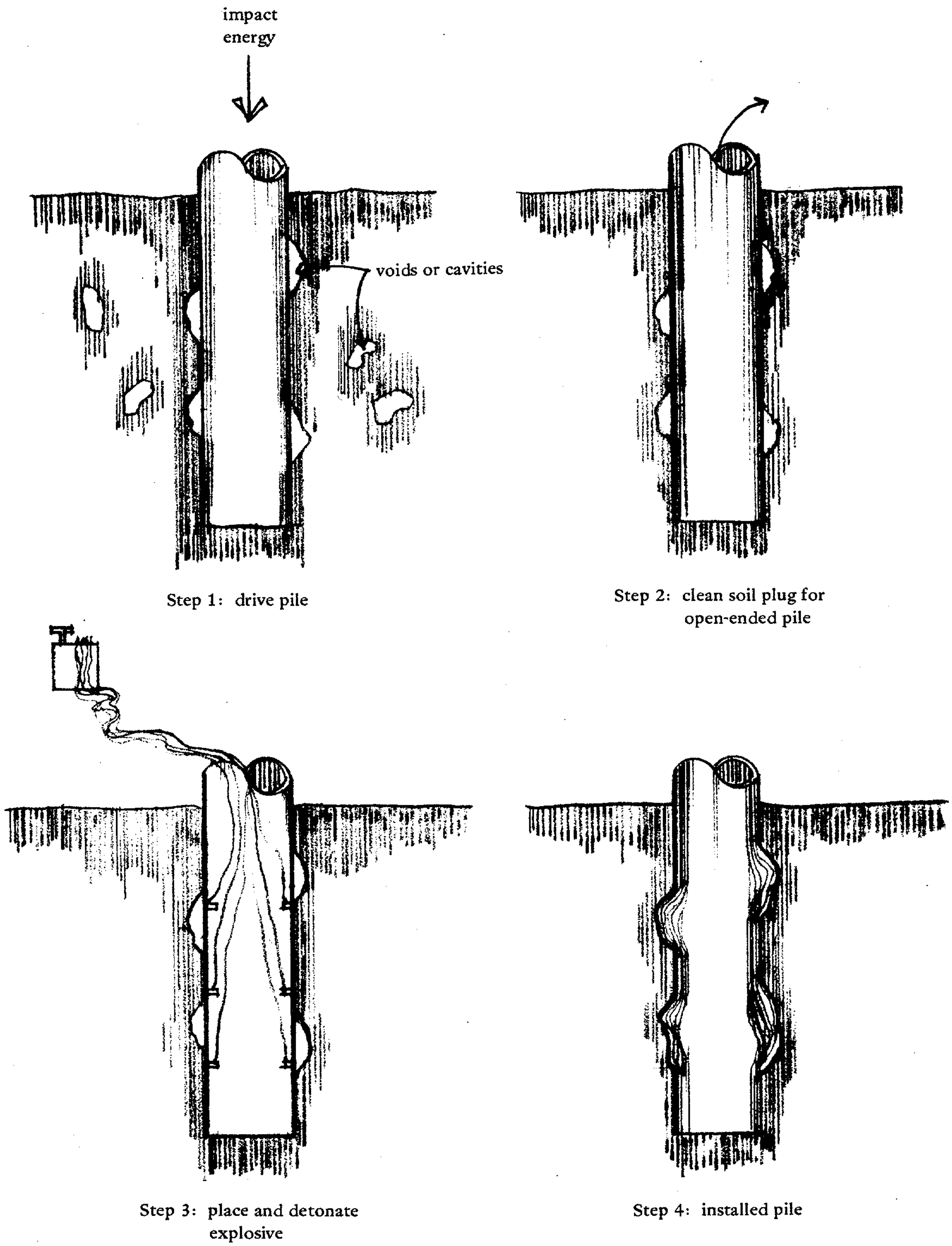


Figure 29. Conceptual drawing of a pressurized pile - Alternate B.

