NBS BUILDING SCIENCE SERIES 142

Load-Displacement Characteristics of Shallow Soil Anchors

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Felix Y. Yokel
Riley M. Chung
Frank A. Rankin
Charles W. C. Yancey

Center for Building Technology
National Engineering Laboratory
National Bureau of Standards
Washington, DC 20234

Prepared for the
Office of Policy Development and Research
U.S. Department of Housing and Urban Development
Washington, DC 20410

U.S. DEPARTMENT OF COMMERCE, Malcolm Baldrige, Secretary
NATIONAL BUREAU OF STANDARDS, Ernest Ambler, Director

Issued May 1982

Library of Congress Catalog Card Number: 82-600509

National Bureau of Standards Building Science Series 142
Nat. Bur. Stand. (U.S.), Bldg. Sci. Ser. 142, 163 pages (May 1982)
CODEN: BSSNBV

U.S. GOVERNMENT PRINTING OFFICE
WASHINGTON: 1981

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402
Price $6.50
(Add 25 percent for other than U.S. mailing)

ABSTRACT

Tests on shallow soil anchors, commonly used by the mobile home industry, including 6-in single helix and 4-in double helix anchors as well as three types of swivel anchors, were conducted on three sites: a silty site, a sandy site, and a clay site. Test variables included direction of anchor installation; direction of loading; anchor depth; size of anchor plate; and cyclic load effects. The effect of these test variables on load-displacement characteristics, measured at the anchor head, is investigated. It is concluded that on most sites the anchor types tested, when installed in accordance with present industry practice for mobile home tiedown systems, did not deliver the anchor performance required in present standards. It is recommended that minimum load capacity requirements for anchors be waived; that all anchors be preloaded to 1.25 times the design load; and that one anchor per mobile home, or three anchors per site if soil conditions are uniform, be preloaded to 1.5 times the design load.

Keywords: anchors; cyclic loading; field testing; flood forces; foundations; load capacity, mobile homes; soil anchors; soil mechanics; stiffness; wind forces.

COVER: *Mobile home damaged by wind. Anchored mobile homes in the vicinity suffered only minor damage.*

PREFACE

This report is part of a study which was sponsored by the Office of Policy Development and Research of the U.S. Department of Housing and Urban Development. The overall objectives of this study were: to determine wind and flood forces acting on mobile homes; to study the performance characteristics of soil anchors; and to develop performance criteria for mobile home foundations with particular emphasis on the tiedown system. In previous stages of this work, measurements were made of the wind forces acting on mobile homes, the state-of-the-art in anchoring technology was studied, and the forces acting on tiedown systems were determined. The work was published in references [13],[1] [12], and [20], respectively. This report deals with the results of experimental and analytical studies of the load capacity of soil anchors used to tie down mobile homes and with methods to insure adequate performance of soil anchors. Initial results of this work were presented in reference [19], pp. 3-20.

This study was performed by the Geotechnical Engineering Group of the Center for Building Technology.

[1] Numbers in brackets refer to the literature references in section 7.

EXECUTIVE SUMMARY

Two hundred and thirty-two anchor tests were conducted on three sites: a silty site; a sandy site; and a clay site. Anchors tested included 6-in single helix, 4-in double helix, and 3-in single helix anchors, and 6-in triangular, 10 1/4 length x 1 3/4-in o.d. pipe, and 6 1/2 length x 1 1/4-in o.d. pipe swivel ("fluke") anchors. Loading conditions included coaxial and noncoaxial (inclined) pull on vertical and inclined anchors installed to their full depth, and coaxial pull on anchors installed at various depths ranging from 1 ft to 4 ft. Modes of loading included monotonic tests, monotonic tests with several intermediate cycles of unloading and reloading, and cyclic tests. The tests were carried to complete withdrawal and graphs of load vs. anchor-head displacement were electronically recorded. Several anchor tests were carried out under submerged conditions. The tests were correlated with determinations of soil conditions by in-situ and laboratory tests. In-situ tests included soil test probe readings, standard penetration tests, and measurement of the anchor installation torque. It was concluded that:

1. The anchors tested did not deliver the anchor performance required by ANSI Standard A119.3 [2].

2. The virgin load-displacement characteristics of anchors are a unique function of installation depth, loading, and soil conditions and are not substantially altered by intermediate unloading and reloading cycles unless a great number of cycles of load close to the load capacity are applied.

3. The initial resistance to displacement of preloaded anchors in all loading modes is much higher than that in the first loading cycle and far exceeds the performance required by ANSI Standard A119.3.

4. Coaxially loaded inclined anchors have smaller load capacities than coaxially loaded vertical anchors but their initial resistance to displacement is similar to that of coaxially loaded vertical anchors.

5. Vertical anchors subjected to inclined loads have higher load capacities than coaxially loaded vertical anchors, but their initial resistance to displacement (if they are not preloaded) is much less than that of coaxially loaded vertical anchors.

6. Swivel ("fluke") anchors can deliver satisfactory performance if properly seated and adequately preloaded.

7. Helix anchors lose their protective paint coat during installation and thus have no corrosion protection.

8. Helix anchors experience bending of the helix in all loading modes and bending of the shaft in noncoaxial loading.

9. Helix anchor hardware tended to have adequate load capacity but was vulnerable in the weld between the shaft and the helix, particularly when subjected to noncoaxial cyclic load.

It is recommended to:

1. Eliminate the minimum load capacity requirements for anchors in present standards and stipulate instead required anchor resistance per mobile home, so that the number of anchors used can be determined in accordance with site conditions.

2. Require that every anchor be preloaded in the direction of the anticipated service load to 1.25 times the working load during installation, and that one anchor per mobile home, or three anchors per site where soil conditions are uniform, be preloaded to 1.5 times the required working load; and that the required working load be the load calculated for the design wind pressure without the increase for foundations presently required in the Federal standard [9].

3. Require that anchors be adequately protected against corrosion by galvanizing or other means; that the corrosion protection be effective for the service life of the mobile home, and remain effective if anchor deformation anticipated under the preload or the service load occurs.

It is noted that if anchors are to be included as part of a permanent mobile home foundation, they should be durable enough to retain their structural integrity throughout the service life of the mobile home; they should be preloaded; and consideration should be given to potential effects of frost heave.

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LIST OF SYMBOLS

A	Projected surface area of anchor plate, ft^2
AH-6	6-in triangular (arrowhead) anchor
B	Width (or diameter) of anchor plate, ft
c	Cohesion, lb/ft^2
D	Average depth of anchor plate below ground surface, ft
D_v	Average depth of anchor plate below ground surface for full-depth vertical anchor, ft
D-4	4-in double helix anchor
H-3	3-in single helix anchor
H-6	6-in single helix anchor
n	Number of specimens listed or number of load cycles
N	Blow count in Standard Penetration Test, blows/ft
N_u	Uplift capacity factor for cohesive soils
N_{qu}	Uplift capacity factor for granular soils
P_c	Cyclic load, lb
P_{4h}	Load at 4-in horizontal displacement, lb
P_p	Preload, lb
P_{2v}	Load at 2-in vertical displacement, lb
P_w	Working load, lb
P-6	6 1/2 x 1 1/4 in pipe anchor
P-10	10 x 1 3/4 in pipe anchor
q	Anchor load capacity per unit anchor plate area, lb/in^2
Q_u	Ultimate load capacity of anchor, lb

Q_u'	Ultimate load capacity determined by pullout after completion of cyclic test, lb
Q_v	Ultimate load capacity of full-depth vertical anchor, lb
R_{10}	Reloading modulus of anchor after 10 load cycles, lb/in
R_{85}	Reloading modulus of anchor at 85 percent of Q_u, lb/in
STP	Soil Test Probe
SPT	Standard Penetration Test
s	Shear strength of soil lb/ft^2
$\bar{s}$	Average shear strength of soil, lb/ft^2
S	Anchor shaft resistance, lb
t	Torque reading in STP test, in-lb
$\bar{t}$	Torque reading in STP test, averaged over anchor depth, in-lb
T	Anchor installation torque, ft-lb
v	Coefficient of variation of a sample. (The coefficient of variation of the population will differ considerably if the sample is small.)
α_1	Angle of applied load with horizontal, degrees
α_2	Angle of anchor shaft with horizontal, measured in same direction and the opposite quadrant of α_1, degrees
γ	In place unit weight of soil, lb/ft^3
Δ_u	Anchor head displacement in direction of pull at ultimate load, in

SI CONVERSION UNITS

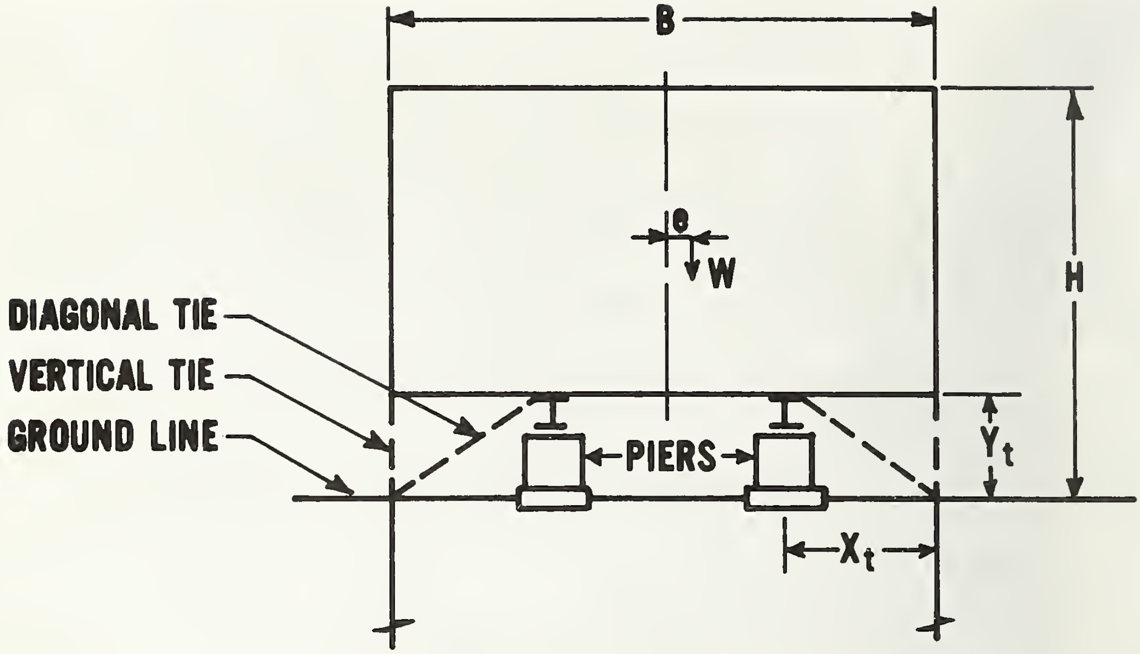
In view of present accepted practice in the U.S. mobile home industry, common U.S. units of measurement are used throughout this report. The table below is presented to facilitate conversion to SI Units.

To Convert From	To	Multiply by
ft	m	0.305
in	mm	25.4
ft^2	m^2	9.29×10^{-2}
in^2	mm^2	645.16
lb (force)	N	4.45
lb/ft^2 (psf)	Pa	47.88
lb/in^2 (psi)	kPa	6.89
lb/ft^3	Kg/m^3	16.02

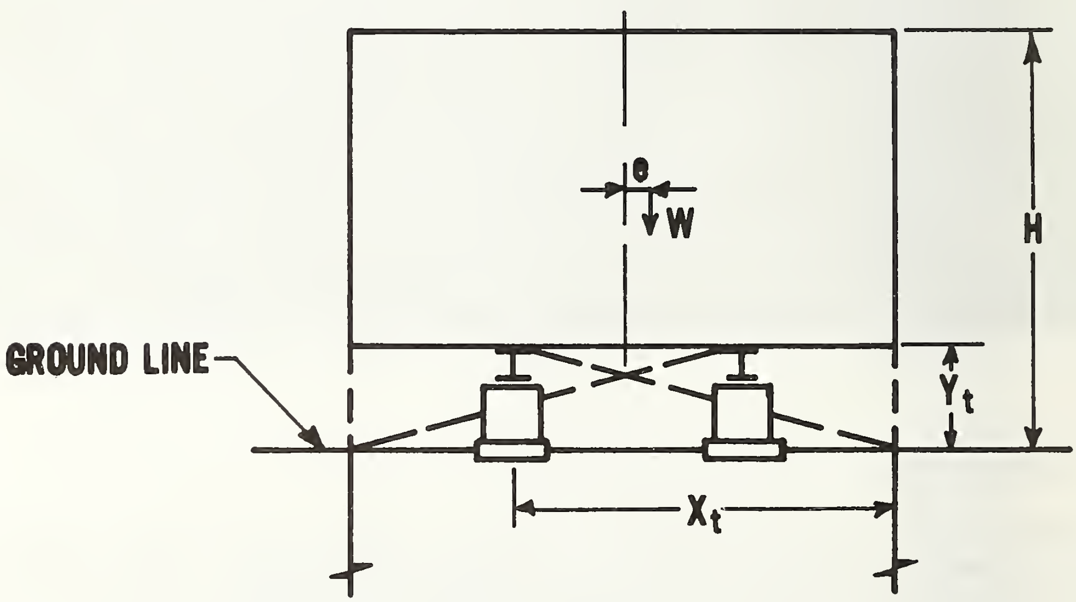
FACING PAGE: *Mobile home anchors pulled out by flood. The bent anchor shafts indicate that large horizontal force components acted on the anchors.*

1. INTRODUCTION

A common foundation type presently used to support mobile homes consists of pairs of piers, about 8 to 10 feet on center, which support the chassis beams of the mobile home unit. In addition, the mobile home is attached to soil anchors by transverse over-the-roof ties and transverse diagonal ties attached to the chassis beam (see figure 1.1). The loads acting on this type of foundation have been studied by Yokel et al. [20]. The horizontal component of the wind load is resisted by the diagonal ties and by whatever horizontal-load resistance is provided by the piers. Since piers are not normally designed



(a) - Near Tie Connection



(b) - Far Tie Connection

Figure 1.1 Typical mobile home tiedown systems to resist horizontal loads, the diagonal ties must provide the necessary horizontal-load resistance. The vertical over-the-roof ties are provided to resist uplift and overturning. It has been shown [20] that if the diagonal ties are attached to the chassis beams adjacent to the anchor [(figure 1.1(a)], the vertical ties are not essential. However, the vertical over-the-roof ties help hold the mobile home together. The vertical ties are also engaged by uplift forces resulting from flooding and they must be used to resist windloads if the diagonal ties are attached as shown in figure 1.1(b). Thus, soil anchors must provide effective resistance to horizontal as well as vertical forces.

Present anchor technology was studied by Kovacs and Yokel [12]. The anchors most frequently used by the mobile home industry are single helix, 6-in diameter anchors installed to a 4-ft maximum depth and double helix 4-in diameter anchors installed to a 2 ft - 9 in maximum depth (figure 1.2). Other types of anchors are also available, but not extensively used at the present time. Miscellaneous hypotheses have been developed which correlate the load capacity of anchors to the shear strength of soils, which in turn can be measured by various in-situ and laboratory tests. However, it was concluded on the basis of available data that the correlation between calculated and measured anchor-load capacities, particularly in granular soils, tends to be poor [12]. In part, our inability to make reliable predictions of anchor-load capacity on the basis of the shear strength of soils is attributable to our inability to make reliable measurements of the in-situ shear strength, particularly that of granular soils. This measurement problem is even more severe at shallow depths, where soils are subjected to many disturbances, such as freezing and thawing, changing moisture content and the effect of root systems and organic matter. Moreover, the most commonly used in-situ test, the Standard Penetration Test (ASTM D 1586) [4] is difficult to interpret at a shallow depth because of the short drill-stem length used [15].

In present practice, the load capacity of mobile home anchors is estimated on the basis of in-situ soil test probe (STP) measurements, coupled with predictions based on the results of pull-out tests conducted in soils with characteristics similar to that of the site. Guidance for this procedure is provided in ANSI Standard A119.3 [2] and in miscellaneous charts published by industry [7]. ANSI Standard A119.3 stipulates in section 4.5.1 that a ground anchor, when installed, shall be capable of resisting an allowable working load at least equal to 3,150 lb in the direction of the tie, plus a 50 percent overload (4,725 lb total) without failure. Failure is defined as an anchor movement of 2 inches at 4,725 lb in the direction of the vertical tie. Anchors which are designed for loads other than "direct withdrawal" (coaxial loads) shall resist an applied design load of 3,150 lb at 45° from the horizontal without displacing the anchor more than 4 inches horizontally at the point where the tie is attached to the anchor. Anchors designed for connection of multiple ties shall be designed to resist the combined working load and overload consistent with the intent expressed in section 4.5.1. The magnitude of the stipulated load capacity in ANSI A119.3 is entirely predicated on the load capacity of presently used steel straps with no regard to whether existing soil anchor technology can provide the stipulated load resistance.

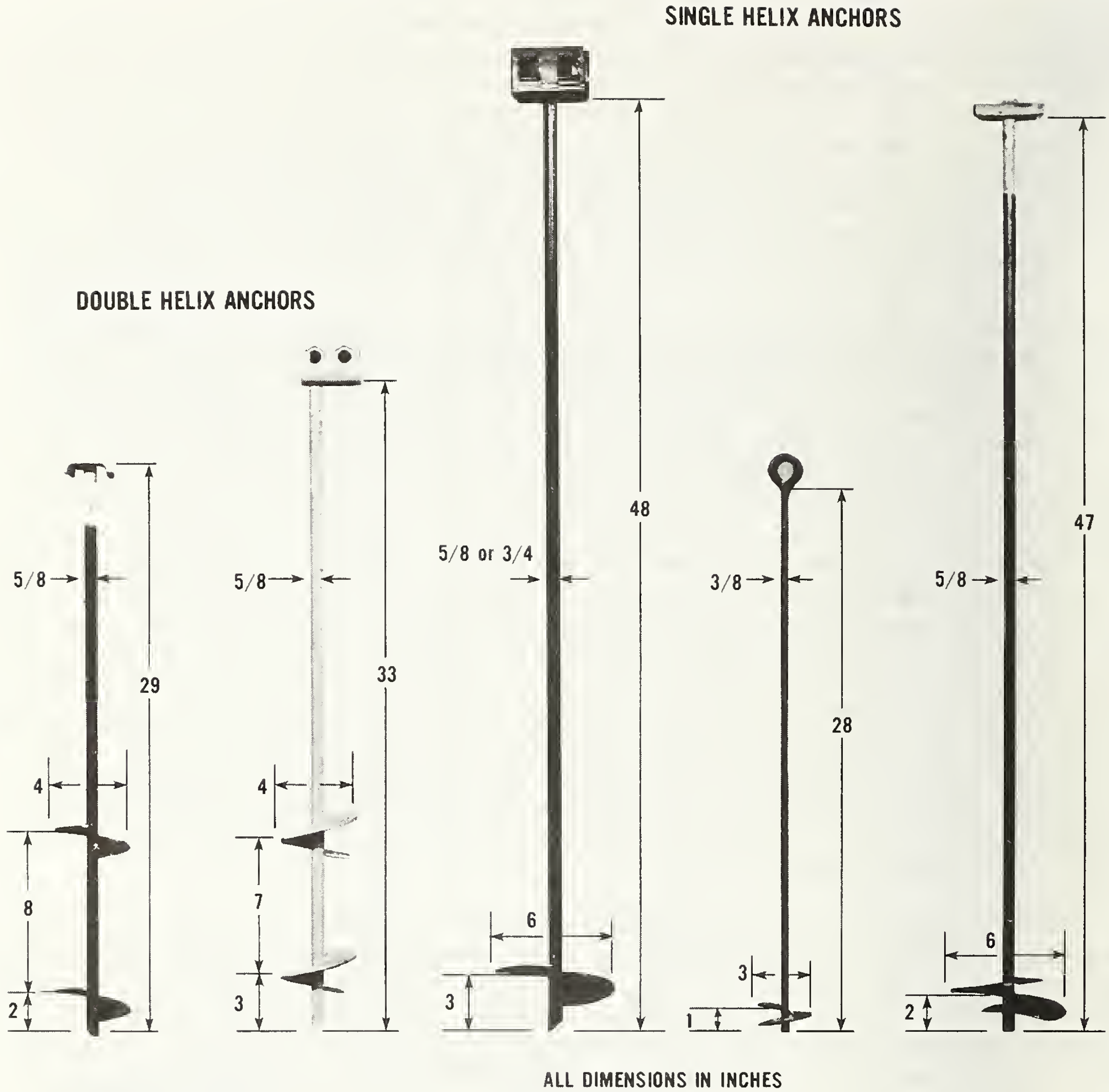


Figure 1.2 Single and double helix anchors

There is evidence that in present practice withdrawal tests are conducted in accordance with the first part of this provision, namely coaxially, and that the anchor capacity is then determined for a 2 in withdrawal. Most available data are limited to this test condition, and therefore do not provide much information on anchor capacity under larger displacements or under inclined loads which have a component normal to the axis of the anchor.

Provisions for mobile home anchors are generally enforced by the States. These provisions are not uniform throughout the United States; some States have no provisions, while others, like Texas [17] require that the anchor resist a load "in the direction of the expected applied loads" of 4,725 lb for single headed anchors and 6610 lb for double headed anchors. In the Texas provision, "failure" is defined as a movement of 3 inches in the direction of the axis of the anchor. This relaxes the more stringent limitation of 4 inches on horizontal movement and 2 inches on axial movement provided in ANSI 119.3. Proof of compliance with the various State provisions can generally be provided by anchor manufacturers or installers by documenting the results of withdrawal tests conducted in soils with characteristics similar to those of the site. Some States will accept the results of tests in artificially prepared soils. Implicit in the use of withdrawal tests as proof of compliance in similar soil conditions are two assumptions:

1. That soil conditions can be characterized well enough so that anchor capacity on a given site can be predicted from test results on similar sites.

2. That anchors will have satisfactory load-displacement characteristics in the horizontal, as well as the vertical direction.

The objective of the test program presented herein was to study the performance characteristics of the most common types of mobile home anchors and to determine how adequate performance can be assured.

FACING PAGE: *X-Y recorder used to plot anchor test results in the field.*

2. SCOPE

Two hundred and thirty-two anchor tests were conducted on the three sites: a sandy site; a silty site; and a clay site. Two hundred and nineteen of these tests were conducted with single and double helix anchors and 13 tests were conducted with self-seating swivel anchors (triangular and pipe). Of these tests, 179 were pullout tests using vertical and inclined axial pulls and inclined pulls at an angle to the anchor shaft; 53 were cyclic tests using several hundred equal loading cycles of vertical coaxial pull or inclined pull at an angle to the anchor shaft.

The soil characteristics of the test sites were determined by two types of in-situ tests: the Standard Penetration Test (ASTM D 1586); and the Soil Test Probe (refer to ANSI A119.3). In addition, disturbed and undisturbed soil samples were analyzed and tested in the laboratory.

Test results were recorded electronically by an x - y plotter as a plot of applied load vs. displacement of the point where the tie is attached to the anchor. Most static tests were carried to complete anchor withdrawal with several intermediate cycles of unloading and reloading in order to provide information of the load-displacement characteristics to the point of incipient loss of load capacity. Cyclic tests were conducted at various load levels and in most instances carried to a point where the probable trend of response to additional load cycles is apparent. Throughout the test program emphasis was placed on the effect of pre-loading on anchor response. Several tests were conducted under submerged conditions in order to study the effect of flooding on anchor capacity.

In this report all the test results are presented in tabular form. In the analysis of the test results, anchor load capacity is correlated with soil strength as measured by in-situ and laboratory tests and an assessment is made of our ability to predict anchor-load capacity. Anchor performance characteristics in various soil types are compared and studied and methods of insuring adequate anchor performance are recommended.

FACING PAGE: *Cyclic load test on Site B. The test rig is in the foreground. Behind the test rig is the oil pump used for cyclic loading. The van in the background housed a generator and data acquisition equipment. Note the 20-ft high banks surrounding the borrow pit in which Site B was located. The original ground surface can be seen on the top of the bank.*

3. TEST SETUP AND PROGRAM

3.1 DESCRIPTION OF TEST SITES

3.1.1 General

There was prior evidence that performance characteristics of soil anchors depend on the type of soil in which the anchors are embedded [12]. Thus, it was important to test anchors on sites with a variety of soil characteristics. It was therefore decided to choose three different sites: a silty site (A), a sandy site (B), and a clay site (C). The three sites selected are listed in table 3.1. Plans showing anchor test and soil-boring locations, as well as test reports containing boring logs and soil test data are included in appendix A. Pertinent site characteristics are described hereafter.

Table 3.1 Test Sites Selected

Site	Predominant Soil Type	Location
A	Silt	NBS grounds, Gaithersburg, MD
B	Sand	Odenton, MD
C	Clay	Upper Marlboro, MD

3.1.2 Test Site A, Silty Soils

Test Site A was explored by eleven 5-ft deep test borings. The borings generally indicate 2 ft of fill consisting of local silty material overlying residual silty soil and quartz-rich schist of the Wissahickon Formation. No groundwater accumulated in the borings during, and up to 5 hours after, the drilling. On the basis of visual observation and laboratory analysis, the soil can be described as brown stiff clayey silt with some fine sand and quartz fragments, and classified as Group ML in accordance with the Unified Soil Classification System (ASTM D 2487 [5]).

Laboratory tests indicate that the soils consisted of 62 to 83 percent by weight of materials passing No. 200 sieve (particle size smaller than 0.074 mm). The liquid limit and plasticity index were 39 and 12 percent, respectively. The SPT "N" values ranged from 7 to 19 blows per foot. Laboratory strength tests determined an unconfined compressive strength of 4000 psf at 4 percent strain. Direct shear tests provide some additional information. Measured natural wet density was 119 lb/ft^3 and dry density was 98 lb/ft^3.

3.1.3 Test Site B, Sandy Soils

Test Site B is located at the bottom of an excavated borrow pit and was explored by four 10 ft deep test borings. No water accumulated in the borings to the depth where the holes caved, which ranged from 5 to 6.5 ft. The deposits on this site can be traced to the Potomac Group which generally consists of interbedded sands and silty clay layers of cretaceous origin which characteristically are overconsolidated. The sands in this formation tend to be medium dense to dense and the silty clays stiff to hard. The borings indicate a 6 to 9 ft thick layer of moist fine-to-course clean sand which rests on sandy silty clay. On part of the test site the above-mentioned sand layer is covered by a 0.5 to 1.0 ft thick crust of dense silty sand fill with gravel. Most of the anchor tests were performed in locations that were not covered by the dense crust. The 6 to 9 ft sand layer can be described as light brown medium dense fine-to-coarse sand with a trace of silt and gravel and classified as Group SP in accordance with ASTM D 2487.

Laboratory tests indicate that the sand deposit had 95 percent by weight of material in the range between No. 4 and No. 200 sieves (particle sizes between 4.699 and 0.075 mm), a natural moisture content of 2 to 4 percent and a natural dry density of 92 to 95 lb/ft^3. The SPT "N" values over a 5 ft depth below ground ranged from 10 to 21 blows per ft. Consolidated undrained triaxial compressive strength tests on reconstituted samples yielded an angle of shearing resistance $\phi = 31°$. The ϕ value obtained from direct shear tests was approximately 29°.

The dense crust had natural dry densities from 105 to 108 lb/ft^3 and SPT "N" values from 20 to 40 blows per foot.

3.1.4 Test Site C, Clayey Soils

Test site C was explored by four test borings from 5 to 5.5 ft deep below ground. Groundwater was observed in all the borings upon completion of the drilling at depths ranging from 1 to 4 ft. While no long-term groundwater observations were made, it was noticed during the field testing that the groundwater table was near the surface. The site is a wooded tract and root systems were encountered on some of the tests. The deposits on Site C are believed to be pleistocene river terrace deposits of the Western Branch of the Patuxent River and generally consist of silty clays overlying sands. On the basis of field observation and in-situ and laboratory tests, the soil is described as a grey, medium stiff to stiff silty clay, with traces of sand and organic matter and classified as Group CL in accordance with ASTM D 2487.

Laboratory tests indicate 70 to 90 percent of material passing No. 200 sieve (particle size smaller than 0.074 mm) and a natural moisture content of 23 percent. Natural dry density varied from 92 to 100 lb/ft^3. The liquid limit varied from 27 to 39 percent and the plasticity index varied from 7 to 19 percent. Standard Penetration Test "N" values varied from 2 to 17 blows per foot. Unconsolidated undrained triaxial compression tests on an undisturbed sample yielded a c of 700 psf and a ϕ of 19°. An unconfined compression test yielded an unconfined compression strength of 1930 psf at 6.7 percent strain.

3.2 TEST SPECIMENS AND PROCEDURES

3.2.1 Anchors Tested

Most anchors tested were of the helical type with 6-in single or 4-in double helixes welded to nominal 5/8 in or 3/4 in shafts. A few tests using three inch single helixes welded to a 3/8 in shaft were used to investigate size effects. See figure 1.2 (page 4) for typical sizes.

Fourteen self-seating swivel (fluke) anchors were also tested. Ten of these were pipe segments of two different sizes (10.25 in long by 1.75 in outer diameter and 6.5 in long by 1.25 in outer diameter) and four were triangular-shaped (arrowhead) plate anchors with 6 in side lengths. See figure 3.1 for details. All the anchors used were commercially available anchors furnished by industry.

3.2.2 Test Apparatus

The anchor tests were performed by the test rig developed for the project as shown in figure 3.2 which has the capability to exert a 10,000 lb vertical or inclined pull against the anchor head. The pulling force can be exerted at an angle to the horizontal of 15° or steeper. The test rig consists of an aluminum tripod with extendable legs (one 4 ft long segment and one 3 ft long segment; in the figure the legs are fully extended). At the apex of the tripod a pulley is installed on a removable axle which is attached to the main leg of the tripod. The anchor is pulled by a chain which passes over the pulley and is attached at one end to the anchor head and at the other end to a pair of push-pull rams which are connected back to back to achieve a long stroke.

The plunger ends of the rams are fitted with a clevis eye and a chain hook, respectively. The clevis eye is attached to a bracket welded to the main leg of the tripod and the chain hook is grabbing the most convenient link of the pulling chain. The rams are designed for a maximum pressure of 10,000 psi which develops a pulling force of 9,800 lb. The 1/2-in thick pulling chain with electro-welded links has a load capacity of 15,000 lb. It was used for pulling because it was readily available and had convenient accessories which made it easy to make length changes as needed.

The front legs of the tripod can be attached to the main leg in two places: they can be attached to the removable pulley axle shown in figure 3.2, or to another location 1 ft down on the main leg which is shown in figure 3.3. This second configuration, which projects the pulley on a 1 ft cantilever, together with the shorter (4 ft) leg length can be used to pull anchors installed under a mobile home. When attached to the axle, the front legs are spread at a fixed 90° angle. The angle between the front legs and the main leg can be adjusted by rotation about the axle.

The tripod is designed to withstand a 10,000 lb pull when the legs are spread so wide that the pulley axle is only 1 ft above ground. In any other position the load capacity would be greater. Except for the pulley and axle, the tripod is fabricated from 6061 T6 high strength aluminum alloy. The legs are made of 4 in diameter tubing. The front legs have a 1/8 in wall thickness and the main leg has a 1/2 in wall thickness. The yoke carrying the pulley and axles, as well as the end sections of the legs are made of solid aluminum.

The legs are restrained from spreading by a 3/8 in thick galvanized aircraft cable and an 8-ft length of chain. The restraining cable that connects the front legs has a fixed length, since the angle between the front legs is fixed

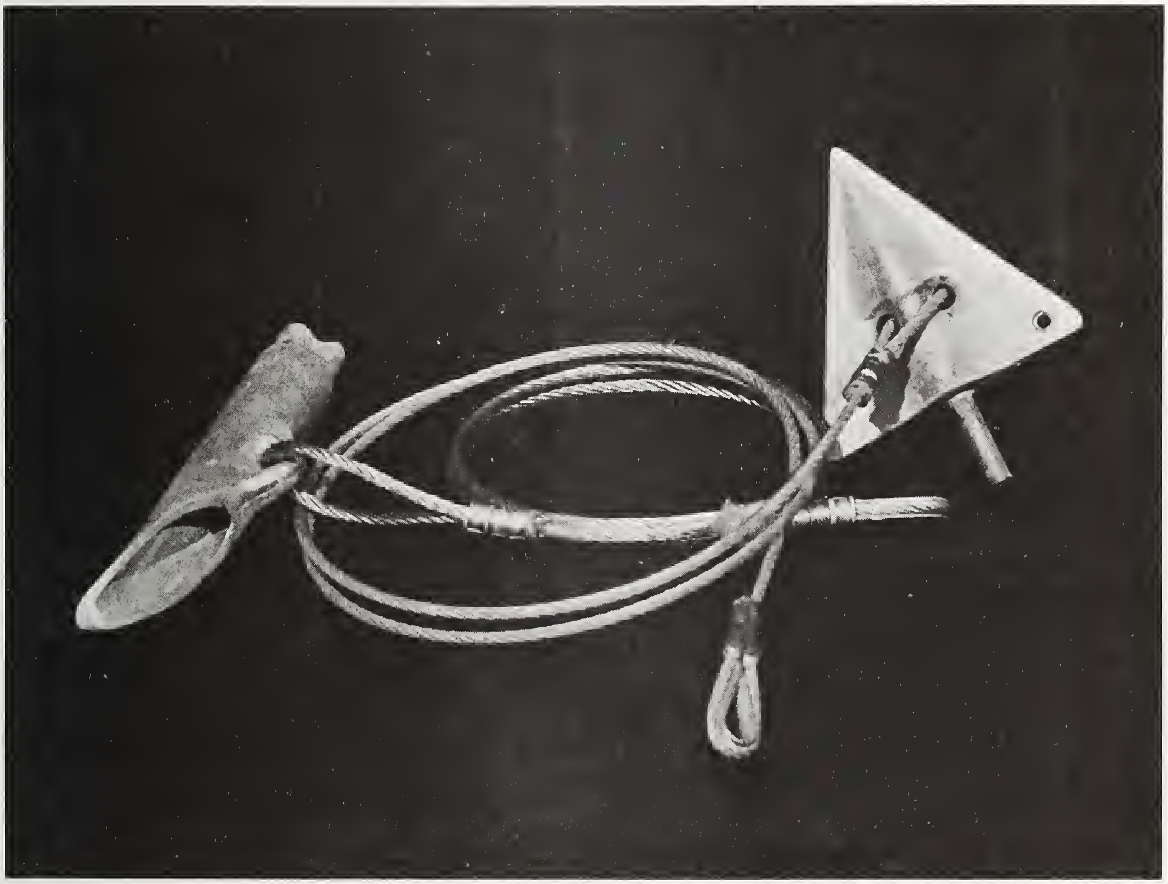


Figure 3.1 Self seating swivel anchors (triangular and pipe) used in the test program

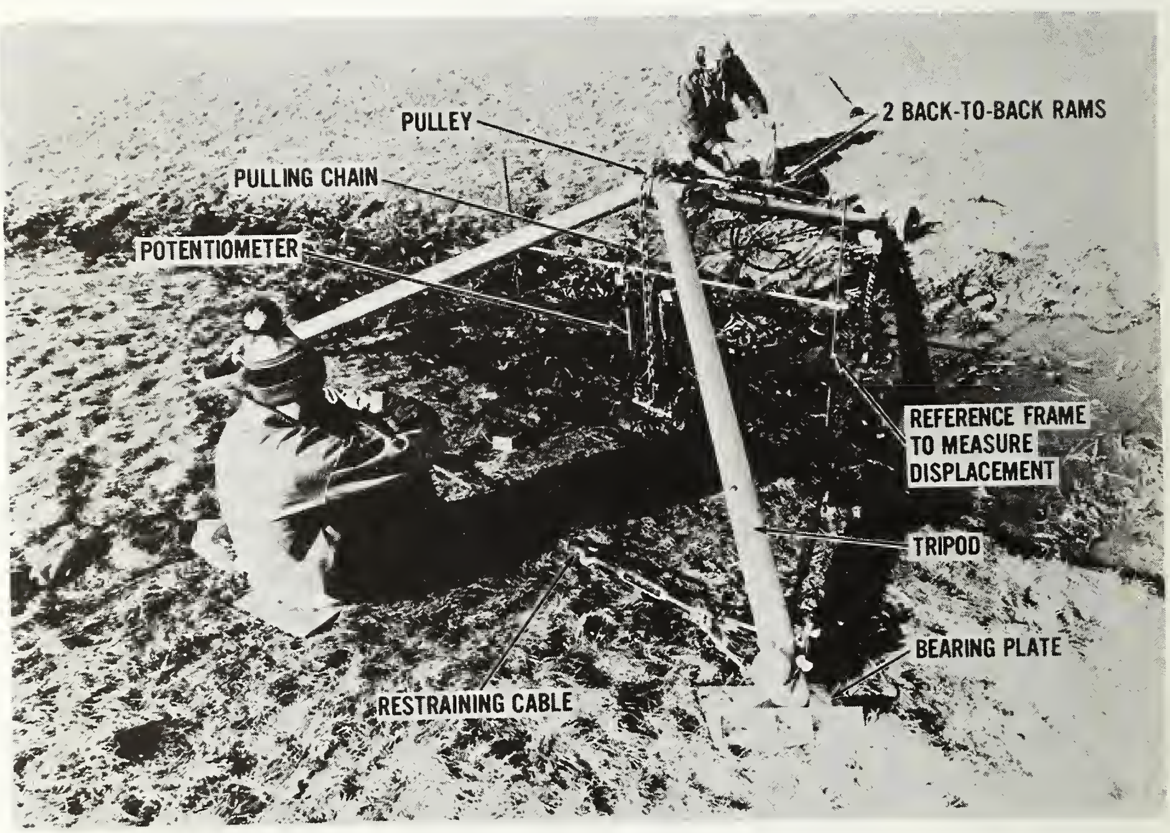


Figure 3.2 Test rig

Figure 3.3 Details of test rig

at 90°. The restraining chain with another segment of cable connects the main leg to the front legs, and is adjustable to accommodate any desired angle between the main leg and the front legs. The restraining chain and cables are designed to resist the forces generated by a 10,000 lb pull in the most unfavorable (lowest) tripod position.

Bearing plates of 1/2 in thick aluminum, 9 in wide by 19 in long, are used to support the legs of the tripod. The plates used under the front legs have a 2 x 2 x 1/4 in aluminum angle welded to the long edge of the top surface of the plate to resist sliding when the tripod is used for inclined pulling. Two aluminum stakes are driven into the ground in front of each bearing plate to provide resistance to lateral forces in angular pulls.

The hydraulic pressure is generated by one of two pumps, depending on the type of testing. For the slow rate (monotonic) testing, a double-acting hand pump is used that has 10,000 psi capability and a useable oil capacity of 126 in^3. A bourdon tube type gage is used for visually monitoring hydraulic pressure. The cyclic testing is done using an electrically driven hydraulic pump mounted on pneumatic tires for rough terrain use. It has a pressure capability of 10,000 psi and a five gallon useable oil supply. The cyclic application of hydraulic pressure is controlled by a program control center having five variations of programmed load application, holding, and removal. A continuous cycling program is used that steps through load application to a given load level, holds the load for a given time, retracts the ram to a position that insures complete removal of the load, counts the cycle and then repeats the cycle.

Suitable signals are provided to the control center by an adjustable pressure switch, a timer, a snap-action limit switch, and an automatic cycle counter. The electricity necessary for running the pump motor, the controller and the instrumentation is provided by a gasoline powered generator mounted on a van which is used to transport the equipment and to serve as a field laboratory that houses the electronics and data recording equipment.

The electronics used consists of a pressure transducer for measuring hydraulic pressure, a 10,000 lb universal load cell, a pair of linear potentiometers and/or a pair of constant-tension cable position transducers to measure anchor-head displacements, an x-y recorder, a volt-ohm meter and a recorder checker. The pressure transducer has a 0-10,000 psi range and is the high output semiconductor instrumented diaphragm type with an output of 25mv/volt full scale. This device is used primarily in the monotonic tests to measure the hydraulic pressure that translates into pulling force. The load cell is used in tension to measure the pulling force between the chain and the head of the anchor and is used in the cyclic tests. Its output is 3mv/volt, full range. The linear potentiometers have a 10 in travel that varies the voltage linearly through the stroke. Electronically the resistances of the two potentiometers are combined to indicate average movement in case the anchor head tilts or rotates. The cable position transducers are also used in the averaging mode for the same reason. These devices are ten-turn precision rotary potentiometers that have a known-diameter pulley mounted on their shafts. A constant tension cable take-up assembly was used in conjunction with the potentiometers by having the cable make one turn around the pulley before exiting the mounting housing. These instruments have many advantages: 1. The stroke can be chosen by changing drive pully diameter; 2. Constant tension makes calibration easy by simply inserting a calibrated length tension link; 3. The constant tension cable device is not influenced by imprecise alignment. The diameter of the pulley used produces a 25 in maximum travel in the test set-up. The recorder checker is used to produce precise voltage for calibration of the x-y recorder.

3.2.3 Testing Procedure

The anchors were installed in a grid pattern in a predetermined area having a a 5 foot grid line spacing (see appendix A). Vertically-installed anchors were spaced 5 ft apart and anchors installed at an angle were spaced at 10 ft. The helical anchors were turned into the ground by an electrically-driven installation tool that turned at nine revolutions per minute. Two 18 in handles were provided on opposite sides of the installing tool to enable two operators to react against the turning torque. Final installation torque was measured with a 0-600 ft lb torque wrench and recorded on the data sheet. The self-seating swivel anchors were installed with an automatic hammering device and a drive rod. Soil test probe readings (STP) were taken at 1-ft depth intervals to the full depth of the probe (48 in). Test probe readings were taken midway between every second pair of anchors. Generally, only enough anchors for one day's testing activity were installed at one time. The tripod was then brought into the position that produced a pull in the desired direction. The bearing pads were placed under the ends of the legs and on angular pulls the stakes were driven into the soil to counteract the pulling force. Two weighted ring stands with an interconnecting rod were used to support the position transducers used to measure anchor movement (see figure 3.2). These position transducers were mounted on opposite sides of the pulling chain in such a way that they measured movement in the direction of the pull. The attachment of the chain to the head of the anchor simulated the typical mobile home tie-down attachment as closely as possible. An appropriate yoke was placed on the anchor head or on the shaft just under the head, protruding from opposite sides of the anchor, to accommodate the attachment of the position transducers. The hydraulic pressure and return lines were attached by means of quick-disconnect couplers. Signal cables from the pressure transducer (or load cell in the cyclic tests) and the position transducers were then connected by mating couplers to the x-y recorder and power supply in the van. Sound powered headphones were used for communicating between the pump operator and the recording technician. After a short calibration check, the test was started.

The monotonic tests were performed at a loading rate of 600 lb per minute until the load capacity of the anchor dropped. At this time, the anchor was pulled out at any convenient speed. Movement of the head of the anchor and the corresponding load were recorded as a load deflection curve plotted on an x-y recorder.