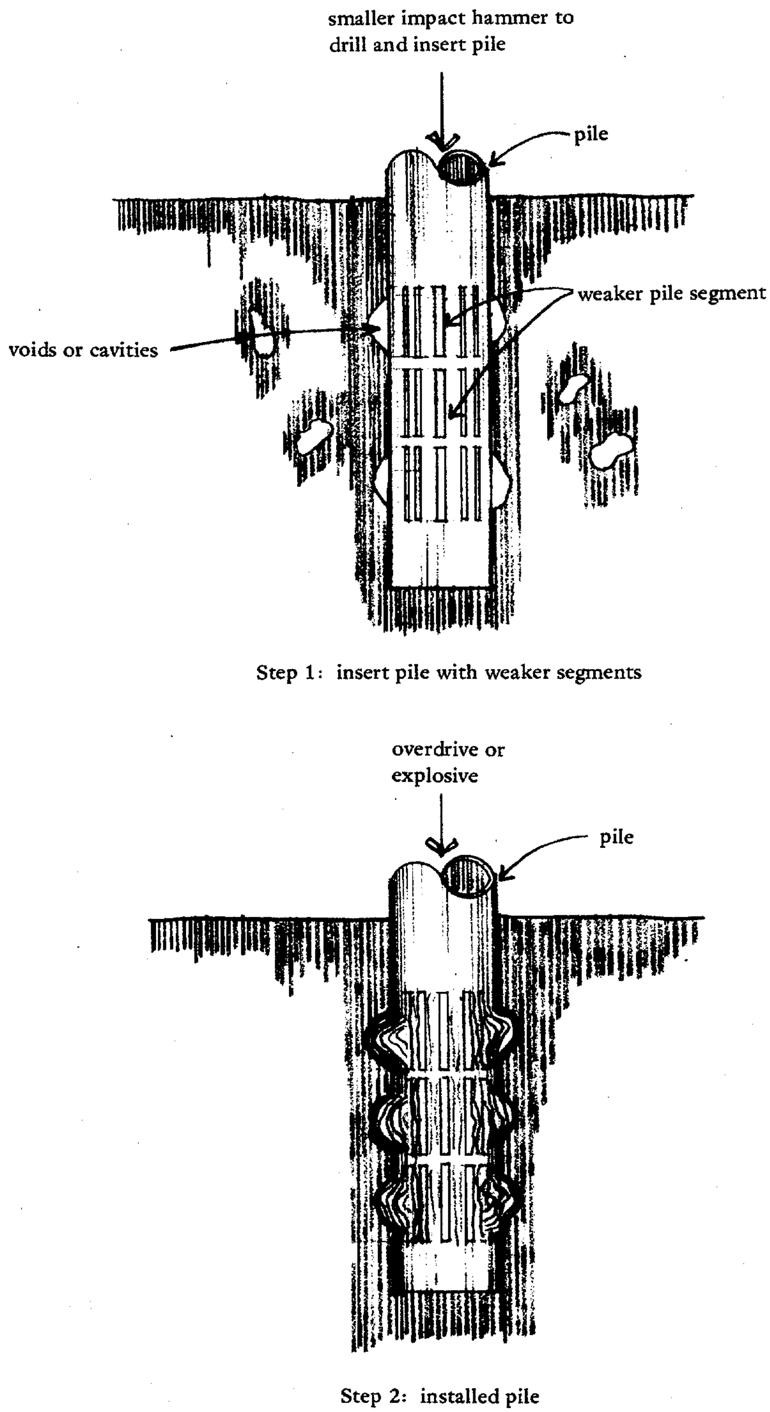
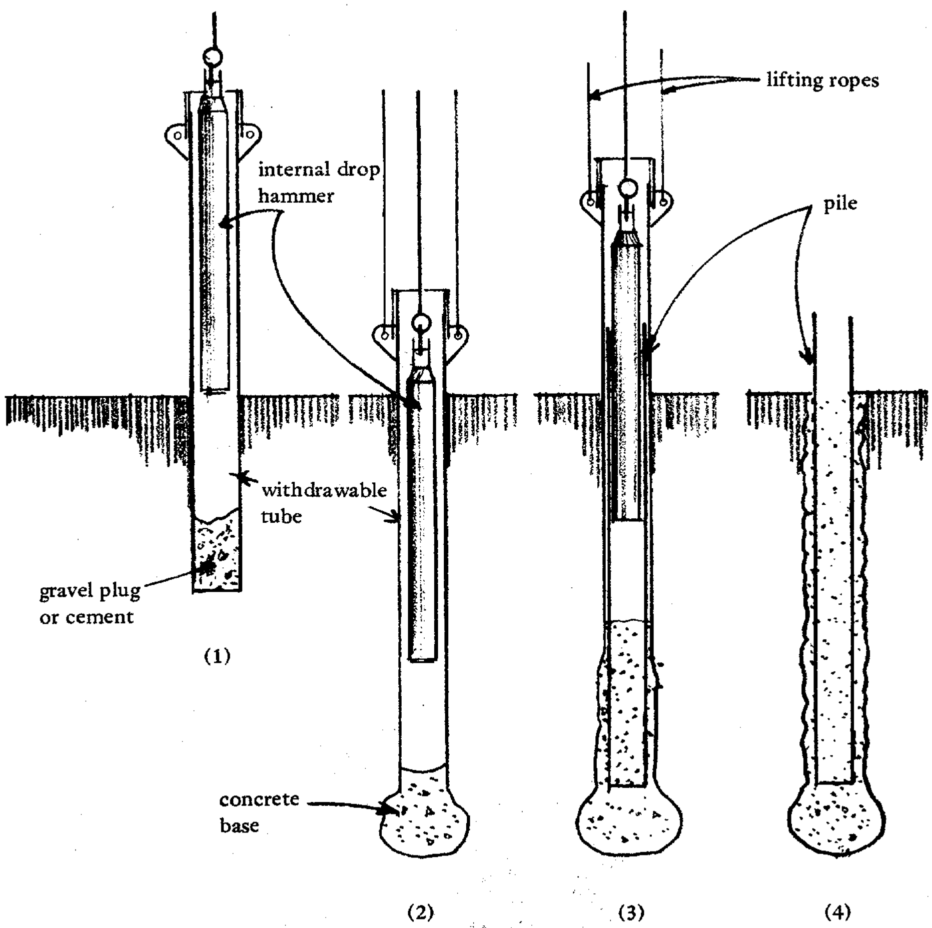


Figure 30. Conceptual drawing of a pressurized pile - Alternate C.



Step: (1) drive tube and plug
(2) compact to create bulb base
(3) place pile and backfill material while removing tube
(4) installed pile

Figure 31. Conceptual drawing of pile with enlarged tip - Alternate A.