The cyclic tests were performed using the same type anchors, installation procedures and pulling configurations as the monotonic tests. The equipment used was described in section 3.2.2. Most cyclic tests were preceeded by one static load cycle to 83 percent of ultimate load capacity (this level is herein called the preload level.) The ultimate load capacity was taken as the average of previously performed adjacent static tests with the same loading configuration. The load level was then adjusted to two-thirds of the ultimate load (the estimated "design" load) or to other predetermined load levels, and 200 to 300 load cycles were applied. The rise time of a load cycle was generally 30 seconds and the peak load was maintained for 2 1/2 seconds. The specimen was unloaded after each load cycle. The preloading cycle and the first five or ten cycles were recorded on the x-y recorder and subsequent cycles were visually monitored with the recording pen lifted off the paper. Periodic cycles (every 20th, every 50th) or significant events such as increasing creep or incipient failure were recorded by returning the pen to the paper. This practice allowed recording of the rate of creep and important events without covering the paper with repetitious lines.

3.3 TEST PROGRAM

3.3.1 General

Anchor performance is affected by many variables and it was realized early in the project that thousands of tests would be needed to obtain a statistically significant number of tests for each condition. It was therefore decided to conduct a more extensive test program on the silt site (Site A) and keep the number of tests on the other two sites to a minimum.

3.3.2 Test Variables

The test variables considered were: anchor type; anchor size; anchor depth; loading conditions; load orientation; soil type; and soil conditions. These variables are discussed hereafter.

(1) Anchor Type:

Several anchor types suitable for mobile home application are commerically available [12]. Presently the most frequently used anchors are the 6 in diameter single-helix and the 4 in diameter double-helix anchor. Very few data were available for these anchors, and the available data provided only limited information [12]. Another type of shallow anchor which could be adapted for mobile-home use is the self-seating swivel anchor. These anchors were extensively used by the Armed Forces and, as a consequence, more test data were available. It was decided to conduct most tests with single- and double-helix anchors and a limited number of tests with self-seating swivel anchors.

(2) Anchor Size:

Anchor size has a major effect on load capacity. It is generally assumed [12] that anchor-load capacity is proportional to the area of the anchor plate which provides resistance to pullout. To study this parameter, several tests were conducted with 3 in diameter single helix anchors. The result of these tests can be compared with those obtained from tests on 6 in single helix anchors. The 3 inch anchor is not used for mobile homes because of its inadequate load capacity.

(3) Anchor Depth:

It has been determined in previous investigations [12] that there are two types of failure mechanisms for soil anchors: the so-called "shallow" anchors fail by pulling with them a body of soil (cylinder, truncated cone or other) which extends to the ground surface; the so-called "deep" anchors fail without causing a substantial disturbance at the ground surface, since the failure (slip) surface surrounding the anchor does not extend to the ground surface (one study also identifies "intermediate" anchors [8]). As a consequence, the change of load capacity with depth differs for these two types of anchors, and different models for predicting load capacity have to be used. The ratio of anchor depth to anchor diameter (D/B) at which an anchor becomes a "deep" anchor has been estimated to vary from about 4 to 8 [14] and it is therefore not clear whether mobile home anchors can be classified as shallow or deep. Several comparative tests were performed to investigate this parameter, using anchor depths from 1 to 4 ft.

(4) Loading Conditions:

Three loading types were under consideration:

1. Monotonic load cycles, sustained for relatively short periods of time
2. Cyclic loads
3. Long-term substained loads

Under actual field conditions, wind would cause one or two cycles of some maximum load sustained for one or several seconds and many cycles of lesser load; floods would probably cause one load cycle with a relatively slow rise time and sustained for some period of time, ranging from several minutes to several hours; long-term sustained loads could be caused by swelling soil conditions or frost heave.

Since swelling and frost heave effects can not be quantified, and it is also reasonable to anticipate that periodic relaxation and re-tightening of straps as a maintenance procedure would be necessary if such effects are experienced, it was decided not to conduct tests for this loading condition.

To assess windload effects, it is important to get an appreciation of the cyclic-load effects associated with a windstorm. An examination of the strip-chart recording of Hurricane Frederick[2] indicates that the maximum wind velocity at the gaging station (86 mph) occurred twice during the 7-hour period of high winds; a wind pressure equal to 75 percent of the maximum was exceeded 10 times; a wind pressure equal to 60 percent of the maximum pressure was exceeded 86 times; and a wind pressure equal to 50 percent of the maximum pressure was exceeded 162 times. Thus, it was decided that test data from 100 to 200 successive load cycles would provide adequate information to assess the probable effects of a major hurricane. Cyclic loads applied ranged from 50 to 75 percent of the ultimate-load capacity of the anchor. Initially, it was planned to perform monotonic and cyclic-load tests. Later in the program, several cycles of unloading and reloading were included in the monotonic tests in order to evaluate strain-hardening effects.

(5) Load Orientation:

Most anchor-test data available are for pullout in the direction of the anchor shaft. However, as shown in the NBS load study [20], the most important function of mobile-home anchors is to resist horizontal loads. Since most helix anchors are installed vertically or near vertically, and the horizontal load is transmitted to the anchor by a strap which is installed at an angle ranging from 15° to 60° to the horizontal, it is necessary to study anchor performance under this loading condition. To explore the full range of conditions that could be encountered, vertical anchors were subjected to vertical, as well as inclined loads, and anchors installed in an inclined position were subjected to loads in the direction of the shaft, as well as loads normal to the direction of the shaft.

(6) Soil Types:

A great number of soil types are encountered in nature, and oversimplification cannot be avoided if an attempt is made to condense these into a few typical cases. There is evidence [12] that there is a fundamental difference between anchor performance in granular and cohesive soils, since these soils have different strength, drainage, and strain hardening characteristics. The three soil types selected for this project were discussed in section 3.1.

(7) Soil Condition:

Soil condition is an important variable, since moisture content changes seasonally on many sites. In this project two conditions were investigated: the in-situ condition, which did involve some minor fluctuation in water content during the duration of the tests, and the submerged condition which was

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[2] The records were taken on September 13, 1979, at Mobile, Alabama, Muncipal Airport in open terrain and obtained from the National Weather Service Station in Mobile, Alabama.

either obtained by existing site conditions, or artificially induced by flooding. The submerged condition is important since it is necessary to determine whether anchor capacity is reduced under flood conditions.

3.3.3 Summary of Test Program

Table 3.1 provides a summary of the test program. To avoid complexity, the parameters explicitly noted were anchor type, soil type, loading condition and load orientation. Anchor size, anchor depth, and soil condition were omitted. These parameters are identified in appendix B where all the test results are presented.

TABLE 3.1 SUMMARY OF TEST PROGRAM

		STATIC LOADING				CYCLIC LOADING	
	Test Condition / Anchor Type						
Silty Soil	Single Helix	38	12	15	9	11	
	Double Helix	12	12	12		2	
	Other	4	6*	3			
Sandy Soil	Single Helix	15	3	3		4	4
	Double Helix	6	6	3		2	8
	Other						
Clayey Soil	Single Helix	8	3	3		4	4
	Double Helix	3	3	3		7	4
	Other						

* These anchors were self-seating swivel anchors. The anchor itself was inserted vertically. However, when the test load is applied, the connecting cable aligns itself in the direction of the load.

FACING PAGE: *Installing vertical anchor on Site A.*

4. ANALYSIS OF TEST RESULTS

4.1 PRESENTATION OF RESULTS

The test results are presented in appendix B. The data were electronically recorded in the field by an x-y plotter in the form of load versus displacement. The resolution of these plots permitted an estimate of displacement to the nearest 0.01 in and loads to the nearest 0.01 kip. This resolution was compatible with the accuracy of the measurements. Since it would be impractical to reproduce the resulting 232 plots in the report, data points that were derived from the plots are presented in tabular form in tables B.1 through B.6.

Results from the monotonic loading tests are given in tables B.1., B.3 and B.5. Each test is identified by letters and a number. The letters identify the site: ST = silt, SD = sand and C = clay. The test identification is followed by test location coordinates. The corresponding locations are identified in appendix A. The third column identifies the anchor type. H-6, D-4 and H-3 mean 6 in single helix, 4 in double helix and 3 in single helix, respectively, AH means arrowhead (triangular) anchor and P 10 and P 6 mean 10 1/4 and 6 1/2 in long pipe anchors. (The arrowhead and pipe anchors are self seating swivel anchors.) The anchors are further identified by make (only identified by letter). The pull direction is identified as A-axial or I-inclined. The angle of load and anchor inclination to the horizontal are identified by α_1 and α_2, respectively. The "depth" identified is the vertical depth from the ground surface to the anchor tip. To get the depth of a helix plate the distance from the tip to the center of the helix (3 in for most anchors tested) must be subtracted (see figure 1.2). The loading is identified as SM-static monotonic and SUR-static with unloading and reloading cycles (cyclic loading tests are identified separately). The soil condition is identified as M(natural moisture content) W-wet or S-submerged. Subsequently, the soil test probe reading - STP and the installation torque-T_i are identified.

Figure 4.1 shows a typical plot of a load-displacement curve for an anchor pulled vertically and installed vertically on Site C (clay). The data points recorded for the monotonic tests with reloading cycles are shown in figure 4.1 and explained hereafter. Note that most "monotonic" tests contained 2 cycles of unloading and re-loading at each 1 kip increment of load and 1 cycle of unloading and re-loading near the point of maximum load (Q_u). The following data points are recorded:

> P_{2v} is the anchor load at 2 in vertical displacement; (P_{4h}, the anchor load at 4 in horizontal displacement is used for anchors installed vertically and pulled at an angle). For anchors installed at an angle to the vertical and pulled axially, the loads recorded under P_{2v} and P_{4h} are the loads for the 2 in vertical and 4 in horizontal displacement components, respectively. In some instances these 2 and 4 in displacements include residual displacements from unloading and reloading cycles.
>
> Q_u is the "ultimate" load (the maximum load attained by the anchor during pullout).
>
> Δ_u is the displacement at ultimate load measured in the direction of the initial pull. For anchors installed vertically and pulled at an angle α_1 to the horizontal, the horizontal displacement component, Δ_{uh}, can be approximately calculated by the equation $\Delta_{uh} = \Delta_u/\cos\alpha_1$, since for these anchors the anchor head displacement tended to be horizontal with only a minor vertical component. For anchors pulled axially or near axially Δ_u is an axial displacement. For the few anchors, where the pull was in a

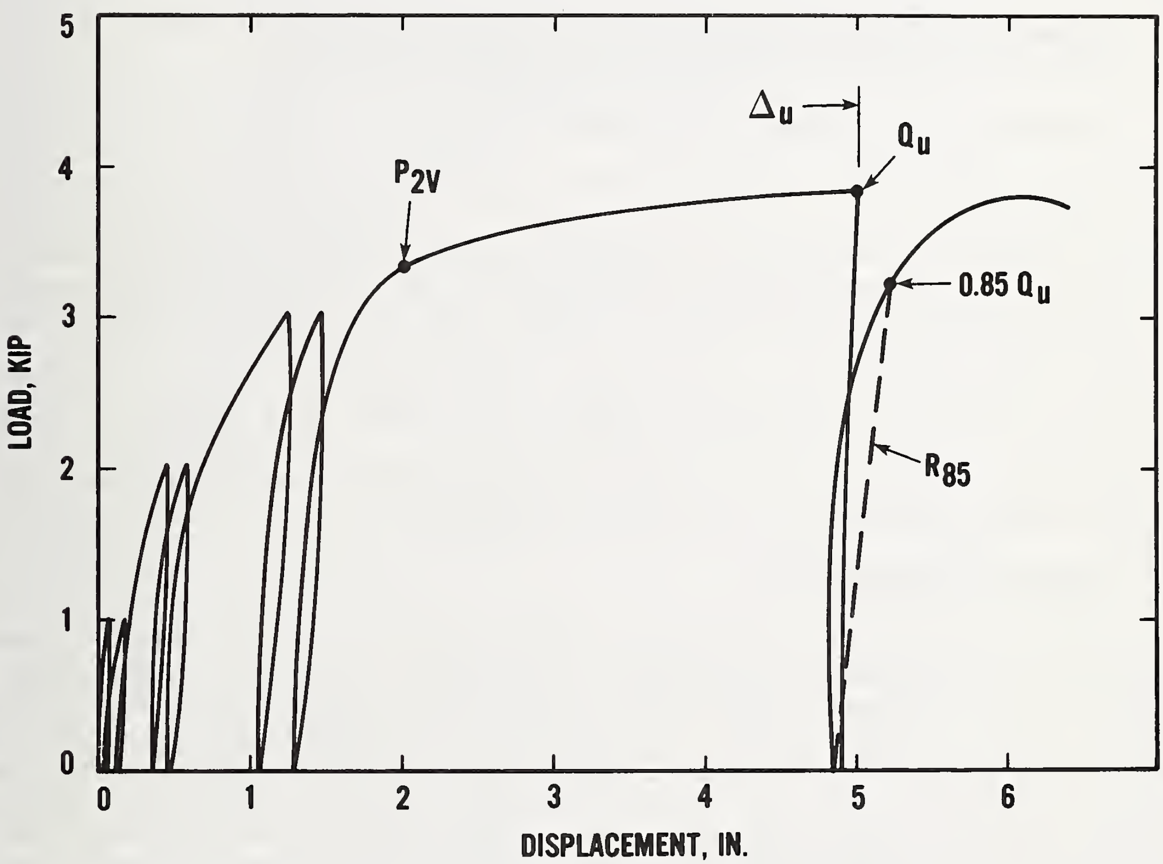

Figure 4.1 Plot of test C-7, vertical 6-in single helix anchor, vertical pull

direction normal to the shaft, Δ_u is the displacement in the direction of the pull.

R_{85} is the secant re-loading modulus at 85 percent of the load before unloading, measured at the highest load at which unloading and re-loading was performed (not necessarily Q_u as in figure 4.2) expressed as a ratio of load to displacement (lb/in).

The results of the cyclic tests are given in tables B.2, B.4 and B.6. The symbols for the data shown in the cyclic tests are explained hereafter:

P_c/Q_u is the ratio of the cyclic load to the estimated ultimate load obtained from monotonic tests in an adjacent location

N is the number of load cycles used

P_p/Q_u is the ratio of preload (if any) to the estimated ultimate load

Δ_1, Δ_{10}, Δ_{100}, and Δ_u are the total displacements in the first, tenth, hundredth and last load cycle. (They are the sum of the displacement caused by the applied load in the last load cycle and the cumulative residual displacements from all previous load cycles.)

R_{10} is the secant re-loading modulus in the 10th load cycle

Q_u' is the ultimate load actually obtained when the anchor was pulled out after completion of the cyclic loading.

4.2 EFFECT OF LOADING RATE

The effect of the loading rate on the characteristics of the load-displacement curve was investigated early in the test program in order to decide on the loading rate to be used in the tests. The fastest rate at which load could be applied in the monotonic tests was limited by the pumping capacity of the manually operated hydraulic system and in each case also depended on the ram displacement associated with a particular load increment. On the average, the fastest initial loading rate was approximately 4,000 lb. per minute. This rate has a tendency to decrease as the ultimate load is approached. Thus it was not possible to investigate dynamic load effects.

The solid curve in figure 4.2 is the record of a test in which load increments were applied at a fast rate, and after each load increment the load was held until no measurable creep was recorded over a 15-minute period. For the load increments up to 4.25 kip, creep virtually ceased after 5 minutes. In the last two load increments creep continued for 20 minutes.

The shaded area in figure 4.2 is the estimated range of creep effects that could be anticipated. The upper bound of this range represents the most rapid load application possible with the available equipment, and the lower bound a

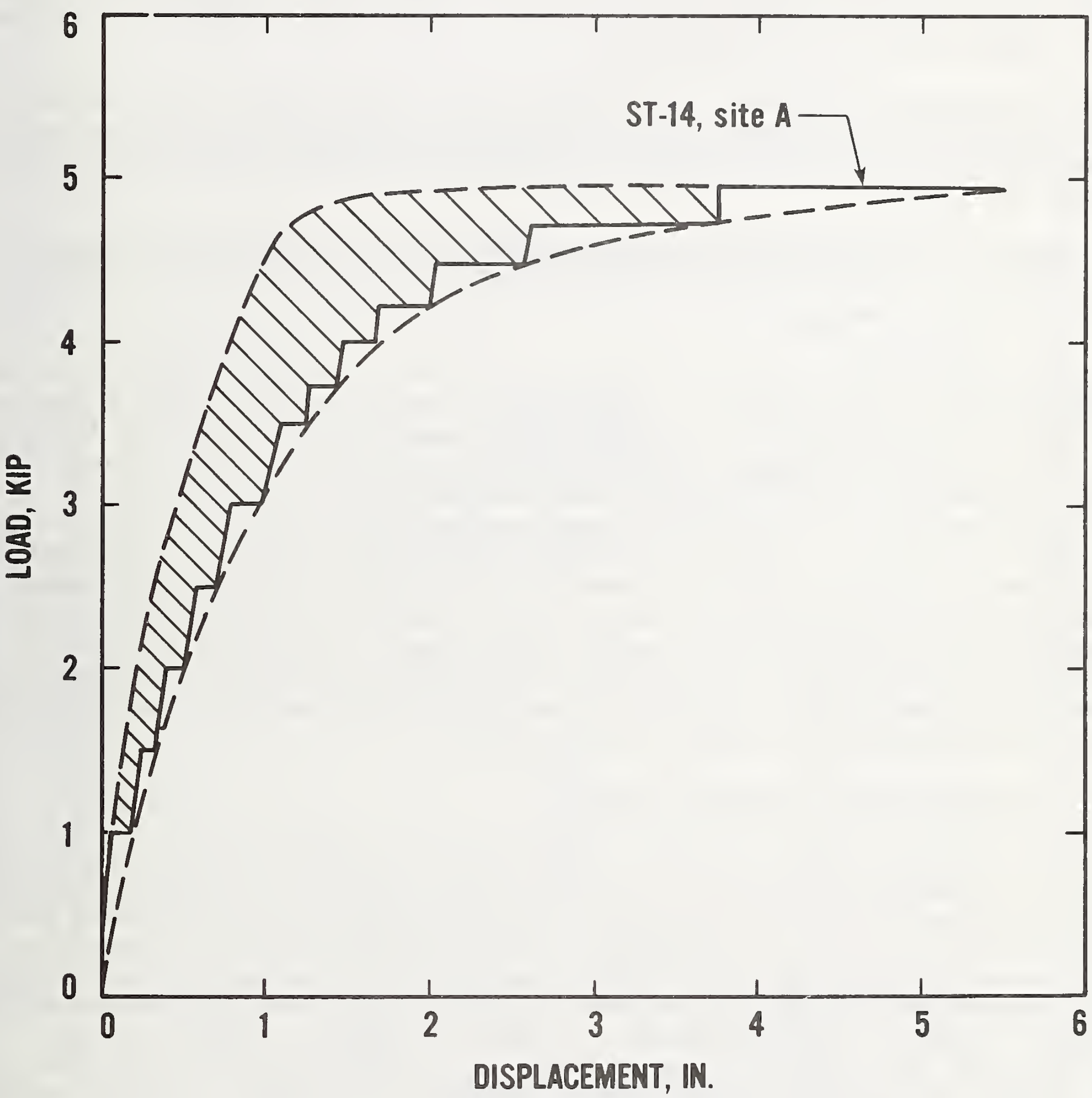

Figure 4.2 Effect of loading rate on load-displacement characteristics

load-displacement relationship that would not be altered by further decreasing the loading rates (except that further creep would probably occur if a load increment would be held for a very long period of time, such as several days). On the basis of this, and several other observations, the load in the monotonic tests was applied at a rate of 600 lb/minute. While this loading rate would not produce the lower-bound curve in figure 4.2, the test results are assumed to be close to the lower-bound curve.

Cyclic loads were applied at a much faster rate, since these were intended to simulate windloads. However, the 30 second rise time for the load was much slower than the typical windload. The loading rate was limited by the capacity of the oil pump.

4.3 STATIC LOAD-DISPLACEMENT CHARACTERISTICS OF 6-in SINGLE HELIX ANCHORS

4.3.1 Monotonic Tests

Figure 4.3 shows the result of a typical monotonic loading test on a 6-in single helix anchor on Site A (silt). The test is a vertical pullout test which was carried to full withdrawal. Vertical displacements of the anchor head in inches are plotted against applied load in kip. Note that the initial portion of the load displacement curve is rather steep and there is a break at point A, at a load of about 1.1 kip. This break was characteristic of many, though not all the tests. There is a gradual, but not very drastic decrease in stiffness until the anchor yields at a load of approximately 4.8 kip. The anchor subsequently maintained its load resistance during an additional 10 in withdrawal, and a reduced load resistance over an even larger range of displacements which is not recorded. Ductile behavior was characteristic of soil anchors tested on Sites A and C (silt and clay), even though the range of displacements over which the load is maintained varied with the soil type.

4.3.2 Unloading and Reloading Cycles

Figure 4.4 shows a vertical pullout test on a 6-in single helix anchor in sand (Site B) installed vertically to its full 4-foot depth. Two cycles of unloading and re-loading were conducted at 1 kip intervals in order to assess the characteristics of pre-loaded anchors. The re-loading curves are generally much steeper than the initial "virgin" loading curve, indicating substantial strain-hardening effects. The characteristics of the curve are interpreted as follows: Whenever load is applied, the soil is compacted, and up to the applied load, its load-displacement characteristics are modified. As soon as the applied load exceeds the pre-load, the load-displacement curve follows the shape of the virgin curve which would be obtained in monotonic loading except that some displacement of the curve will have occurred as a result of the unloading and reloading cycles. The initial break in the virgin curve, which was observed at point "A" on figure 4.3, can probably be attributed to preconsolidation of the soil which was about equivalent to a 1.1 kip anchor pull.

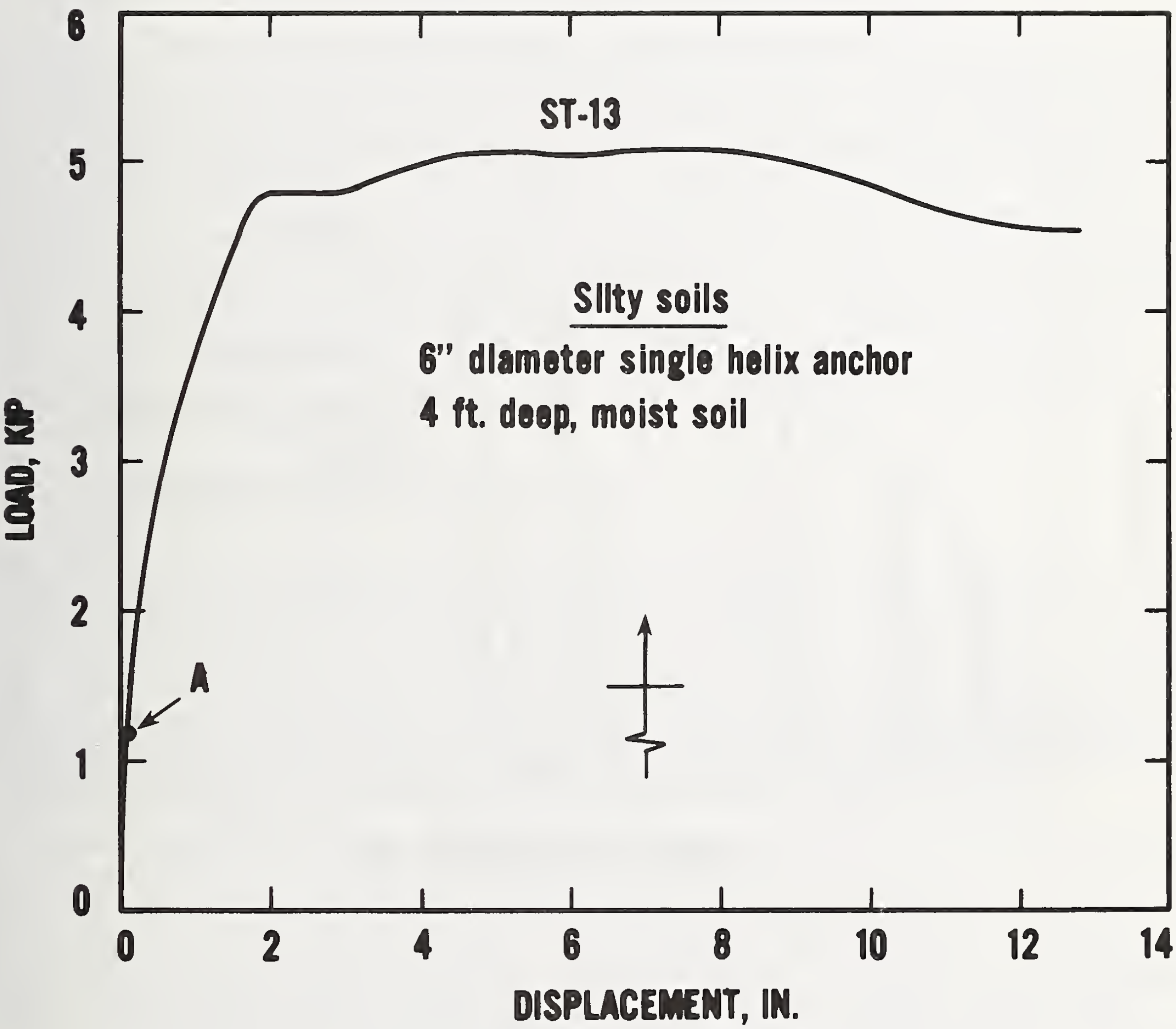

Figure 4.3 Monotonic loading test in silt

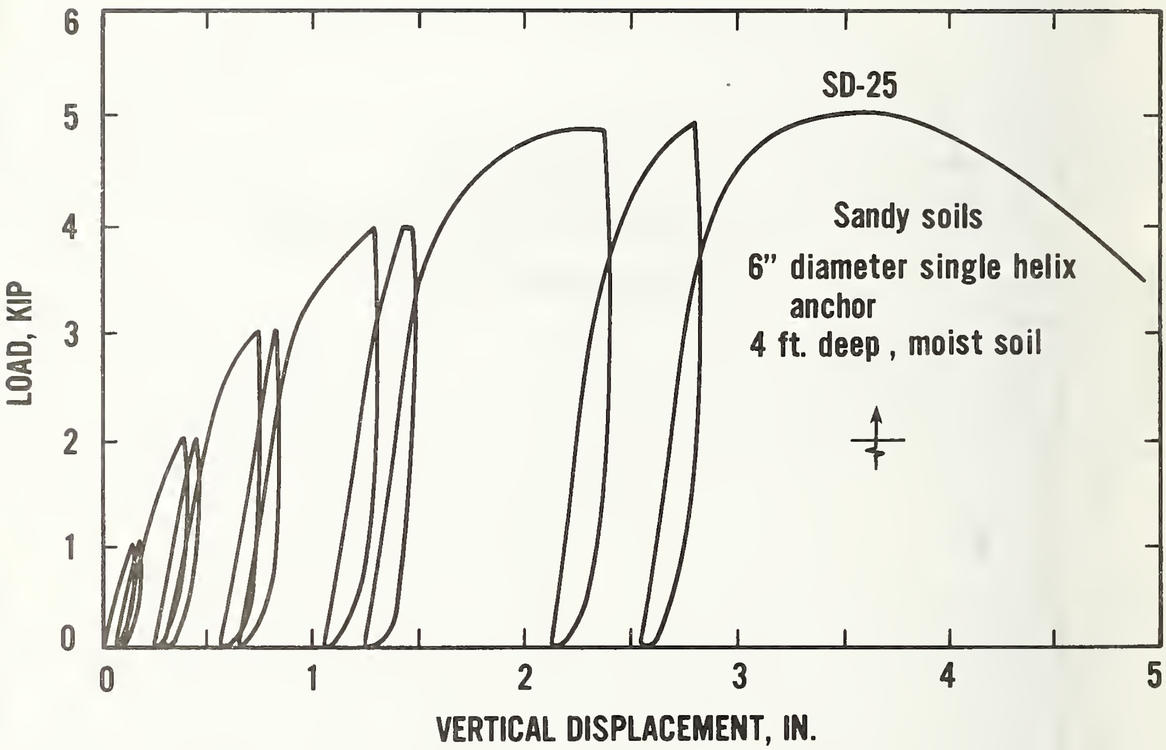

Figure 4.4 Pullout test with unloading and reloading cycles in sand

4.3.3 Anchors Installed at an Angle and Pulled Coaxially

Figure 4.5 shows the load-displacement curve for an approximately coaxial pull on an anchor which was installed on Site B (sand) at an angle of 45° to the horizontal. Note that this curve is similar to the one shown in figure 4.4, except that the load capacity is much lower because of the reduced anchor depth due to the 45° installation angle. The break in the virgin curve is very pronounced and occurs at about 0.5 kip.

4.3.4 Anchors Installed Vertically and Pulled at an Angle

Figure 4.6 shows the load-displacement curve for a vertically installed anchor on Site B, pulled at an angle of 40° to the horizontal. The initial stiffness of this anchor is very low (only 1.2 kip capacity at a 4-inch displacement), since the 5/8-inch thick shaft developes very little lateral soil resistance as it is pulled horizontally into the soil. However, as the shaft is bent in the direction of the pull the soil resistance increases and the ultimate pullout resistance exceeds that for a vertical pull. The initial, flatter slope of the re-loading curves is attributable to the elastic rebound of the anchor shaft which occurs before the soil resistance is engaged. Otherwise the re-loading curves show characteristics similar to those in the previously discussed tests. The anchor deformation in these tests is illustrated in figure 4.7 which shows a 4-in double helix anchor which was exposed by excavation after completion of a similar test.

4.3.5 Effects of Loading Configuration

The reconstructed virgin curves for the tests shown in figures 4.4 through 4.6 which are for tests performed in similar soil conditions, are plotted in figure 4.8. When these envelope curves were drawn, displacement caused by the unloading and re-loading cycles were estimated and subtracted from the total displacement. The figure illustrates the difference in the performance characteristics. Note the large displacement required to develop load resistance in a vertically installed anchor subjected to diagonal load, the most commonly encountered situation associated with present mobile home anchoring technology.

The load capacity (ultimate load) of the coaxially pulled inclined anchor (test SD 29) was about 50 percent of the load capacity developed in test SD 25. Anchor SD 25 was installed at a depth of 4 ft and anchor SD 29, because of its inclination, at a depth of 2.8 ft. Thus a 30 percent increase in embedment depth caused a 50 percent increase in load capacity. Anchor SD 30 developed a higher load capacity than anchor SD 25, however a much larger displacement occurred before the full load capacity was developed. Much of this displacement is attributable to the bending of the anchor shaft (see figure 4.7). The load capacity of anchor SD 30 was 6 kip and that of SD 29 was 5 kip. Thus load capacity increased by 20 percent. This trend was consistently observed, regardless of soil conditions. The trend is further illustrated in figure 4.9, which shows a comparative plot of tests on vertically installed anchors performed at different load inclinations. These tests were performed on Site A (silty

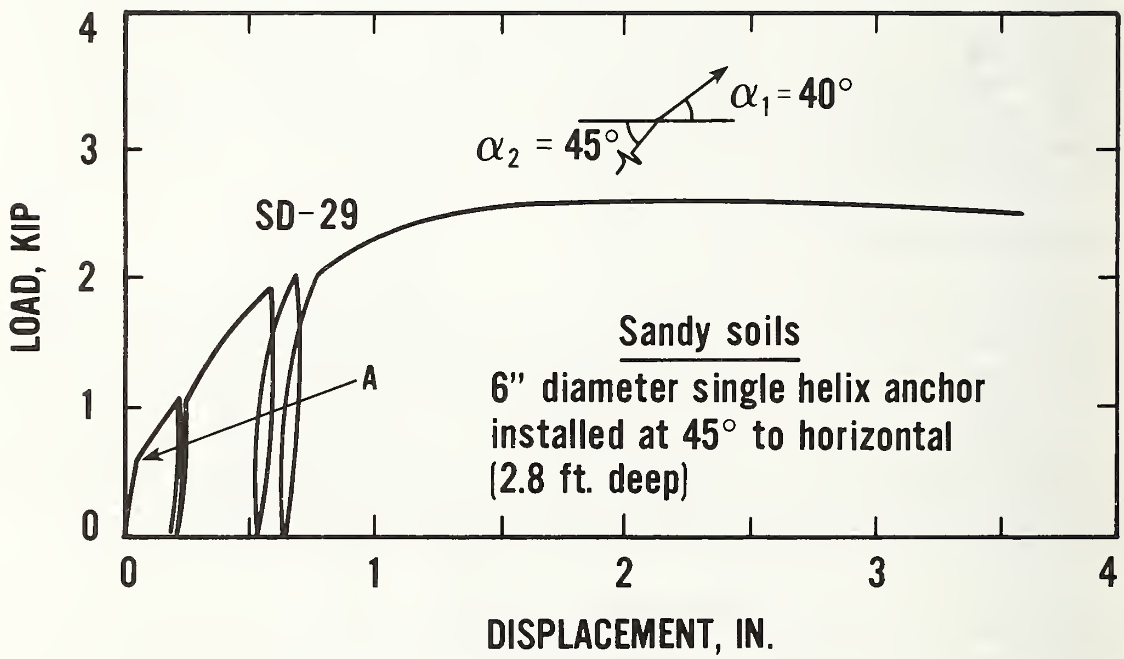

Figure 4.5 Coaxial pullout test on inclined 6-in single helix anchor in sand

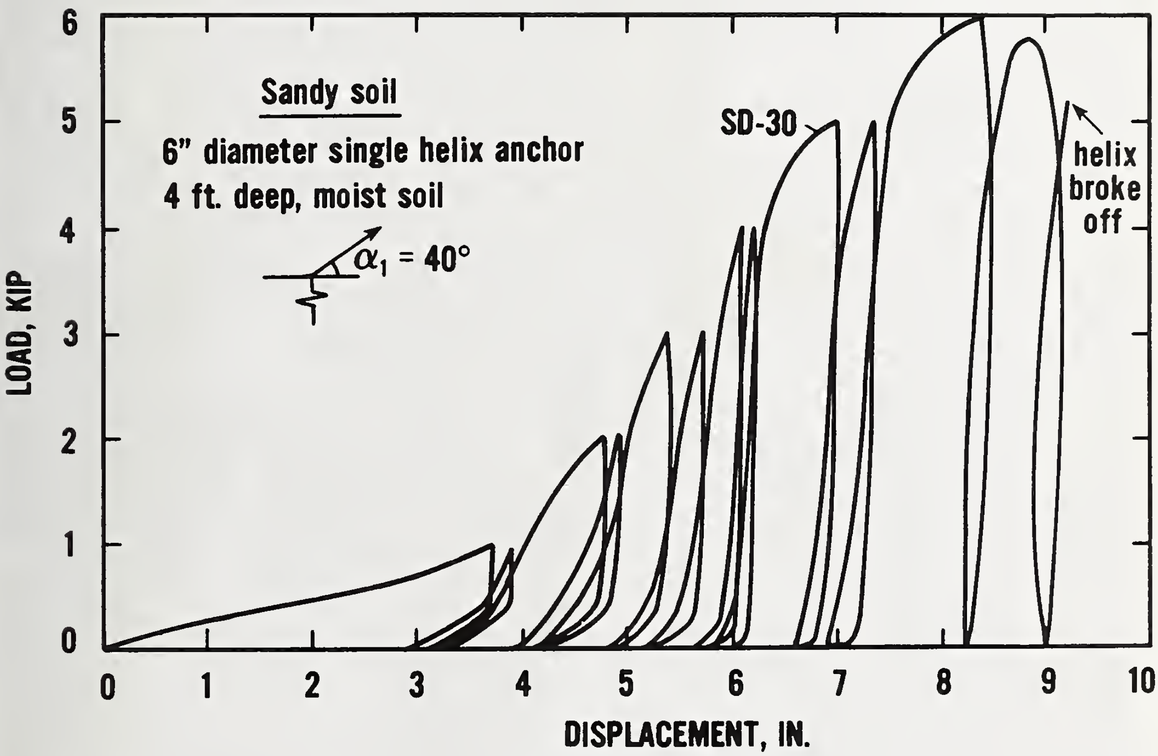

Figure 4.6 Inclined pullout test on a vertically installed 6-in single helix anchor in sand

Figure 4.7 Anchor deformation in inclined test on vertically installed 4-in double helix anchor

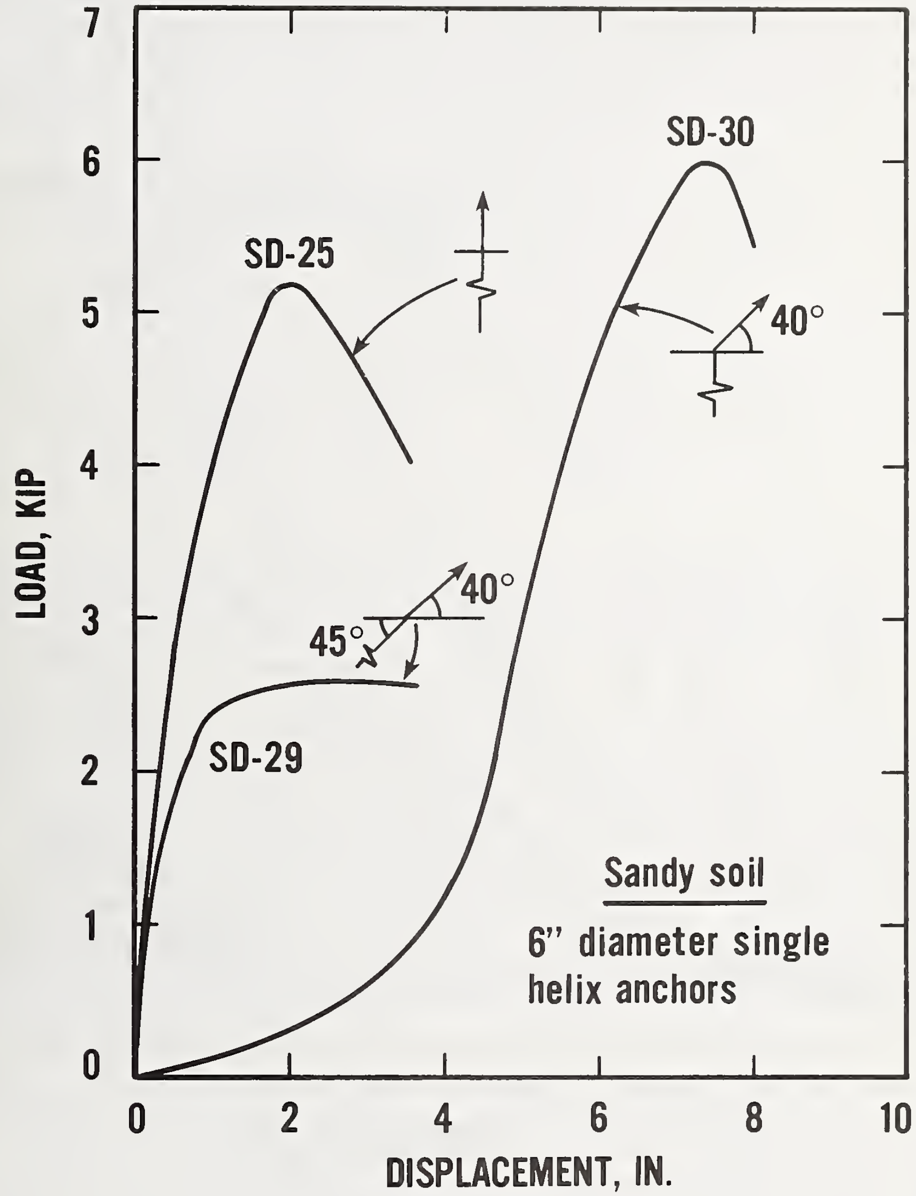

Figure 4.8 Effect of loading configuration on load-displacement characteristics of anchors

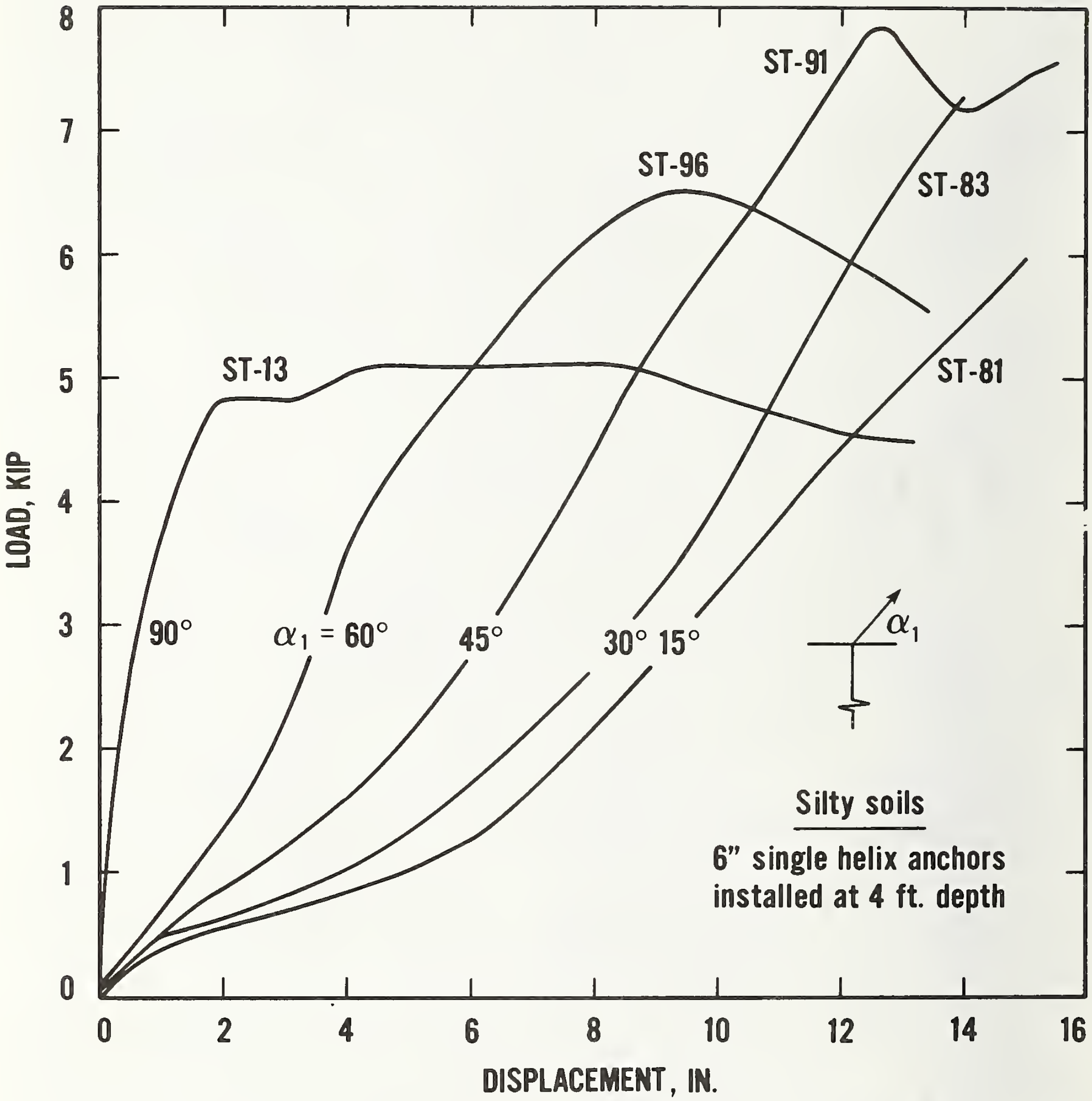

Figure 4.9 Effect of load inclination on the load-displacement characteristics of vertically installed anchors

soils). Note that as angle α_1 decreases from 90° to 15°, the load-deflection slope (stiffness) of the virgin load-displacement curve decreases, but the load capacity of the anchor increases (at the flatter angles the full load capacity could not be realized because of failure of the anchor hardware). It is interesting to note that while the virgin load-displacement curves show great differences in stiffness, the re-loading characteristics are quite similar for the various load inclinations and resemble those illustrated in figure 4.6. This is shown in table 4.1.