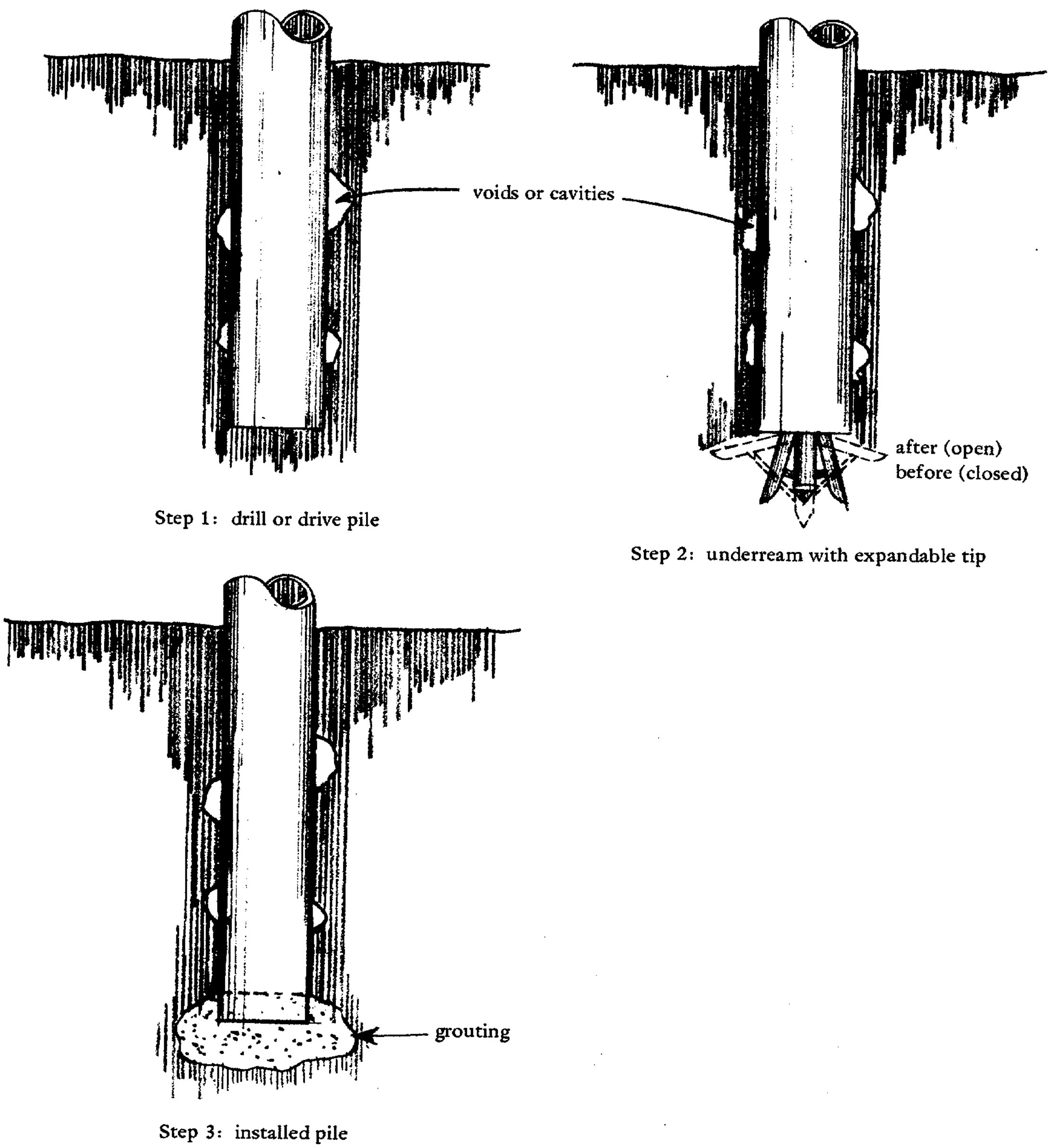


Figure 32. Conceptual drawing of pile with enlarged tip - Alternate B.