Table 4.1 Reloading Moduli (R_{85}) for the Tests Shown in Figure 4.9

Test Specimen	α_1°	R_{85}, lb/in
ST 13	90	-
ST 96	60	3980
ST 91	45	3610
ST 83	30	5020
ST 81	15	3500

There is no re-loading curve for test ST 13 since the importance of the re-loading characteristics was only realized at a later stage in the test program. However, data from test ST 122, a vertical pull-out test conducted at the same site, indicate a re-loading modulus of 17,000 lb/in.

Note from figure 4.6 that the re-loading curves can be divided into two segments. Initially, there is a displacement of the order of about 1/2 in where the re-loading curve is relatively flat. This part of the re-loading curve is attributable to the bending of the anchor shaft which rebounds elastically when it is unloaded. Subsequently, the re-loading curves are very steep with moduli similar to the one observed for test ST 122 (see above). The two segments of the re-loading curve are combined when R_{85} is determined. <u>It is important to note that the re-loading moduli observed in the inclined tests of vertically-installed anchors far exceed those that would be required to satisfy present standards and regulations</u> [2, 17].

Another loading configuration that was tested on the silty site is non-coaxial loading on inclined anchors (α_2 = 135°, α_1 = 15°, 45° and 60° - see sketch in figure 4.10). Similar configurations occur in practice, since it is difficult to insert vertical soil anchors under the outer walls of an installed mobile home. Typical test results are shown in figure 4.10. Because of the large displacements associated with this loading configuration the displacement scale is compressed. Note that, as in the case of vertical anchors with inclined loading, the stiffness of the anchor decreases as the angle of the load with the horizontal decreases. Test ST 91 was plotted in figure 4.10 for comparison. Note that the performance of anchors ST 102, 105 and 108 is very poor when compared with vertical anchors with inclined loading, which also experience considerable displacements before developing their load capacity. <u>It is obvious</u>

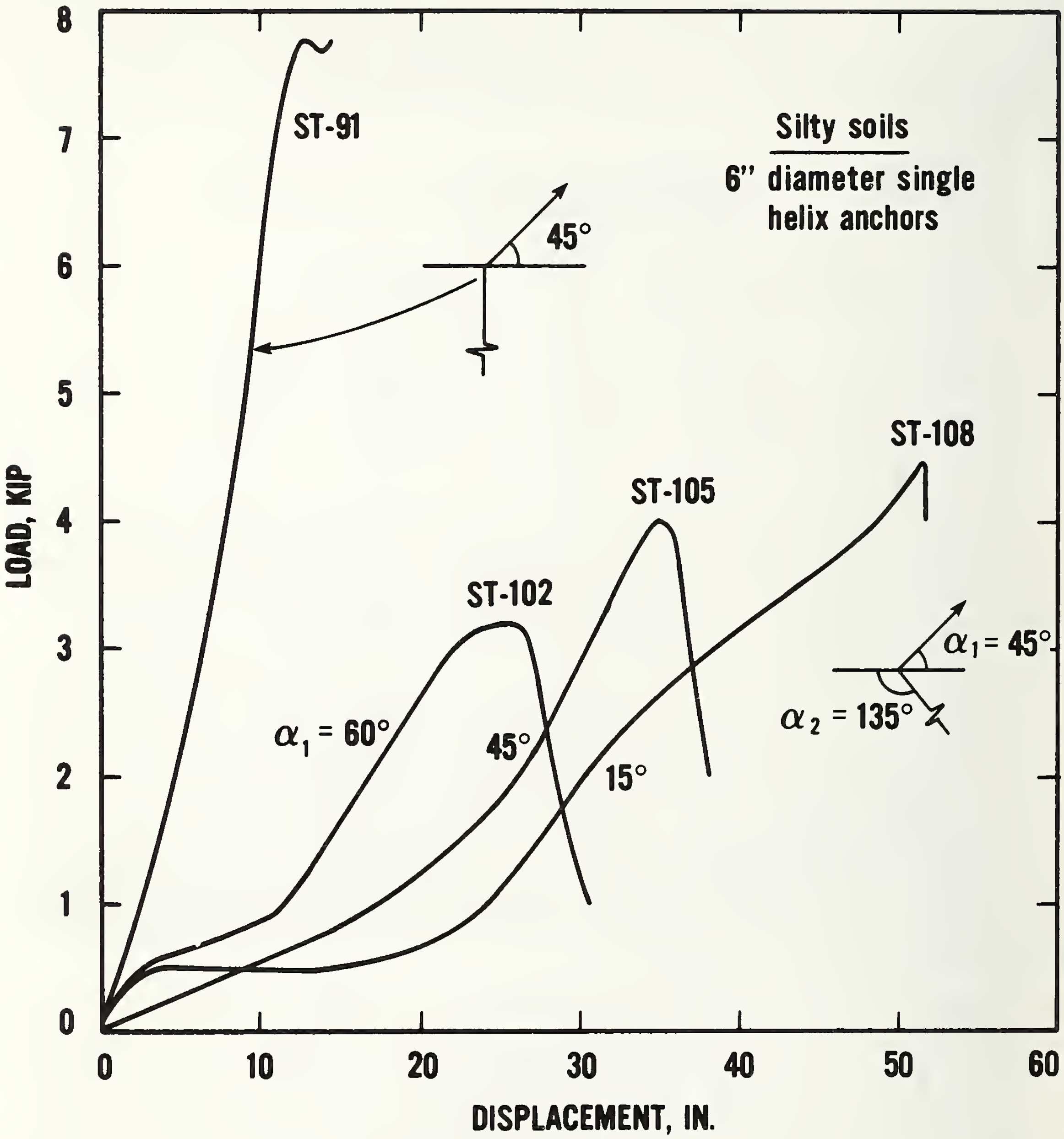

Figure 4.10 Load-displacement characteristics of inclined anchors subjected to non-coaxial loads on the silty site.

from the test results that this is the least desirable loading configuration tested. Thus if anchors are tilted away from the mobile home during installation their stiffness and load capacity are likely to decrease.

4.3.6 Effects of Inclination and Depth on the Load Capacity of Coaxially Loaded Anchors

The effect of the angle of load and anchor inclination on coaxially loaded inclined anchors is illustrated in figure 4.11. As expected, the load capacity decreases as the angle of anchor installation becomes flatter. Anchor capacity in this case may be influenced by two opposing effects: As the installation angle becomes flatter, the depth of the anchor decreases since the embedment length of approximately 4 ft remains the same. On the other hand, some test results and load capacity hypotheses [14, 11] indicate that, for a given depth, anchor capacity increases as the installation angle becomes flatter. However, these observations are not corroborated by other investigators. For instance Harvey and Burley [10] found that coaxial pull-out capacity for shallow anchors in sand for the same depth of embedment is approximately the same for vertically installed and inclined anchors. The test results obtained in this project provide some information that can be compared with the above discussed data. Figure 4.12 shows the results of pull-out tests of anchors installed at various depths on Site A. Note that there was no significant difference between the anchors installed at 3 and 4 ft depths. This is an indication that anchors deeper than 3 ft experienced local failure (the failure surface did not extend to the ground surface). The depth to diameter (D/B) ratio for the 3 ft deep anchors is 5.5 and that for the 4 ft deep anchors is 7.5, and the observation that the 3 ft deep anchors acted like deep anchors would be in agreement with data obtained by others [8, 12].

Two interesting observations can be made from the comparative plots in figure 4.12: 1. As the anchors become shallower, their peak capacity is reached at an increasingly smaller displacement and their "ductility" decreases. This is probably related to the failure surface developing as the anchor is withdrawn. 2. The load capacity of an anchor, embedded at a given depth is not unique and depends on the original embedment depth. Thus the anchor which was initially embedded 3 ft resisted more than 4.5 kip after it was withdrawn 13 inches and was 1 ft 11 in deep. At the same depth the initially 2-ft deep anchor resisted only a 3 kip load. This phenomenon can be explained by the development of a unique failure surface for each anchor depth, which is associated with the virgin load-deflection curve. The soil mass within this surface is compacted by the applied load. This compaction accounts for the strain hardening effect evident in the re-loading curves. When the anchor is pulled out to a shallower depth the compacted soil mass within the slip surface moves up with it, and the shape of the slip surface does not substantially change.

Figure 4.13 shows a comparison between the pullout strengths of co-axially loaded vertical and coaxially loaded inclined anchors for various installation depths. For the 1 ft depth the inclined anchor had substantially higher strength than the vertical anchor. However, for the other depths the results

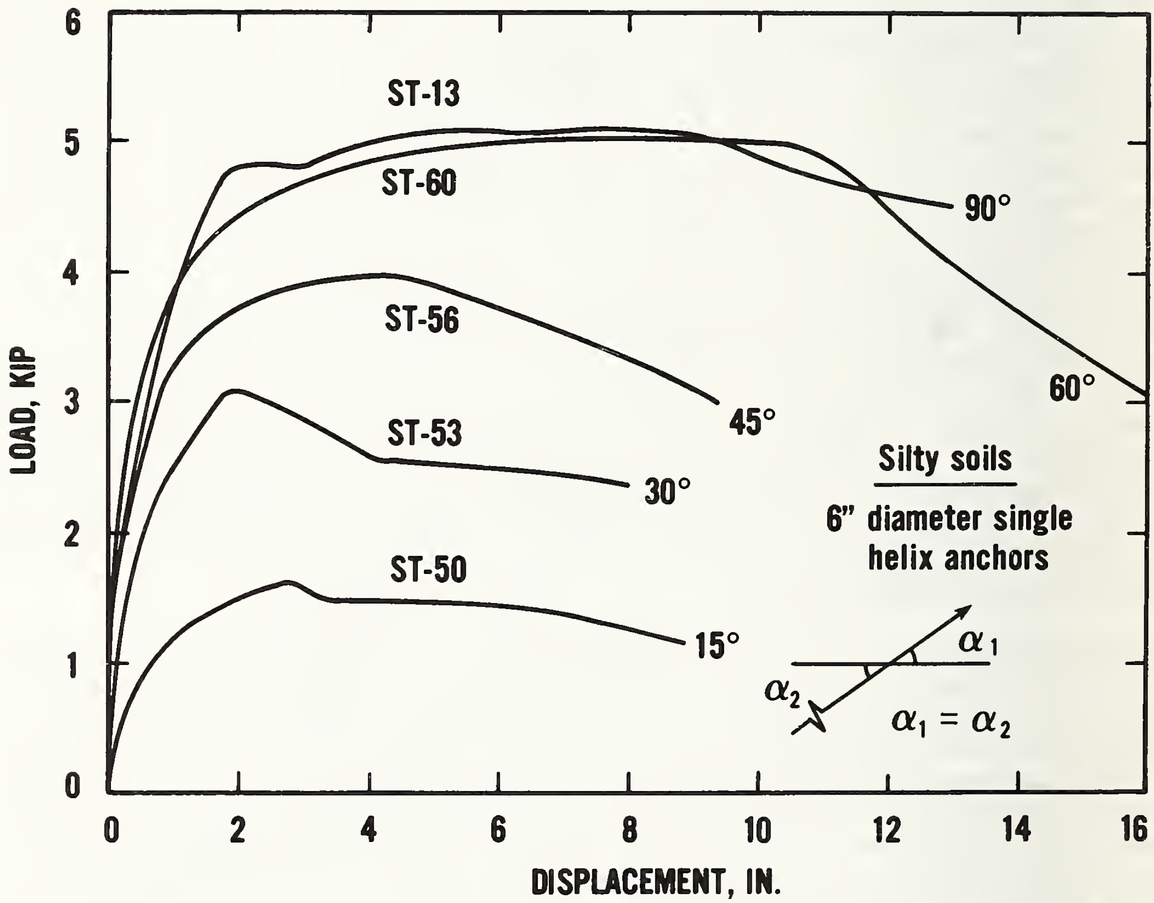

Figure 4.11 Load-deflection curves for coaxially loaded 6-in single helix anchors installed at various angles

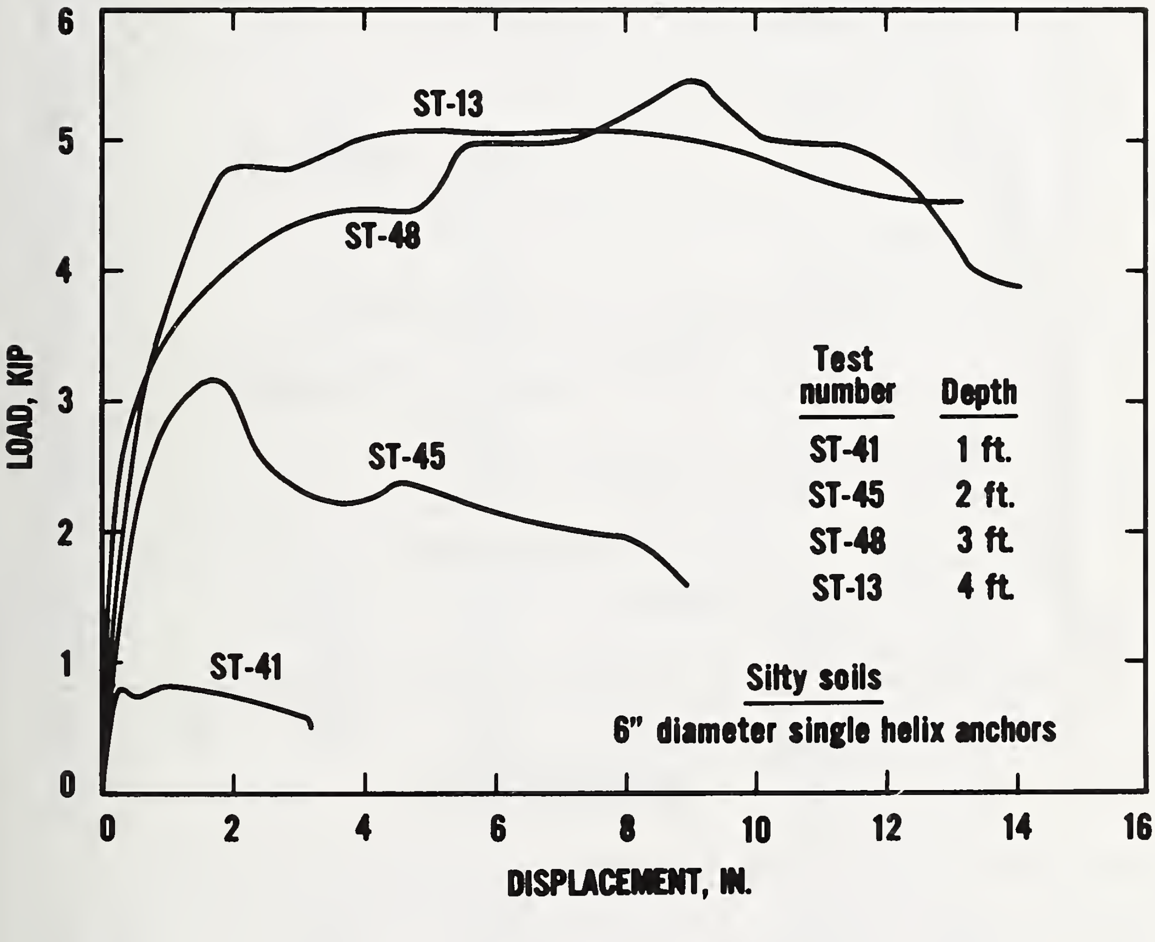

Figure 4.12 Effects of embedment depth on the load capacity of vertically installed 6-in single helix anchors subjected to coaxial load

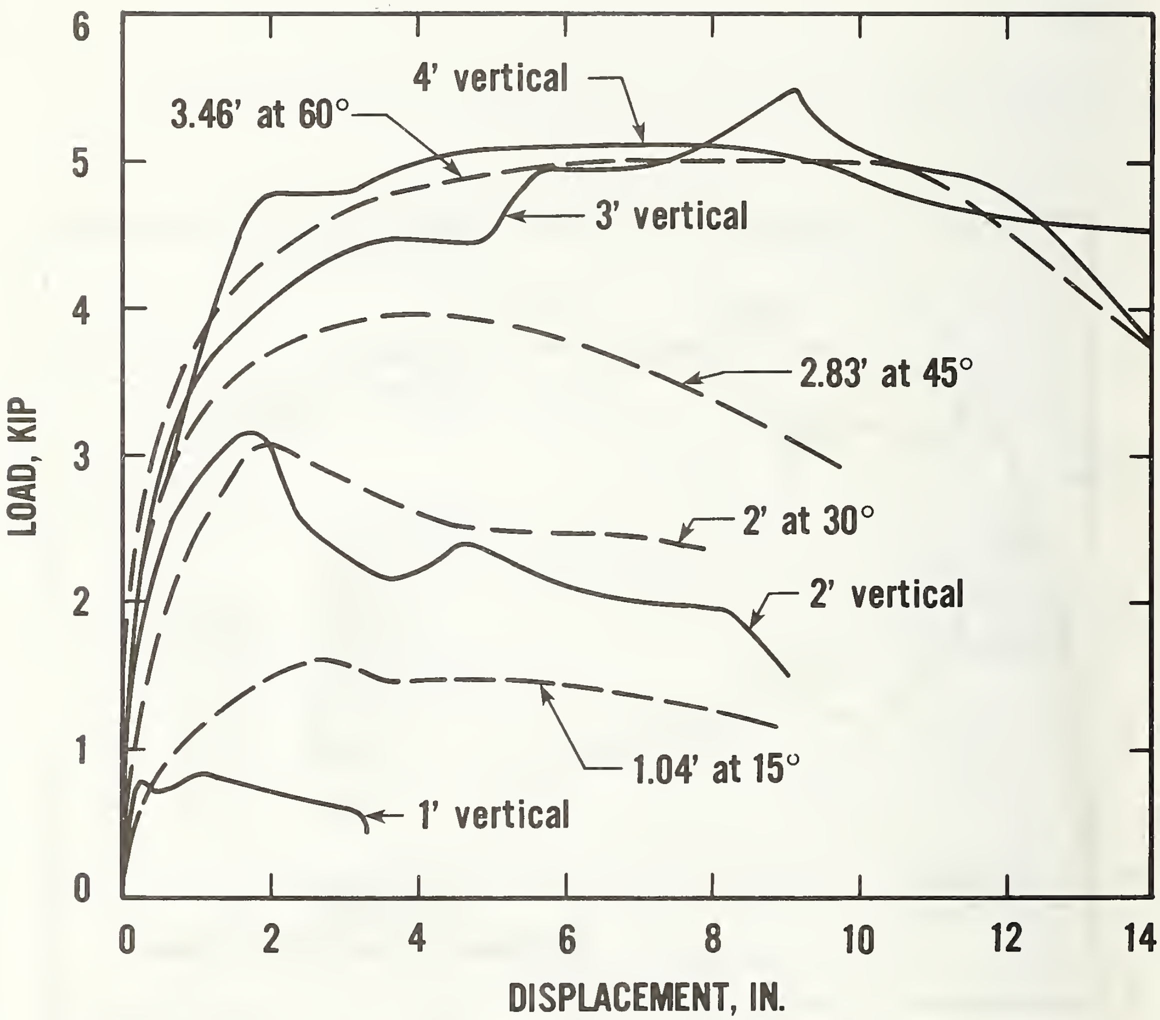

Figure 4.13 Comparison between inclined and vertical coaxially pulled 6-in single helix anchors

of the inclined-anchor tests fall into the same pattern as those of the vertical-anchor tests when anchor depth is correlated with load capacity, and there is no evidence that the load capaity of inclined anchors is greater than that of vertical anchors installed at the same depth. To ascertain whether the test results are significant, all the test results are tabulated in table 4.2.

Table 4.2 Comparison of the Strength of Coaxially Loaded Anchors Installed at Various Depths

Test No.	Depth, ft	Inclination α_2	Q_u, lb	$\overline{Q}_u$, lb	v
ST 40	1	90° (vertical)	700		
ST 41	1	90°	800	840	0.19
ST 42	1	90°	1020		
ST 49	1.02	15°	1280		
ST 50	1.02	15°	1610	1447	0.11
ST 51	1.02	15°	1450		
ST 43	2	90°	3120		
ST 44	2	90°	3300	3200	0.03
ST 45	2	90°	3180		
ST 52	2	30°	3420		
ST 53	2	30°	3120	2880	0.24
ST 54	2	30°	2090		
ST 55	2.83	45°	4220		
ST 56	2.83	45°	4000	4247	0.06
ST 57	2.83	45°	4520		
ST 46	3	90°	5250		
ST 47	3	90°	5600	5217	0.08
ST 48	3	90°	4800		
ST 58	3.46	60°	5280		
ST 59	3.46	60°	4300	4563	0.14
ST 60	3.46	60°	4110		

$\overline{Q}_u$ = average load capacity, lb
v = coefficient of variation of the sample

It can be seen from table 4.2 that with the exception of tests ST 54, ST 59 and ST 60 the trend is reasonably consistent. This leaves the question why the 15° anchors have a consistently higher load capacity than the 1 ft deep vertical anchors, while in all other instances the load capacity of inclined anchors was approximately equal to that of vertical anchors of the same depth. The explanation is probably in the fact that the pull exerted by the 15° anchors is predominantly horizontal creating a failure surface the geometry of which differs substantially from that of the 1 ft deep vertical anchor. The field notes indicate that the failure of the 1 ft deep vertical anchors created a 24 in diameter soil mound and that of the 15° anchors a 22 x 32 in mound "along the axis of the anchor".