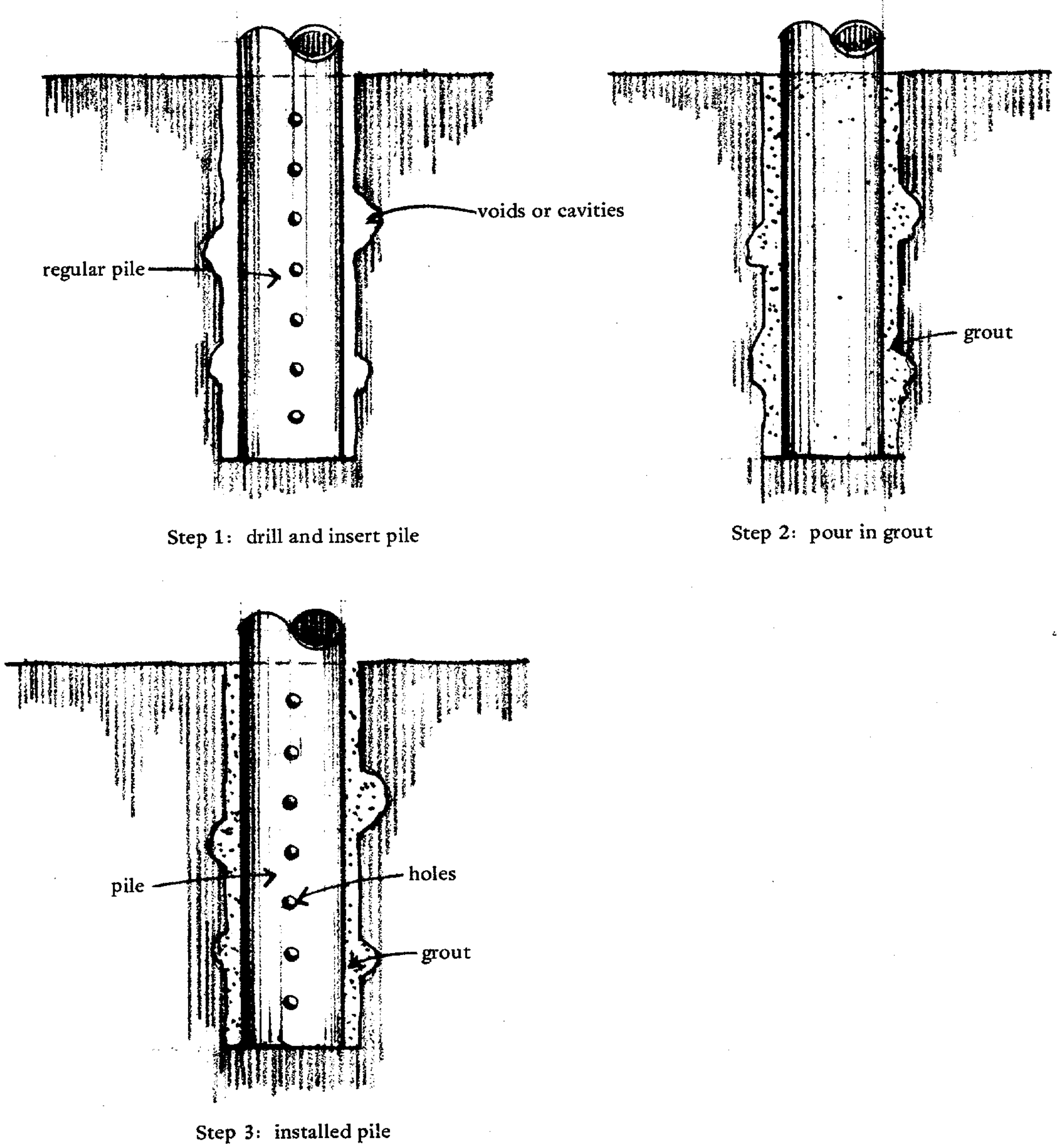


Figure 33. Conceptual drawing of modified drilled-and-grouted pile.

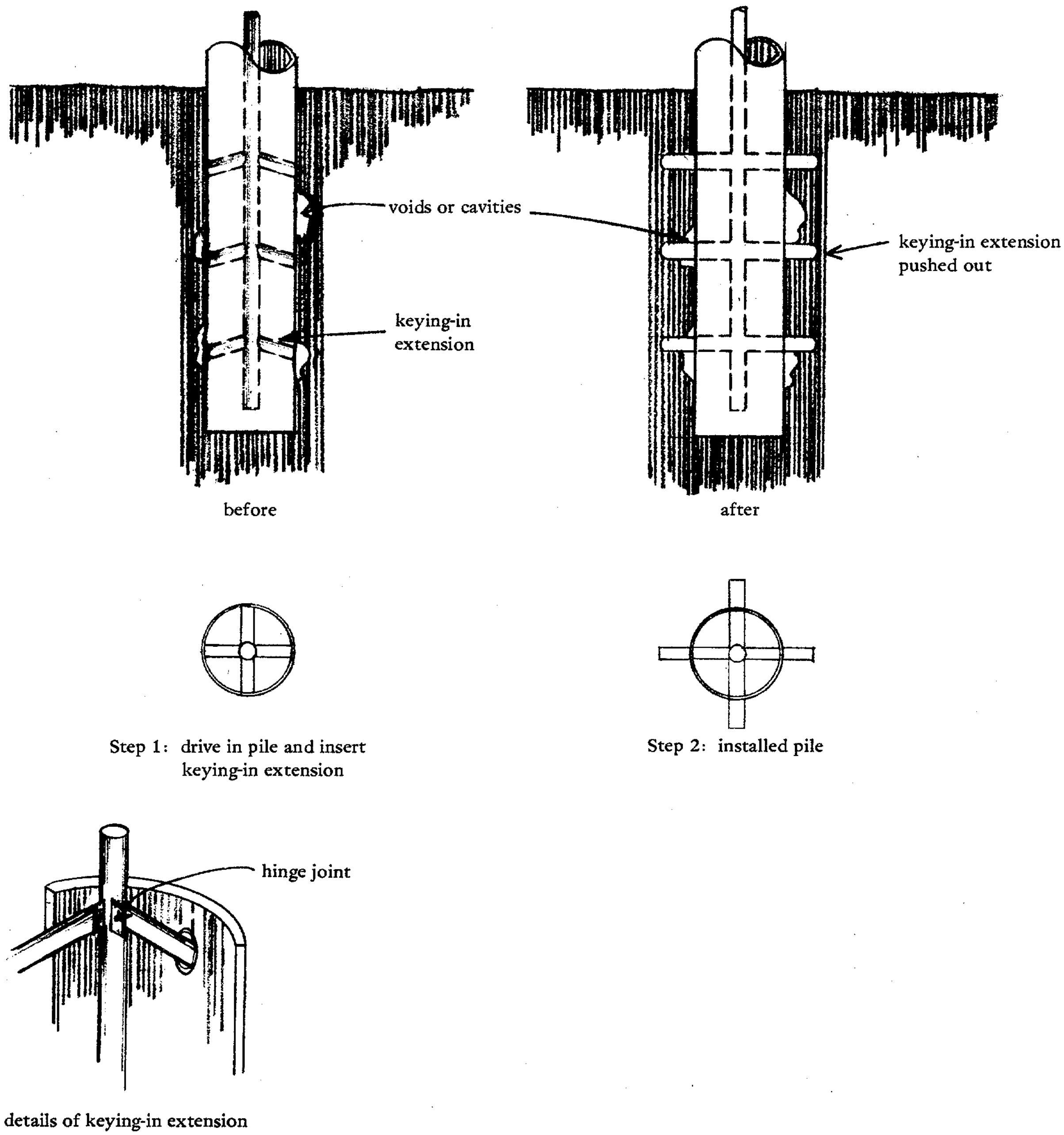


Figure 34. Conceptual drawing of keyed-in pile.

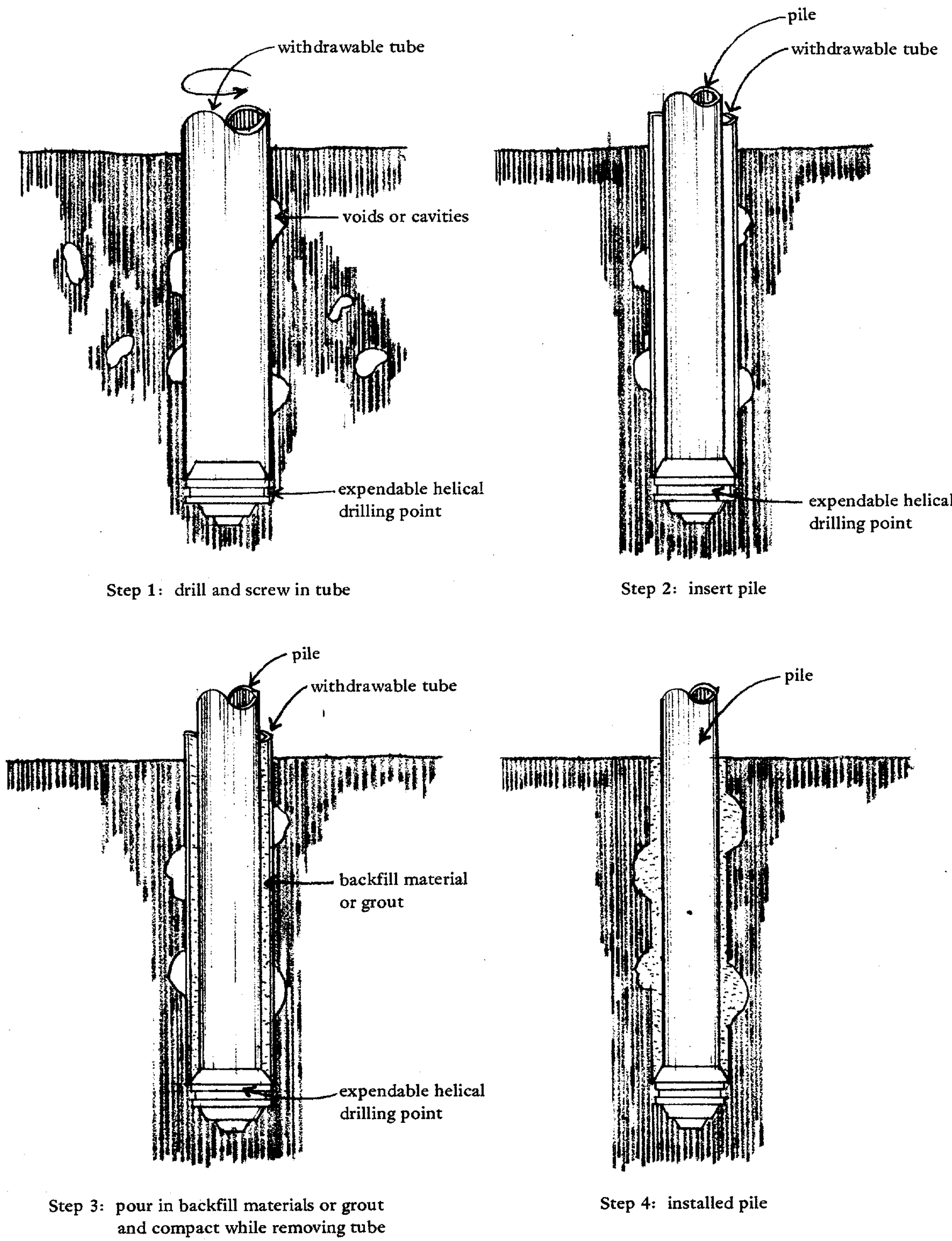


Figure 35. Conceptual drawing of drilled and screwed pile.