4.3.7 Comparison of Coaxially Loaded Inclined Anchors and Vertical Anchors Subjected to Inclined Loads

Anchors which have to resist inclined loads (containing a horizontal load component) are in present practice installed vertically. However, they could also be installed at an angle in order to resist the load in coaxial pull. The drastic difference between these two conditions was shown in figure 4.8. The general load-displacement characteristics of the virgin curves and the reloading curves have been previously discussed. The question arises how the great difference in load capacity (ultimate strength) between these two conditions can be explained. Part of the explanation is related to anchor depth. An anchor installed at an angle of 45° will have only 70 percent of the full depth, while the vertical anchor is installed to its full depth. It has been previously shown for coaxially loaded anchors that except for very flat angles inclined anchors have about the same load capacity as vertical anchors of equal depth. On the other hand, vertical anchors subject to an inclined pull consistently developed higher load capacities than axially pulled vertical anchors of equal depth. It is believed that the moment transmitted to the helix plate and the lateral pressure transmitted by the anchor shaft to the soil play a major part in this increased load capacity by subjecting the soil mass on the load side of the anchor to compression. The compressive load in turn will tend to increase the shear strength of the soil mass on the load side of the anchor. An examination of figure 4.7 reveals evidence of the compression of the soil mass on the load side of the anchor.

4.4 COMPARISON BETWEEN DIFFERENT ANCHOR TYPES

4.4.1 4-in Double Helix Anchors

Figure 4.14 shows a comparison between the load deflection curves for 6-in single helix and 4-in double helix anchors on the silty site. The broken lines are for 6-in single helix anchors and include coaxial tests on vertical and 45° inclined anchors (ST 13 and ST 56) and a 45° pull on a vertically installed anchor (ST 91). The tests on the 4-in double helix anchors are shown by the solid lines. Test ST 16 corresponds to test ST 13, ST 69 to ST 56 and ST 89 to ST 91. Two trends are obvious. The 6-in single helix anchors develop higher

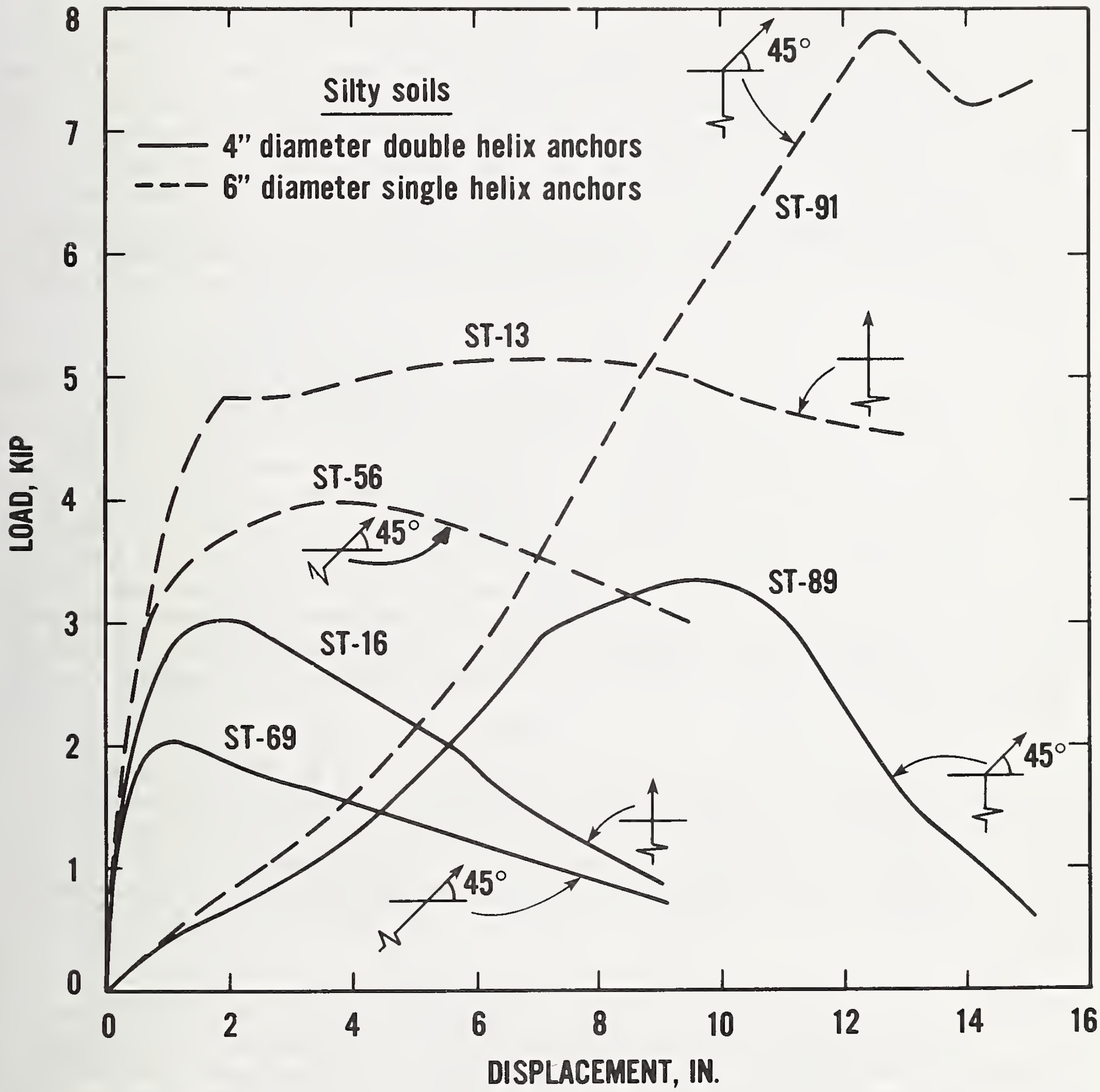

Figure 4.14. Comparison between the load-displacement characteristics of 6-in single helix and 4-in double helix anchors

load capacities and also tend to exhibit more ductility (except for test ST 91 which may have resulted in a hardware failure).

Several factors combine to produce the difference in performance: 1. The embedment depth of the helix of the 6-in single helix anchor is deeper (3.7 ft vs 2.6 ft); 2. The 6-in single helix plate has a larger area (2.25 times the area) and 3. The 4-in double helix anchor has two helixes. Many authors claim that anchor capacity is proportional to the area of the anchor plate [12]. For the tests plotted on figure 4.14 the capacity ratios are 1.73 for coaxially pulled vertical anchors, 2 for coaxially pulled anchors inclined 45° and 2.7 for vertical anchors pulled at 45°. This can be compared with the area ratio of 2.25. It should be noted that the correlation between the load capacities of the two anchor types is probably affected by all the factors mentioned above (areas of helix plate, anchor depth, and the presence of the second helix in the 4 in anchors).

Another aspect of anchor performance that can be compared are initial stiffness and ductility. Anchor ST 13 was very ductile, while its 4-in single helix counterpart, anchor ST 16 rapidly lost load capacity after the maximum load was attained. The most likely explanation for this difference is that the failure mechanism of anchor ST 13 made it a deep anchor (the slip surface did not extend to the ground surface) while anchor ST 16 acted as a shallow anchor. There was a considerable difference in stiffness between anchors ST 91 and ST 89, namely, the 6 in single helix anchor had smaller lateral displacement than the 4-in double helix anchor. This difference was consistenly observed in all the anchor tests and was not anticipated, since it was thought that the long slender shaft of the 6 in single helix anchor would provide less resistance to lateral displacement.

In figure 4.15 load-displacement curves for vertical 4-in double helix anchors pulled at various angles are compared with each other. The trend observed is similar to that shown in figure 4.9, namely, the stiffness decreases and the load capacity tends to increase as the angle of pull decreases from 90° to 15°. As previously noted for the 45° pull, the stiffness as well as the load capacity of the 4-in double helix anchors are smaller than those of the 6-in single helix anchors.

In figure 4.16 tests on coaxially loaded inclined 4-in double helix anchors, installed at various angles to the horizontal (α_1), are compared. This figure should be compared with figure 4.11 for 6-in single helix anchors. Note that in figure 4.11 the vertical anchor and the 60° inclined anchor had similar load capacities. After comparison with pullout tests at various depths this phenomenon was taken as an indication that both of these anchors acted as deep anchors, and thus their load capacity did not significantly diminish with depth. In the case of the 4 in double helix anchors figure 4.16 gives a clear indication that all the anchors acted as shallow anchors. A tabulation of the depth ratio versus the load capacity ratio is given in table 4.3, to show the average trend of the test results. The load capacity of most of the inclined anchors is roughly proportional to their depth. As in the case of the 6-in single helix

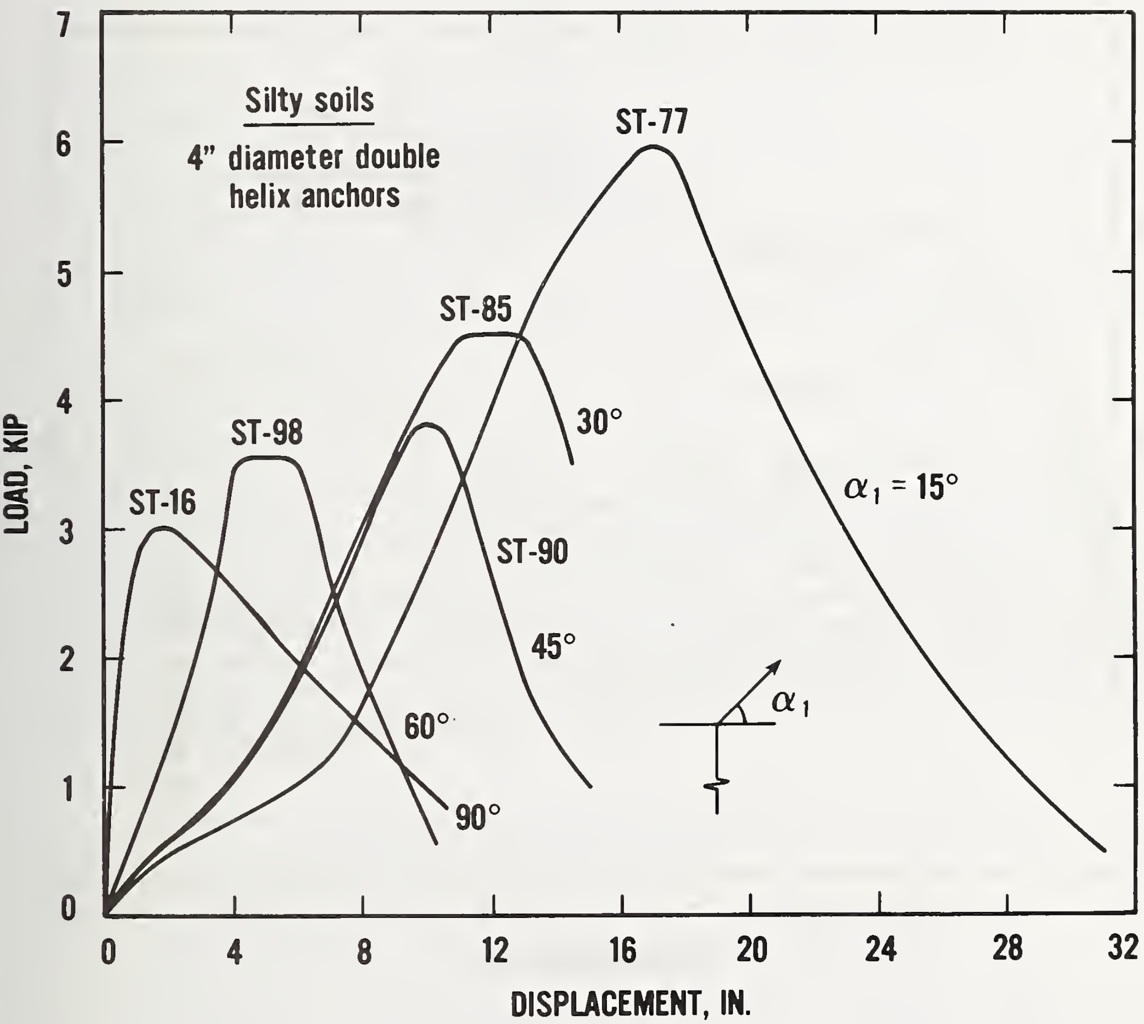

Figure 4.15. Comparison of load-displacement characteristics of vertically installed 4-in double helix anchors pulled at different angles

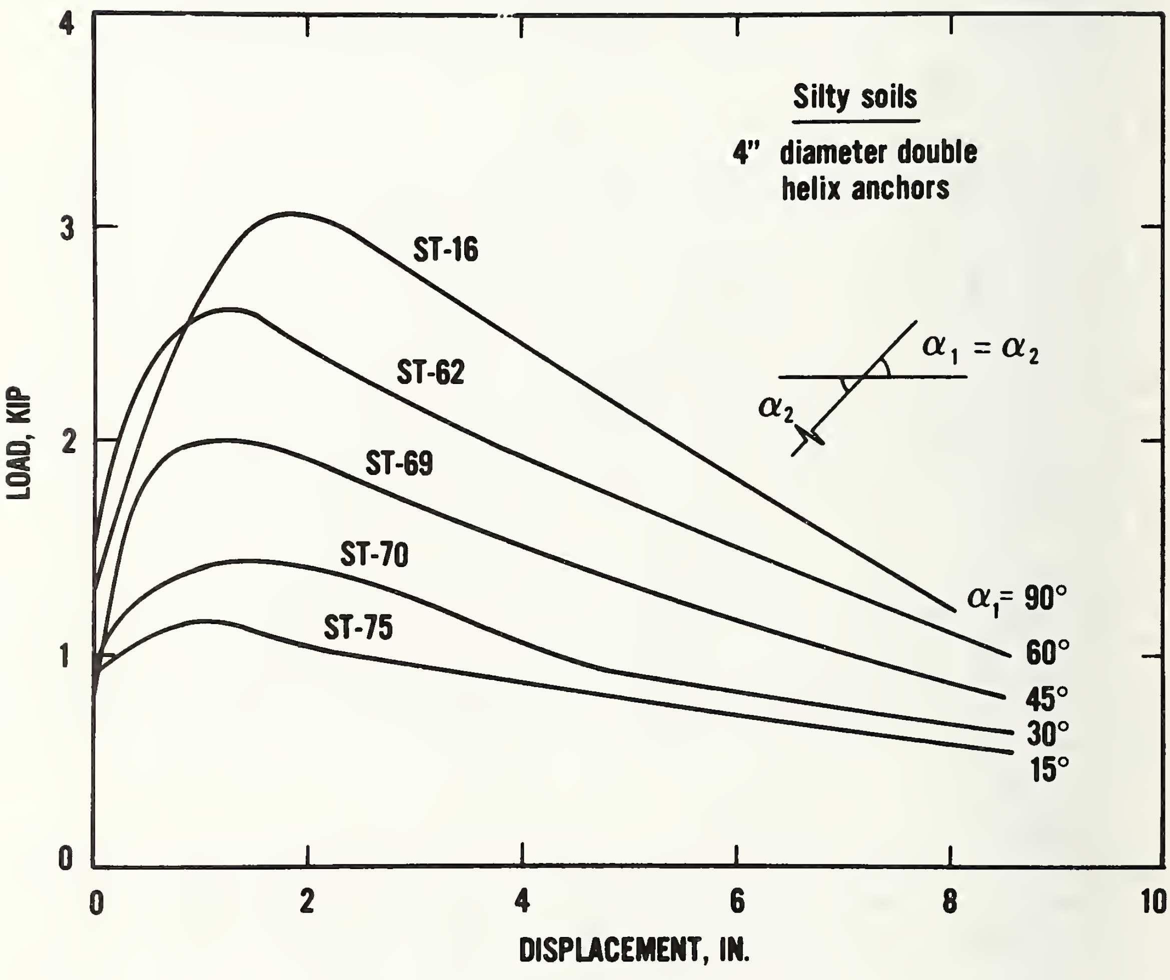

Figure 4.16. Comparison of load-displacement characteristics of coaxillay loaded 4-in double helix anchors installed at different angles

anchors the load capacity of the 4-in double helix anchors at the 15° inclination was higher than that predicted by the overall trend.

Table 4.3 Effect of Anchor Inclination on the Load Capacity of Coaxially Loaded 4-in Double Helix Anchors in Silty Soils

α_2	n	$\bar{Q}_u$, lb	v	D/Dv	$\bar{Q}u/\bar{Q}uv$
90°	6	2733	0.13	1	1
60°	3	2677	0.02	0.87	0.98
45°	3	2030	0.15	0.71	0.74
30°	3	1517	0.09	0.50	0.56
15°	3	1307	0.07	0.26	0.48

α_2 = angle of anchor shaft with horizontal
n = number of test performed
$\bar{Q}_u$ = average load capacity, lb
v = coefficient of variation
D/Dv = ratio of depth of inclined anchor to depth of vertical anchor
$\bar{Q}u/\bar{Q}uv$ = ratio of average load capacity of inclined anchors to that of vertical anchors.

4.4.2 3-inch Single Helix Anchors

The 3-inch single helix anchors were tested in order to evaluate size effects. In practice, these anchors do not have sufficient load capacity to be useable for mobile home tiedowns. This anchor is like a scaled-down 6-inch single helix anchor and should therefore afford a good comparison. The depth of the helix plate of a vertically-installed 3-inch helix anchor is approximately 25 inches, which gives an D/B ratio of 8.3. Thus, the vertical anchors probably acted like "deep" anchors. Table 4.4 gives a comparison of the load capacities of 6-inch and 3-inch anchors:

Table 4.4 Comparison of Load Capacities of 3-in and 6-in Single Helix Anchors

Anchor Size	Loading	No. Tested	$\bar{Q}_u$ lb	v
6-inch	Vertical	8	5212	0.12
3-inch	Vertical	5	1650	0.17
6-inch	45° Coaxial	3	4247	0.06
3-inch	45° Coaxial	3	1102	0.02

In accordance with the above tabulation, the ratio of load capacities between the 6-inch and the 3-inch anchor was 3.16 for the vertical anchors and 3.85 for the 45° anchors. The ratio of the helix areas is 4. Thus, the load capacity of the 3-inch anchors was higher than the capacity that would be predicted on the basis of the ratios between the helix areas. This phenomenon will be further discussed under "prediction of anchor-load capacity."

Typical 3-inch single helix anchor tests are shown in figure 4.17. Note that the vertical scale was expanded because of the small failure loads. The behavior of these anchors was rather ductile which is taken as another indication that they acted as deep anchors.

4.4.3 Self-seating Swivel Anchors

In figure 4.18, the load-displacement characteristics of the self-seating swivel anchors tested are compared with those of 6-inch single helix anchors. Because of the large displacement associated with the virgin load-displacement curves for self-seating swivel anchors, the displacment scale was compressed. Note that extremely large displacements are required to develop the load capacity of the swivel anchors. Part of this displacement is attributable to the fact that the anchor must be rotated (upset) before it develops substantial resistance. Note that for both, the vertically-pulled and diagonally-pulled swivel anchors, a displacement of about 15 inches was required to develop a 4 kip load capacity. Upon unloading, these anchors had re-loading characteristics similar to those of the helix anchors, and in the case of the 45° pull, the re-loading characteristics of the swivel anchors were superior to those of the 6-in single helix anchors. (Note that these anchors have cables, and that under coaxial pull, these cables will extend (elongate) much more than the 5/8 in or 3/4 in anchor stems of the helix anchors.)

Hereafter are some comparisons which give an indication of the effect of anchor area and direction of pull on load capacity.