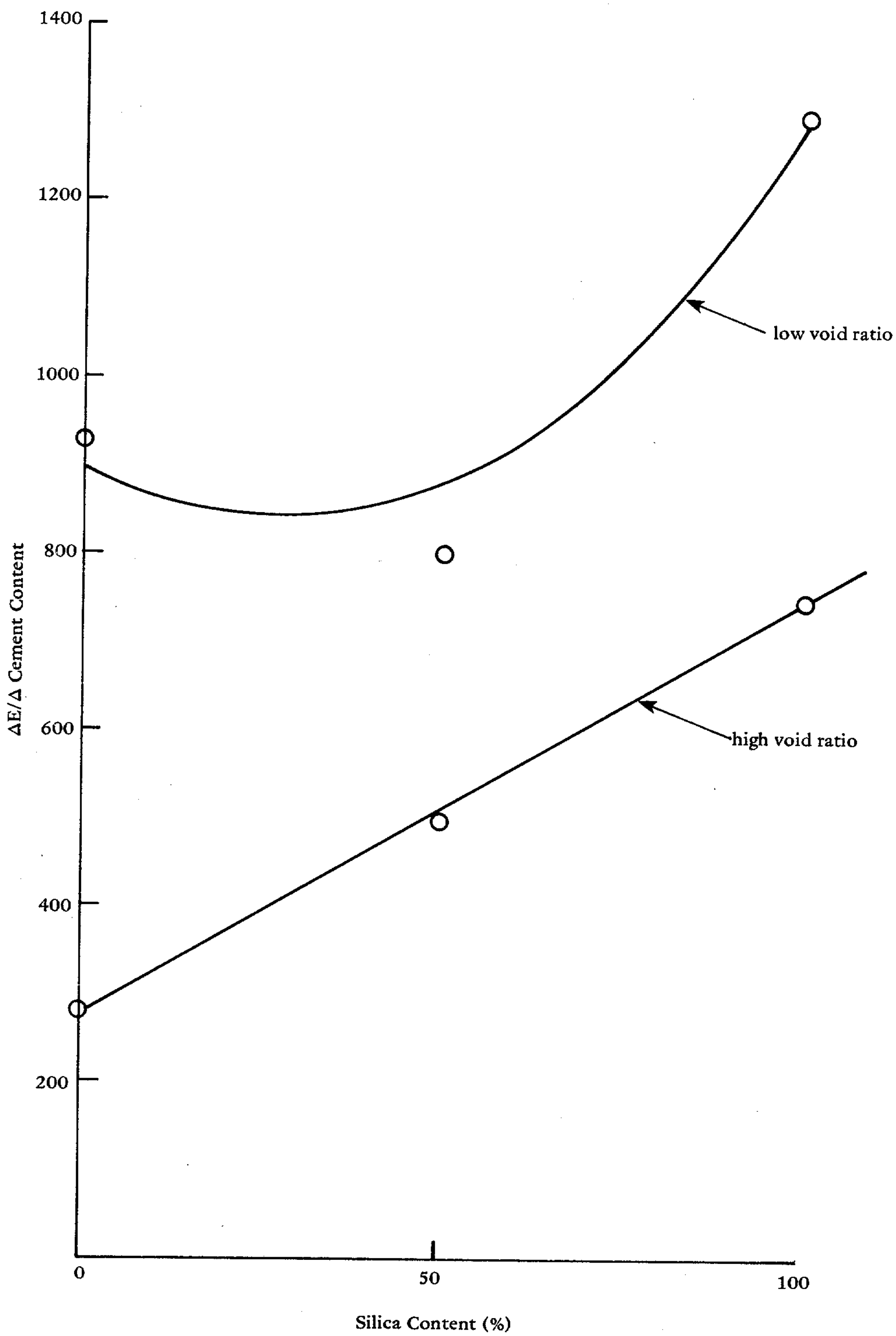
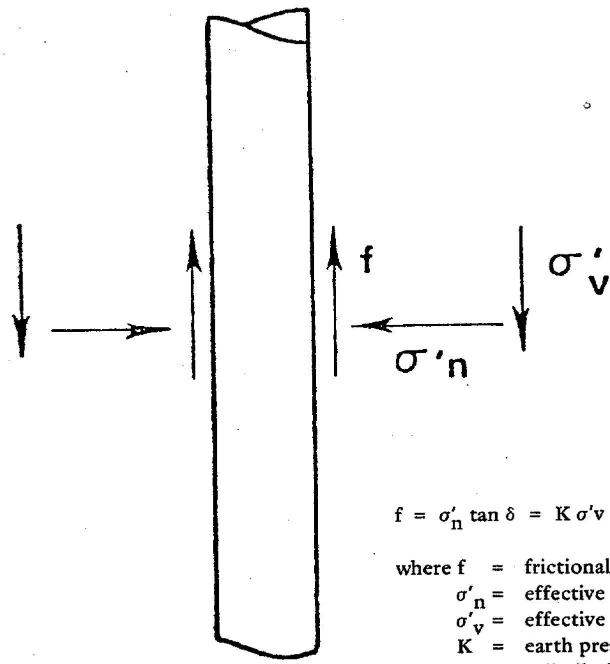


Figure 36. Comparison of high and low void ratio with change of silica content.



$f = \sigma'_n \tan\delta = K\sigma'v \tan\delta$

where f = frictional resistance
σ'_n = effective lateral stress
σ'_v = effective vertical stress
K = earth pressure coefficient
δ = soil-pile friction angle

Figure 37. Stress condition on pile surface.

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NAVCHAPGRU CO Williamsburg VA; Engineering Officer, Code 60 Williamsburg, VA; Operations Officer, Code 30 Williamsburg, VA
NAVCOASTSYSCEN CO, Panama City FL; Code 423 Panama City, FL; Code 715 (J Quirk) Panama City, FL; Code 715 (J. Mittleman) Panama City, FL; Code 772 (C B Koesy) Panama City FL; Library Panama City, FL; PWO Panama City, FL
NAVCOMMAREAMSTRSTA SCE Unit 1 Naples Italy; SCE, Wahiawa HI; Sec Offr, Wahiawa, HI
NAVCOMMSTA Code 401 Nea Makri, Greece; PWD - Maint Control Div, Diego Garcia Is.; PWO, Exmouth, Australia
NAVCONSTRACEN Code 00U15, Port Hueneme CA
NAVEDTRAPRODEVCEN Technical Library, Pensacola, FL
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NAVEODTECHCEN Code 605, Indian Head MD
NAVFAC PWO, Brawdy Wales UK; PWO, Centerville Bch, Ferndale CA; PWO, Point Sur, Big Sur CA
NAVFACENGCOM Alexandria, VA; Code 03 Alexandria, VA; Code 03T (Essoglou) Alexandria, VA; Code 04T4 (D. Potter) Alexandria, VA; Code 04T5, Alexandria, VA; Code 04A1 Alexandria, VA; Code 06, Alexandria VA; Code 09M54, Tech Lib, Alexandria, VA; Code 100, Alexandria, VA; Code 1002B (J. Leimanis) Alexandria, VA; Code 1113, Alexandria, VA
NAVFACENGCOM - CHES DIV. Code 101 Wash, DC; Code 405 Wash, DC; Code 406 Washington DC; Code 407 (D Scheesele) Washington, DC; Code FPO-1C Washington DC; FPO-1 Washington, DC; Code 11, Washington DC; Library, Washington, D.C.
NAVFACENGCOM - LANT DIV. Code 1112, Norfolk, VA; Code 405 Civil Engr BR Norfolk VA; Eur. BR Deputy Dir, Naples Italy; Library, Norfolk, VA

NAVFACENGCOM - NORTH DIV. (Boretsky) Philadelphia, PA; CO; Code 04 Philadelphia, PA; Code 04AL, Philadelphia PA; Code 09P Philadelphia PA; Code 405 Philadelphia, PA; ROICC, Contracts, Crane IN
NAVFACENGCOM - PAC DIV. (Kyi) Code 101, Pearl Harbor, HI; CODE 09P PEARL HARBOR HI; Code 2011 Pearl Harbor, HI; Code 402, RDT&E, Pearl Harbor HI; Library, Pearl Harbor, HI
NAVFACENGCOM - SOUTH DIV. Code 1112, Charleston, SC; Code 405 Charleston, SC; Code 406 Charleston, SC; Code 411 Soil Mech & Paving BR Charleston, SC; Library, Charleston, SC
NAVFACENGCOM - WEST DIV. Code 04B San Bruno, CA; Library, San Bruno, CA; O9P/20 San Bruno, CA; RDT&ELO San Bruno, CA
NAVFACENGCOM CONTRACTS AROICC, Quantico, VA; Colts Neck, NJ; Contracts, AROICC, Lemoore CA; Dir. of Constr, Tupman, CA; Eng Div dir, Southwest Pac, Manila, PI; OICC, Southwest Pac, Manila, PI; OICC-ROICC, NAS Oceana, Virginia Beach, VA; OICC/ROICC, Balboa Panama Canal; OICC/ROICC, Norfolk, VA; ROICC AF Guam; ROICC Code 495 Portsmouth VA; ROICC Key West FL; ROICC, Diego Garcia Island; ROICC, Keflavik, Iceland; ROICC, NAS, Corpus Christi, TX; ROICC, Pacific, San Bruno CA; ROICC, Point Mugu, CA; ROICC-OICC-SPA, Norfolk, VA
NAVFORCARIB Commander (N42), Puerto Rico
NAVHOSP CO, Millington, TN
NAVMAG PWD - Engr Div, Guam; SCE, Guam; SCE, Subic Bay, R.P.; Security Offr, Hawaii
NAVOCEANO Library Bay St. Louis, MS
NAVOCEANSYSCEN Code 09 (Talkington), San Diego, CA; Code 4473 Bayside Library, San Diego, CA; Code 4473B (Tech Lib) San Diego, CA; Code 5204 (J. Stachiw), San Diego, CA; Code 5214 (H. Wheeler), San Diego CA; Code 5221 (R.Jones) San Diego Ca; Code 5322 (Bachman) San Diego, CA; Hawaii Lab (R Yumori) Kailua, HI; Hi Lab Tech Lib Kailua HI
NAVORDMISTESTFAC PWD - Engr Dir, White Sands, NM
NAVORDSTA PWO, Louisville KY
NAVORDSYSUPPC Code 32, Sec Mgr, San Diego, CA
NAVPGSCOL C. Morers Monterey CA; Code 61WL (O. Wilson) Monterey CA; E. Thornton, Monterey CA; PWO Monterey CA
NAVPHIBASE CO, ACB 2 Norfolk, VA; COMNAVBEACHGRU TWO Norfolk VA
NAVFACENGCOM - LANT DIV. Code 401D, Norfolk, VA
NAVPHIBASE Dir. Amphib. Warfare Brd Staff, Norfolk, VA; Harbor Clearance Unit Two, Little Creek, VA; PWO Norfolk, VA; SCE Coronado, SD,CA; UDT 21, Little Creek, VA
NAVREGMEDCEN Code 29, Env. Health Serv, (Al Bryson) San Diego, CA; PWD - Engr Div, Camp Lejeune, NC; PWO, Camp Lejeune, NC; SCE; SCE San Diego, CA; SCE, Camp Pendleton CA; SCE, Guam
NAVSCOLCECOFF C35 Port Hueneme, CA; CO, Code C44A Port Hueneme, CA
NAVSCSOL PWO, Athens GA
NAVSEASYSCOM Code 0325, Program Mgr, Washington, DC; Code PMS 395 A 3, Washington, DC; Code PMS 396.3311 (Rekas), Wash., DC; Code SEA OOC Washington, DC
NAVSECGRUACT PWO Winter Harbor ME; PWO, Adak AK; PWO, Torri Sta, Okinawa
NAVSECGRUCOM Code G43, Washington DC
NAVSHIPREPFAC Library, Guam; SCE Subic Bay; SCE, Yokosuka Japan
NAVSHIPYD Bremerton, WA (Carr Inlet Acoustic Range); Code 202.4, Long Beach CA; Code 202.5 (Library) Puget Sound, Bremerton WA; Code 280, Mare Is., Vallejo, CA; Code 280.28 (Goodwin), Vallejo, CA; Code 400, Puget Sound; Code 440 Portsmouth NH; Code 440, Norfolk; Code 440, Puget Sound, Bremerton WA; Code 457 (Maint. Supr.) Mare Island, Vallejo CA; L.D. Vivian; Library, Portsmouth NH; PWD (Code 420) Dir Portsmouth, VA; PWD (Code 460) Portsmouth, VA; PWO Charleston Naval Shipyard, Charleston SC; PWO, Bremerton, WA; PWO, Mare Is.; Tech Library, Vallejo, CA
NAVSTA CO Roosevelt Roads P.R. Puerto Rico; Code 16P, Keflavik, Iceland; Dir Engr Div, PWD, Mayport FL; Engr. Dir., Rota Spain; Long Beach, CA; PWD (LTJG.P.M. Motolenich), Puerto Rico; PWD - Engr Dept, Adak, AK; PWD - Engr Div, Midway Is.; PWO, Keflavik Iceland; PWO, Mayport FL; SCE, Guam, Marianas; SCE, Pearl Harbor HI; SCE, San Diego CA; SCE, Subic Bay, R.P.; Security Offr, San Francisco, CA
NAVSUPPACT Engr. Div. (F. Mollica), Naples Italy; PWO Naples Italy
NAVSUPPFAC PWD - Maint. Control Div, Thurmont, MD
NAVSUPPO Security Offr, Sardinia
NAVSURFWPNCEN PWO, Dahlgren VA
NAVTECHTRACEN SCE, Pensacola FL
NAVWARCOL Dir. of Facil., Newport RI
NAVWPNCEN Cmdr, China Lake, CA; Code 2636 China Lake; PWO (Code 266) China Lake, CA; ROICC (Code 702), China Lake CA
NAVWPNSTA Code 092, Colts Neck NJ; Code 092, Concord CA; Engrng Div, PWD Yorktown, VA
NAVWPNSTA PW Office Yorktown, VA
NAVWPNSTA PWD - Maint. Control Div., Concord, CA; PWD - Supr Gen Engr, Seal Beach, CA; PWO Colts Neck, NJ; PWO, Charleston, SC; PWO, Seal Beach CA
NAVWPNSUPPCEN Code 09 Crane IN
NCTC Const. Elec. School, Port Hueneme, CA