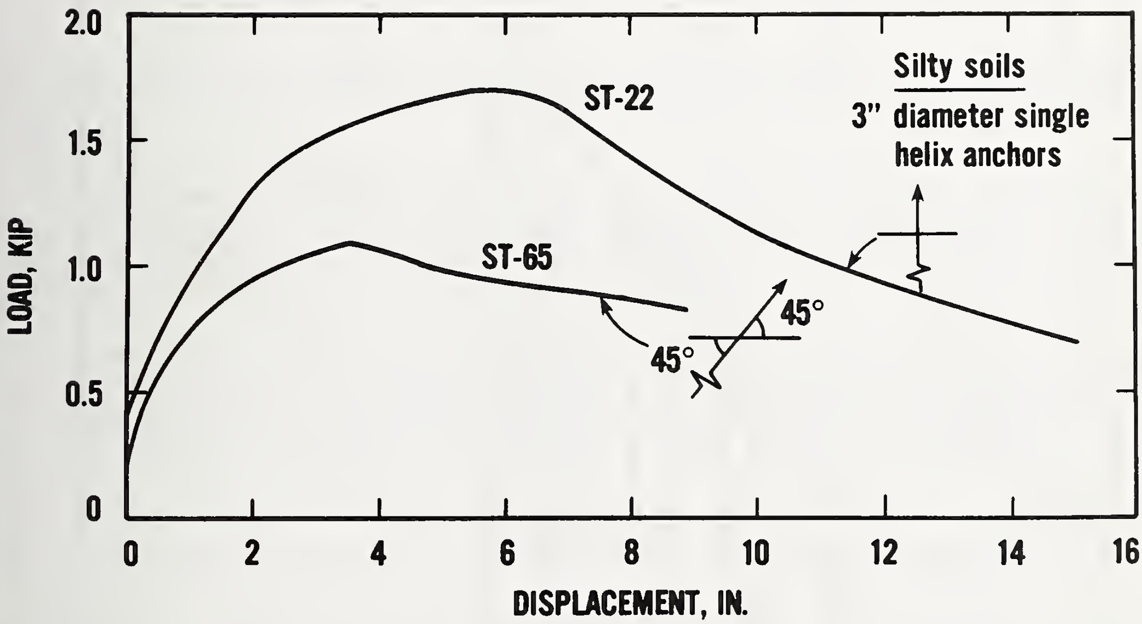

Figure 4.17 Tests of 3-in single helix anchors in silty soils

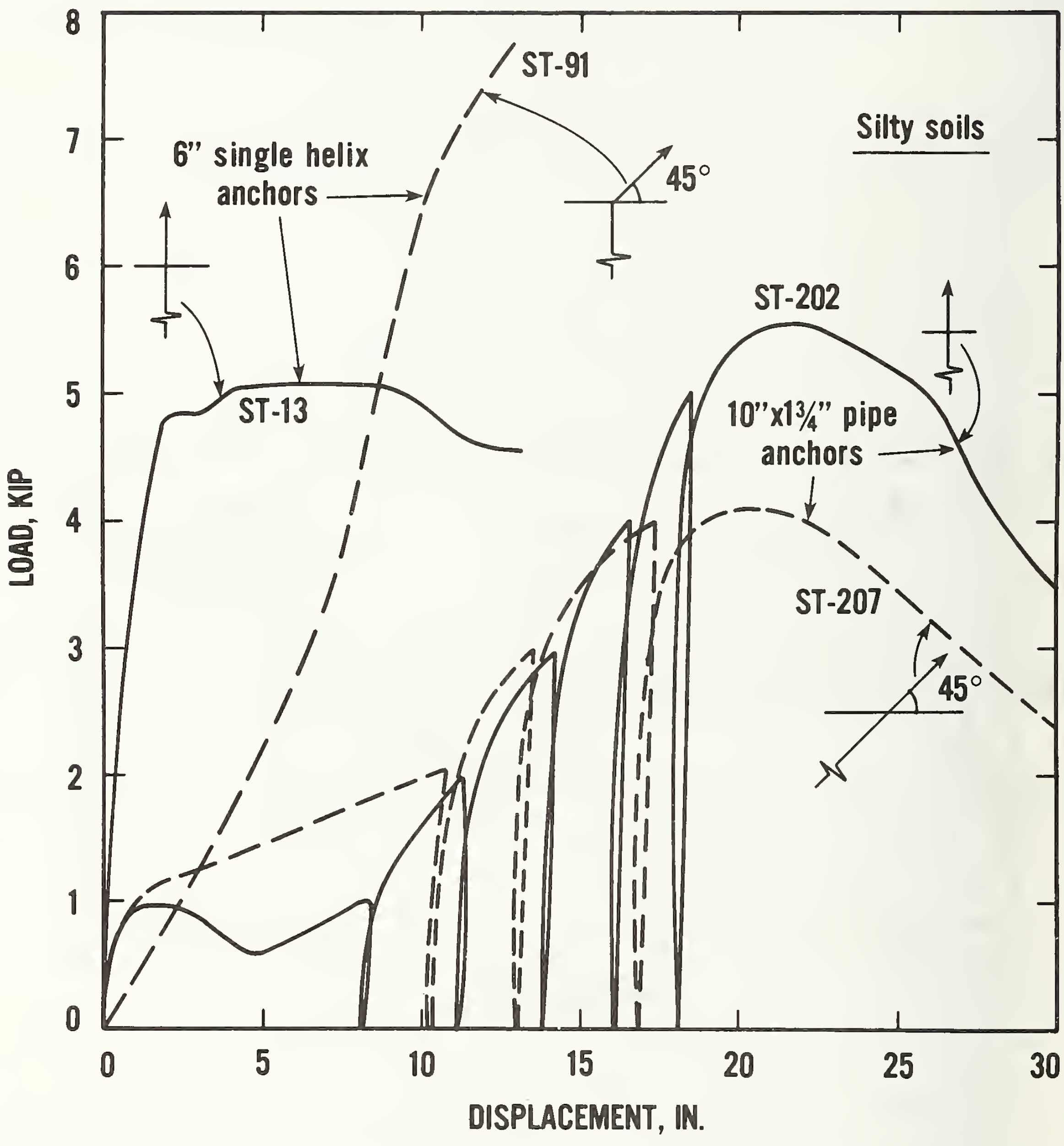

Figure 4.18. Load-displacement characteristics of self seating swivel anchors in silty soil

Table 4.5 Load Capacity of Self-Seating Swivel Anchors Tested in Silty Soils

Anchor Type	Projected Area in^2	n	α_1	$\overline{Q}_u$, lb
Pipe	17.9	2	Vert.	5,400
Pipe	17.9	3	60°	5,100
Pipe	17.9	2	45°	3,250*
Pipe	8.5	1	Vert.	3,300
Pipe	8.5	1	60°	2,800
Pipe	8.5	1	45°	2,500
Triangular	15.6	1	Vert.	5,100
Triangular	15.6	1	60°	5,300
Triangular	15.6	1	45°	5,000

* One of these anchors was not fully seated during the pull. The other anchor had a load capacity of 4,200 lb.

Table 4.5 represents very few specimens and hence there is not enough evidence to establish definitive conclusions. However, the table reveals some consistent trends:

1. Load capacity increases with the projected area of the anchor. This relationship seems to hold even when two entirely different anchor types are compared, and will be further discussed under "prediction of load capacity."

2. Vertically-pulled pipe anchors seem to have a higher load capacity than those pulled at an angle and load capacity seems to decrease when the angle of pull with the horizontal decreases. (All pipe anchors were installed vertically. An initial pull was needed to orient the cable in the direction of the pull.)

3. The triangular anchors' load capacity does not seem to change with the angle of pull.

It is also of interest to note that one of the pipe anchors pulled was not fully seated by the load.

Another important characteristic that needs to be examined is the re-loading modulus. These moduli do not seem to be affected by the angle of pull and appear to be rather consistent. They are tabulated in table 4.6. It is obvious from table 4.6 that properly seated, pre-loaded swivel anchors could provide load-displacement characteristics superior to those required in the ANSI A.119.3 standard, provided that they develop the required load capacity.

Table 4.6 Re-loading Moduli of Swivel Anchors Tested in Silty Soils

Anchor Type	Projected Area in^2	No. of Tests	R_{85} lb/in	Coefficient of Variations of R_{85}
Pipe	17.9	5	10,400	0.1
Pipe	8.1	3	6,500	0.07
Arrowhead	15.6	3	7,300	0.05

4.5 EFFECT OF SOIL CHARACTERISTICS

4.5.1 Coaxially Loaded Vertical Anchors

Figure 4.19 shows a comparison between the load-displacement characteristics of coaxial pullout tests on 6-inch single helix anchors tested on the sand, silt and clay site. Test ST13 is from the silty site (Site A), Test SD25 from the sandy site (Site B) and Test C7 from the clay site (Site C). Note that tests ST13 and C7 exhibit considerable ductility, while anchor SD25 rapidly lost its load capacity after the peak resistance was developed. The characteristics illustrated by the figure were typical for most tests on the three sites. The difference in ductility between the anchors on the sandy site, and those on the silty and clay sites is attributable to the shear-strength characteristics of these soils. The clay derives most of its strength from cohesion which does not substantially decrease with shear strain or minor decrease in depth. The results of the unconfined compression and direct shear tests for the silt indicate that this material also has substantial cohesive strength as well as frictional shearing resistance. The sand, on the other hand, derives its strength from frictional shearing resistance. To the extent that there is cohesive strength in the sand, it is derived from cementation and thus would disappear as soon as a slip surface develops. Thus the shear strength of the sand depends primarily on confining pressures which, in turn, are a function of present overburden pressure and overconsolidation (a stress history of higher vertical pressures in the past). Present overburden pressures are a function of depth and thus tend to decrease as the anchor is pulled out. Increased confining pressures due to overconsolidation are relieved as shear deformations become large, and thus will rapidly disappear as the anchor is withdrawn. There is evidence that the sand deposits on Site B are overconsolidated. The site itself was on the bottom of a borrow pit from which 20 to 30 ft. of material were removed. In addition, it has been determined that these sand deposits were overconsolidated during their geologic history. It will be shown later in this report that the magnitude of the load capacity of the anchors on Site B gives further corroborative evidence of overconsolidation.

4.5.2 Vertically Installed Anchors Pulled at an Angle

Figure 4.20 compares the load-displacement characteristics of vertical anchors installed on the three test sites and pulled at a 40° angle. Note that the

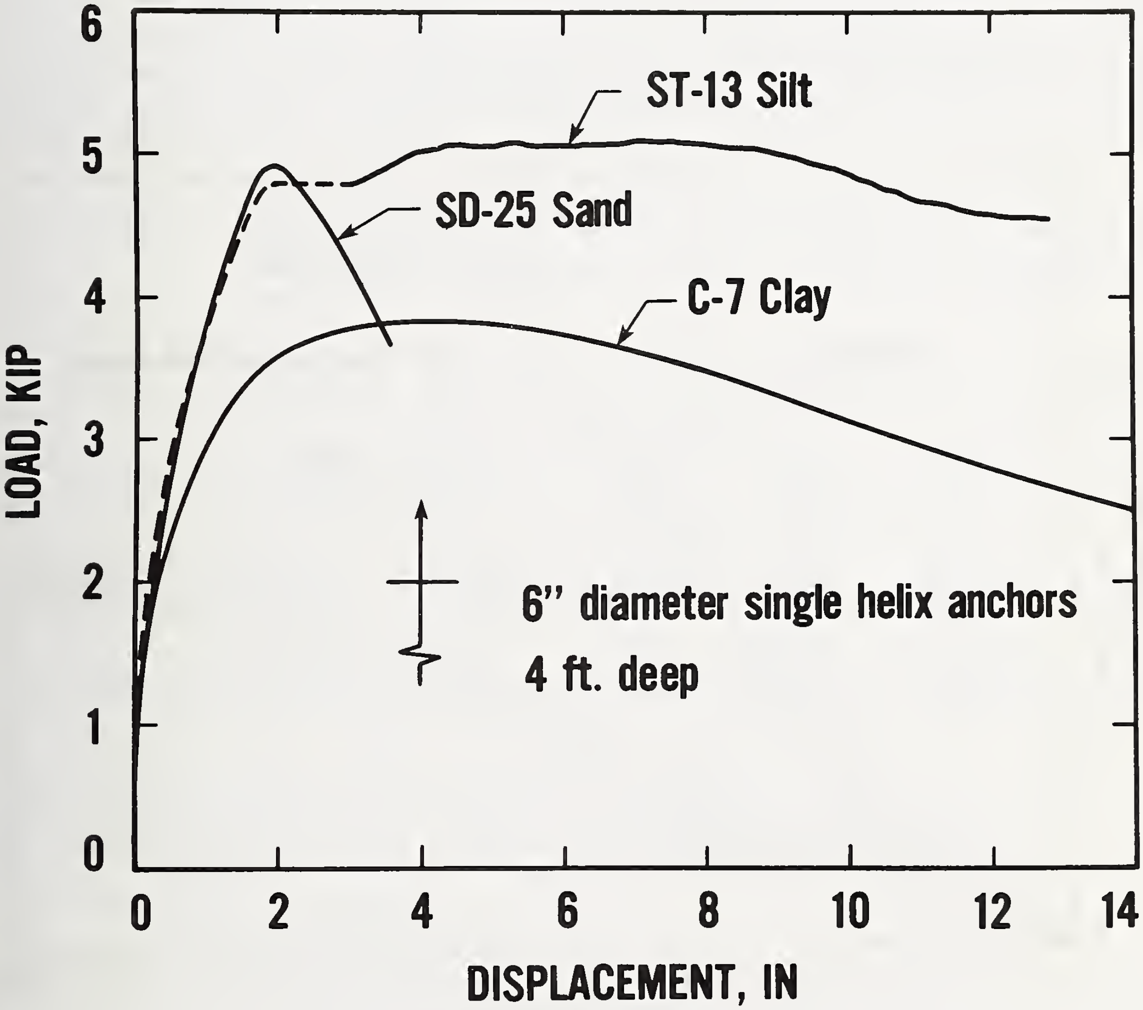

Figure 4.19 Load-displacement characteristics of coaxially loaded 6-in single helix anchors on the sand, silt and clay sites.

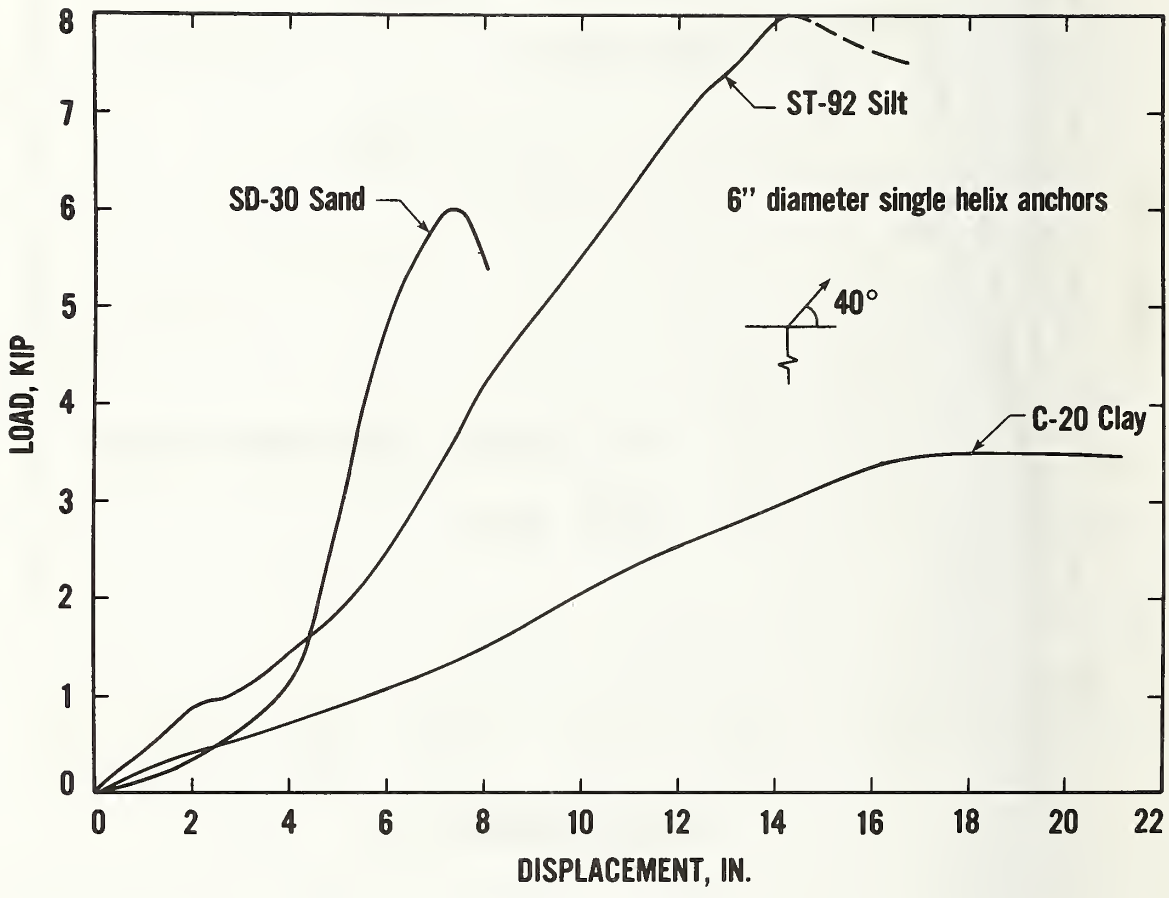

Figure 4.20 Vertically installed 6-in single helix anchors pulled at a 40° angle on the sand, silt and clay site

initial anchor stiffness on the sandy site was less than that on the silt and clay sites. However, the stiffness on the sandy site increased rapidly with increasing loads and the peak resistance was reached at a smaller displacement than that in the silt and clay site. It is noteworthy that while on the sandy and silty site the load capacity of vertical anchors pulled at an angle tended to be higher than that of axially-pulled vertical anchors, a similar increase in load capacity did not occur on the clay site. This can be explained by the fact that the compressive forces exerted in this loading mode on part of the soil mass surrounding the anchor (see section 4.3.7) substantially increased the shear strength of the sands and silts, which increases with increasing confining pressures, but not that of the clays, which entirely depends on cohesion and thus tends to be independent of confining pressures. Table 4.7 summarizes the observed trends.

Table 4.7 Comparison of the Effects of Load Inclination on the Load Capacities of Vertically Installed Anchors on Sites, A, B, and C

Soil	Anchor	$\bar{Q}_{uv}$, lb	v	$\bar{Q}_{ui}$, lb	v	$\bar{Q}_{ui}/\bar{Q}_{uv}$
	6" S.H.	5170	0.10	7930	0.02	1.53
Silt	4" D.H.	2730	0.13	3623	0.07	1.33
	6" S.H.	5290	0.08	6190	0.05	1.17 a/
Sand	4" D.H.	1610 b/	0.08	2740	0.16	1.70
	6" S.H.	3430	0.16	3270	0.21	0.95
Clay	4" D.H.	1930	0.03	2130	0.16	1.10

$\bar{Q}_{uv}$ = Average ultimate load capacity in vertical pull, lb

$\bar{Q}_{ui}$ = Average ultimate load capacity in inclined pull (45°), lb

v = Coefficient of variation

a/ Helix in the anchor broke off

b/ Tests SD2 and SD3 were excluded from average because they were influenced by a dense 1 ft. crust overlying the sand. Inclusion of those tests would inrease Q to 2,300 lb. and v to 0.41

4.5.3 Effect of Depth and Anchor Inclination of Coaxial Load Capacity

It is important to determine whether the trends which were observed on Site A also occurred on Sites B and C. The observed trends are summarized in the following tables.

In table 4.8a, Q_u is the average load capacity and v is the coefficient of variation. Whenever v is not given, the average is an average of only two tests. Even though the number of tests was limited, some trend can be recognized from table 4.8b.

In the silt, the anchors probably acted as "deep anchors" from a depth somewhere between 3 and 4 ft. Above this "critical" depth, load capacity seems to be roughly proportional to depth. On the clay site, load capacity is roughly proportional to depth, if depth is expressed as a fraction of the full (45 in.) depth.

For the sand, load capacity decreases more rapidly with decreasing depth. Thus when the depth was 50 percent of the full 45-in depth, the load capacity was only 38 percent of the full load capacity, and at 70 percent of the depth, the load capacity was 50 percent. This trend is consistent with the shear strength characteristics of the sands. When shear strength is primarily a function of cohesive strength, the strength change with depth will be more moderate than for the case where shear strength depends on confining pressures.

Except for the case of the 15° pull which was noted in the discussion of the tests on the silty site, table 4.8 gives no indication that inclined anchors have higher load capacities than vertical anchors installed to the same depth.

Effects of anchor inclination on load capacity of coaxially loaded 4-inch double helix anchors are shown in table 4.9.

The data give no indication that the trend which emerges for the silt site is also valid for the sand and clay sites. Unfortunately data for the latter two sites are not sufficient to establish any trends.

4.5.4 Reloading Moduli

The re-loading characteristics recorded are R_{85}, the secant re-loading modulus at 85 percent of the load before unloading for the static tests, and R_{10}, the secant re-loading modulus in the tenth load cycle for the cyclic tests. Table 4.10 gives the range of measured re-loading moduli for the three sites. Since the importance of re-loading characteristics was not recognized at the outset of this testing program, only limited data are available for the silty site. These data include only one static coaxial test on a helix anchor but many static tests on helix anchors which were installed vertically and pulled at an angle, for which the re-loading modulus included the rebound of the anchor shaft which generally accounts for most of the displacement. There is a possibility that the re-loading moduli on the clay site, where the soil was

Table 4.8 Effect of Anchor Depth and Inclination on the Load Capacity of Coaxially Loaded 6-in Single Helix Anchors

Table 4.8a: Summary of Load Capacity

		Silt		Sand		Clay	
Depth	α_2	Q_u, lb	v	Q_u, lb	v	Q_u, lb	v
1'	90°	840	0.19			910	
1.02'	15°	1450	0.11				
2'	90°	3200	0.03	2020		1800	—
2'	30°	2880	0.24				
2.83'	45°	4250	0.06	2650	0.05	2530	0.06
3'	90°	5220	0.08	4480	-	2980	-
3.46'	60°	4560	0.14				
4'	90°	5170	0.10	5290		3430	0.16

Table 4.8b: Load Reduction Ratios

			Silt	Sand	Clay
Depth[a/]	α_2	D/D_v[b/]	Q_u/Q_{uv}[c/]		
1'	90°	0.20	0.16		0.27
1.02'	15°	0.26	0.28		
2'	90°	0.47	0.62	0.38	0.52
2'	30°	0.50	0.56		
2.83'	45°	0.71	0.82	0.50	0.74
3'	90°	0.73	1.01	0.85	0.87
3.46'	60°	0.87	0.88		
4'	90°	1.00	1.0	1.0	1.0

a/ "Depth" is the total depth to the tip of the anchor.

b/ Depth ratio D/D_v is the actual depth to the center of the helix divided by the depth of the vertical anchor to the center of the helix.

c/ Q_u = average ultimate load of tests.

Q_{uv} = average ultimate load in vertical pull of anchors installed to their full depth.