NCBC Code 10 Davisville, RI; Code 15, Port Hueneme CA; Code 155, Port Hueneme CA; Code 1552 (Brazele) Port Hueneme, CA; Code 155B (Nishimura) Port Hueneme, CA; Code 156, Port Hueneme, CA; Code 156F (Volpe) Port Hueneme, CA; Code 430 (PW Engrng) Gulfport, MS; Library, Davisville, RI; PWO (Code 80) Port Hueneme, CA; PWO, Gulfport, MS; Technical Library, Gulfport, MS
NCBU 411 OIC, Norfolk VA
NCR 20, Commander; 30, Guam, Commander; 30th Det, OIC, Diego Garcia I
NMCB 3, SWC D. Wellington; 74, CO; FIVE, Operations Dept; Forty, CO; THREE, Operations Off.
NOAA (Mr. Joseph Vadus) Rockville, MD; Library Rockville, MD
NORDA Code 410 Bay St. Louis, MS; Code 440 (Ocean Rsch Off) Bay St. Louis MS
NRL Code 5800 Washington, DC; Code 5843 (F. Rosenthal) Washington, DC; Code 8441 (R.A. Skop), Washington DC
NSC Code 54.1 Norfolk, VA; SCE Norfolk, VA; SCE, Charleston, SC
NSD SCE, Subic Bay, R.P.
NTC OICC, CBU-401, Great Lakes IL; SCE, San Diego CA
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PACMISRANFAC HI Area Bkg Sands, PWO Kekaha, Kauai, HI
PERRY OCEAN ENG R. Pellen, Riviera Beach, FL
PHIBCB 1 P&E, San Diego, CA; 1, CO San Diego, CA
PMTC Code 3331 (S. Opatowsky) Point Mugu, CA; Code 4253-3, Point Mugu, CA; EOD Mobile Unit, Point Mugu, CA
PWC ACE Office Norfolk, VA; CO, (Code 10), Oakland, CA; Code 10, Great Lakes, IL; Code 105 Oakland, CA; Code 105, Oakland, CA; Code 121.1, Oakland, CA; Code 128, Guam; Code 154 (Library), Great Lakes, IL; Code 200, Great Lakes IL; Code 400, Great Lakes, IL; Code 400, Pearl Harbor, HI; Code 400, San Diego, CA; Code 420, Great Lakes, IL; Code 420, Oakland, CA; Code 424, Norfolk, VA; Code 500 Norfolk, VA; Code 500, Great Lakes, IL; Code 500, Oakland, CA; Code 700, San Diego, CA; Code 800, San Diego, CA; Library, Code 120C, San Diego, CA; Library, Guam; Library, Norfolk, VA; Library, Pearl Harbor, HI; Library, Pensacola, FL; Library, Subic Bay, R.P.; Library, Yokosuka JA; Maint. Control Dept. Oakland CA; Production Officer, Norfolk, VA
SPCC PWO (Code 120) Mechanicsburg PA
SUPANX PWO, Williamsburg VA
TVA Solar Group, Arnold, Knoxville, TN
UCT ONE OIC, Norfolk, VA
UCT TWO OIC, Port Hueneme CA
U.S. MERCHANT MARINE ACADEMY Kings Point, NY (Reprint Custodian)
US DEPT OF INTERIOR Bur of Land Mgmnt Code 583, Washington DC
US GEOLOGICAL SURVEY Off. Marine Geology, Piteleki, Reston VA
US NAVAL FORCES Korea (ENJ-P&O)
USCG G-EOE-4 (T Dowd), Washington, DC; Library Hqs Washington, DC
USCG R&D CENTER CO Groton, CT; D. Motherway, Groton CT; Library New London, CT
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