Table 4.9 Effects of Anchor Inclination on the Load Capacity of Coaxially Loaded 4-in Double Helix Anchors

Table 4.9a. Summary of Load Capacities

		Silt		Sand		Clay	
σ_2	D/D_v	$\overline{Q}_u$, lb	v	$\overline{Q}_u$, lb	v	$\overline{Q}_u$, lb	v
90°	1	2730	0.13	1610[a/]	0.08	1930	0.03
60°	0.87	2680	0.02				
45°	0.71	2030	0.15	1813	0.24	1780	-
30°	0.5	1500	0.08				
15°	0.26	1310	0.07				

Table 4.9b. Load Reduction Ratios

		Silt	Sand	Clay
σ_2	D/D_v		Q_{uv}/Q_u	
90°	1	1	1	1
60°	0.87	0.98		
45°	0.71	0.74	1.13	0.92
30°	0.50	0.55		
15°	0.26	0.48		

a/ Tests conducted on the part of the sand site overlain by the dense crust are not included in this average.

saturated, are influenced by the buildup of porewater pressure gradients. If this was the case, the moduli may be substantically lower in very slow tests. Some general conclusions can be drawn from table 4.10.

1. In all instances, the re-loading modulus exceeded the stiffness requirements of ANSI Standard A119.3 by a substantial margin (see section 4.8).

2. The re-loading moduli in the coaxial tests tended to be much higher than those in the non-coaxial tests.

3. The re-loading moduli in the cyclic tests tended to exceed those in the static tests with the same loading conditions.

4. The re-loading moduli of the helix anchors varied over a considerable range; those of the swivel anchors tended to be quite predictable and increased with increasing anchor-plate (projected anchor area) size.

4.6 PREDICTION OF ANCHOR-LOAD CAPACITY

4.6.1 General

Three methods have been used to predict anchor-load capacity:

1. Correlation of anchor-load capacity with in situ tests [in particular, correlation with Soil Test Probe (STP) measurements are widely used by industry];

2. Determination of load capacity on the basis of measured or estimated shear strength characteristics of the soil and some analytical model which assumes a failure mechanism;

3. Determination of load capacity on the basis of pullout tests in similar soil conditions.

Method 1 is used by industry with some success. Many theoretical studies have been conducted in conjunction with method 2; however, it is diffult, and relatively expensive to determine the shear strength of the soil, and the characteristics of soils at shallow depths increase these difficulties. Method 3 is used by many states to certify anchors; however, its validity is questionable when tests performed at one site are used to predict anchor capacity at another site where the shear strength of the soil may be different.

4.10 Comparison of Reloading Moduli Measured on the Three Test Sites

Soil Type	Anchor Type	α_1	α_2	R_{85} in lb/in	R_{10} in lb/in
SILT	H-6	90°	90°	17,000	20,500 - >50,000
		60°	90°	1,700 - 2,300	
		45°	90°	2,700 - 3,600	
		30°	90°	2,100 - 5,000	
		15°	90°	3,500 - 7,800	
	D-4	90°	90°		>50,000
		60°	90°	2,000 - 3,700	
		45°	90°	3,400 - 6,200	
		30°	90°	2,000 - 2,450	
		15°	90°	3,400 - 5,200	
	AH-6[a/]	90°	90°	7,100	
		60°	90°	7,700	
		45°	90°	7,100	
	P-10[a/]	90°	90°	10,200	
		60°	90°	9,000 - 11,300	
		45°	90°	11,300	
	P-6[a/]	90°	90°	6,400	
		60°	90°	6,200	
		45°	90°	7,100	
SAND	H-6	90°	90°	10,000 - 24,000	12,400 - 20,600
		40°	45°	14,200 ->50,000	
		45°	90°	5,900 - 6,200	13,500 - 20,700
	D-4	90°	90°	11,700 - 42,000	>50,000
		45°	40°	>50,000	
		45°	90°	10,100 - 16,400	7,400 - 23,000
CLAY	H-6	90°	90°	7,600 - 13,700	9,000 - >50,000
		45°	45°	13,600 - 21,200	
		45°	90°	8,500 - 9,300	12,000 - 40,000
	D-4	90°	90°	13,500 - 42,500	20,000 - >50,000
		45°	45°	8,500 - 21,300	
		45°	90°	1,700 - 14,700	3,500 - 5,700

a3 In the swivel anchors which were installed vertically and pulled at an angle, the cable aligned itself in the direction of the pull.

4.6.2 Correlation of Anchor Capacity with In Situ Tests

(1) General

Three in-situ test methods were used to determine soil properties: the Soil Test Probe (STP); the installation torque of the anchor; and the Standard Penetration Test (SPT).

(2) Soil Test Probe (STP)

Correlations between STP readings and coaxial pullout tests on vertical 6-in. single helix anchors installed to their full depth are given in figure 4.21. The STP torque correlated with the test results is the reading taken with the tip of the instrument at 4 ft. depth, which is influenced by the shear strength of the soil between the depths of 3 and 4 ft. It should be noted that while in the silty soil the readings between 2 ft and 4 ft tip elevation did not tend to increase very much, there was a steady increase in the torque with depth in the sand.

The solid points in the figure indicate ultimate strength (Q_u) and the open points anchor load at 2 in withdrawal (P_{2v}). Load P_{2v} was plotted because it corresponds to the "load capactiy" as defined in ANSI Standard A119.3. Round points are for silt, triangles for sand and squares for clay. Note that there is a definite correlation between STP reading and pullout strength. For the tests plotted, average Q_u can be estimated by the equation

$$Q_u = 2300 + 11t \qquad \text{(eq. 4.1)}$$

where: Q_u = average pullout strength in lb.
t = STP torque at 4 ft tip penetration in in.-lb.

A reasonable lower bound for Q_u is given by:

$$Q_u \geq 1300 + 11t \qquad \text{(eq. 4.2)}$$

Tests ST26 and ST25 were not considered in deriving eqs. 4.1 and 4.2. The load-displacement characteristics of Test ST26 were different from those of other anchors, indicating that perhaps a root or some other object impeded the withdrawal. The STP reading for Test ST25 showed a sudden drastic increase in torque at the 4 ft level, while the torque at other depths was relatively low. Thus, it is reasoned that the probe hit an obstruction or bedrock. This condition would not increase anchor capacity. Note that in figure 4.21 all soil types fall into the same pattern.

A reasonable lower bound for P_{2v} is given by:

$$P_{2v} \geq 600 + 11t \qquad \text{(eq. 4.3)}$$

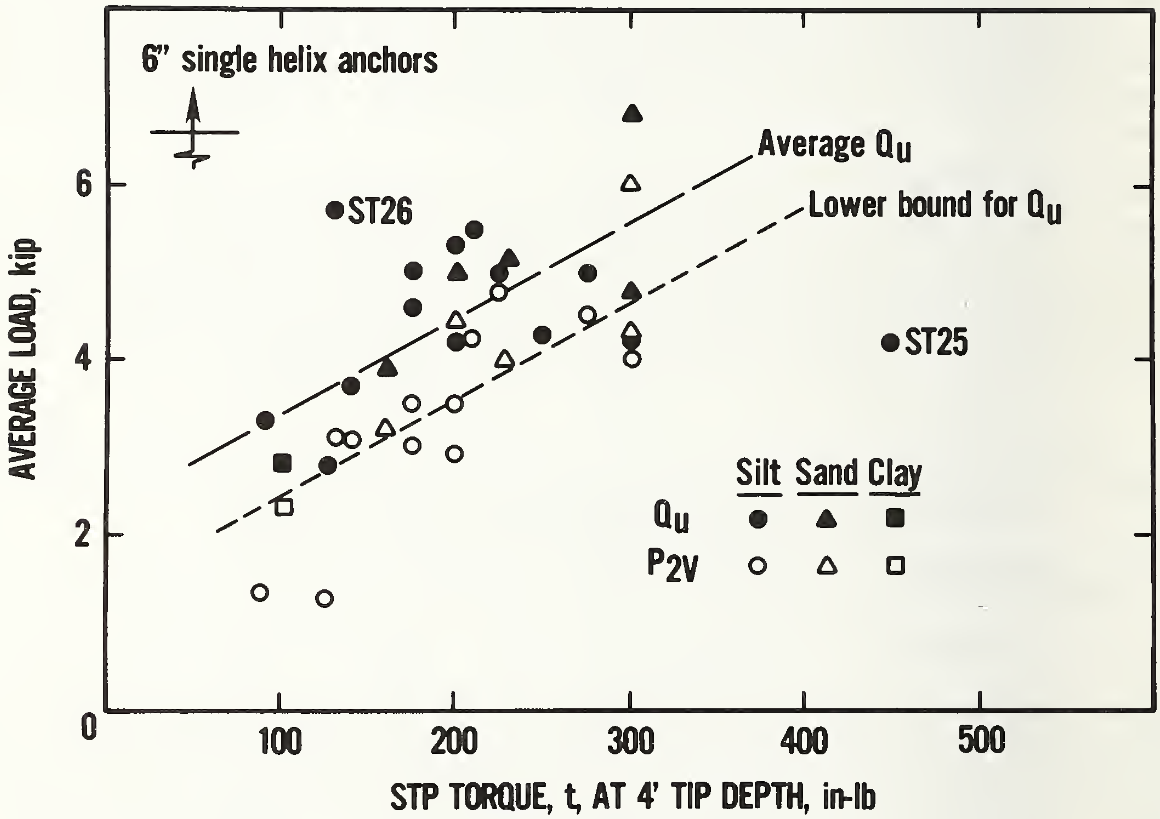

Figure 4.21 Correlation between Soil Test Probe readings and coaxial load capacity of vertically installed 6-in single helix anchors

The correlation between pullout tests of 4-in double helix anchors and corresponding STP readings is shown in figure 4.22. Note that the STP readings were low for clay and high for sand when compared with the silt readings. The explanation for the low readings in clay is that the readings were affected by pumping action resulting from excess pore water pressure buildup and perhaps sensitivity of the clay (see also discussion of shear strength prediction on page 70). The readings in sand were high, since the 4-in double helix is basically a shallow anchor and its load capacity is influenced by the shear strength of the soil between the depths of 0 and 2.5 ft (the load capacities of the 6-in single helix anchors which are deep anchors is more closely related to the shear strength of the soil near the helix). The STP readings were taken with the tip at 3 ft and the STP helix between 2 and 3 ft. This position perhaps best characterizes the shear strength close to 3 ft depth. To get a better correlation, the STP reading was averaged over the 2.5 ft depth of the lower anchor helix. This correlation is plotted in figure 4.23, and it can be seen that the sand and silt tests fall into a consistent pattern. A reasonably conservative prediction could be made by this equation:

$$Q_u \geq 13\overline{t} \qquad \text{(eq. 4.4)}$$

where: $\overline{t}$ is the torque averaged over the anchor depth

Since there is a definite correlation between the STP and anchor capacity, the question arises whether the STP can also be used to measure the shear strength of soils. This question was investigated using the "shallow" anchor tests, i.e., those anchor tests which gave evidence that the failure surface extended to the ground surface. These include anchors up to 3 ft deep (refer to figure 4.13 and table 4.8b). To calculate average shear strength, a cylindrical failure surface was assumed, extending from the helix plate to the ground surface. Even though it has been shown that the actual failure mechanism is more complex (for instance Balla [6]), the assumed surface is a possible mechanism, and the shear resistance thus computed would therefore be equal to, or smaller than, the shear strength of the soil and constitute a lower bound for the shear strength. There is field evidence that the body of soil initially displaced may have been greater than the assumed cylinder (refer to notes on soil mounds formed in tables in appendix B.) However, as can be seen from figure 4.24, that experimental evidence does not preclude the assumed cylindrical surface as the primary mechanisms.

Figure 4.25 shows a plot of average shear resistance on the assumed cylindrical failure surface against average STP readings. Tests C6, SD24 and SD47 which are for 4 ft deep anchors were included since there is evidence that, unlike in the silt, the 4 ft deep, 6-in single helix anchors in the sand and clay were on the borderline between deep and shallow anchors. A reasonable lower bound for the shear resistance, which in turn is a lower bound for the in-situ shear strength of the soil is given by the equation:

$$s = 5t \qquad \text{(eq. 4.5)}$$

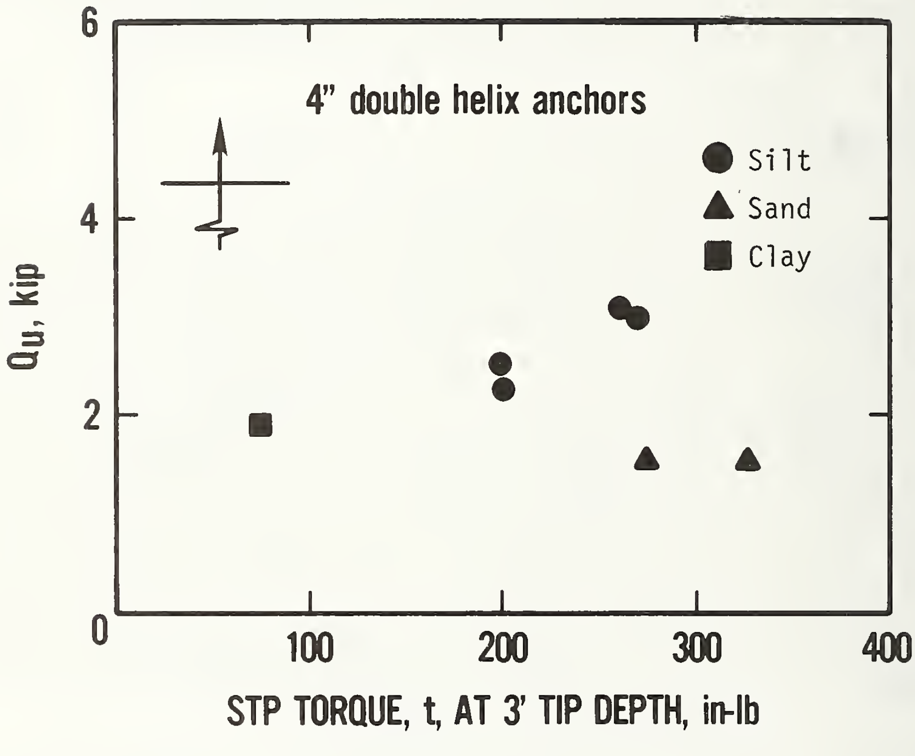

Figure 4.22 Correlation between Soil Test Probe readings and coaxial load capacity of vertically installed 4-in double helix anchors

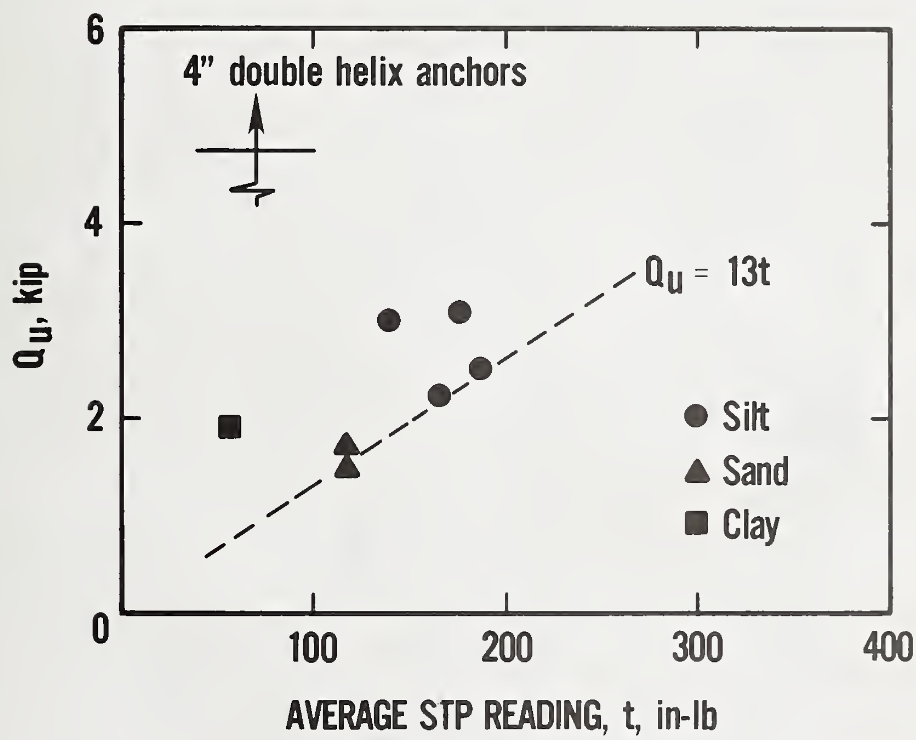

Figure 4.23 Correlation between Soil Test Probe readings averaged over a 2.5 ft depth and the load capacity of coaxially loaded 4-in double helix anchors

Figure 4.24 3 ft deep 6-in single helix anchor after pullout on the sandy site

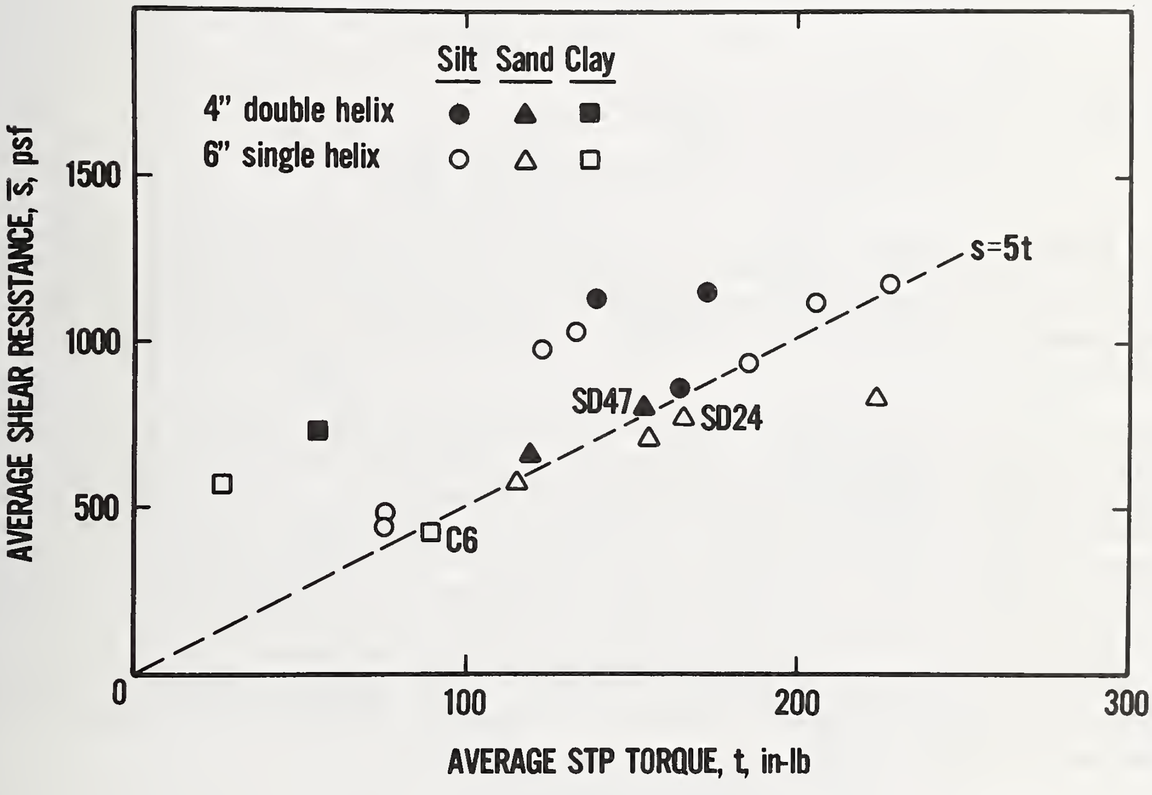

Figure 4.25 Relationship between STP measurements and the shear resistance of soil

where: s = shear strength in psf

t = STP torque in in-lb

A statistical analysis of the results gives the following values:

for 10 tests in silt:

$$\bar{s} = 6.4t; \; v = 0.20 \qquad \text{(eq. 4.6)}$$

for 6 tests in sand:

$$\bar{s} = 4.82t; \; v = 0.13 \qquad \text{(eq. 4.7)}$$

for the combined sand and silt tests (16 tests)

$$\bar{s} = 5.81t; \; v = 0.22$$

where: $\bar{s}$ = average shear strength in psf
t = STP reading in in-lb
v = coefficient of variation of shear strength

The results for the clay site are inconsistent, with s/t ranging from 4.88 to 22.67. The extremely low torque readings on 2 out of the 3 tests on the clay site are attributed to porewater pressure buildup and resulting pumping action, and perhaps sensitivity of the clay. Thus the STP may not be a good tool in saturated clays.

The s/t ratio for the sand tended to be lower than that for the silt. A possible explanation of this phenomenon is the fact that in the silt torque did not change much between the depths of 1 and 3 ft. In the sand the torque steadily increased with depth. The shear strength of the soil affects the STP torque in two ways: by resistance at the lower tip of the STP; and by skin friction exerted on the helix of the instrument. If the measurement is primarily affected by tip resistance, then the shear strength measured when the tip of the STP is at 4 ft may be characteristic for the depths from 3.5 to 4.5 ft, rather than for the depths from 3 to 4 ft as was assumed herein. This would result in a lower s/t ratio if the soil strength increases with depth and the average is calculated by the method used herein.

Further studies will be required to refine the use of the STP for the in situ measurement of the shear strength of soils.

(3) <u>Installation Torque, T</u>

The correlation between installation torque and anchor strength is shown in figures 4.26 and 4.27, for 6-in single helix and 4-in double helix anchors, respectively. For the 6-in single helix anchors the scatter is considerable

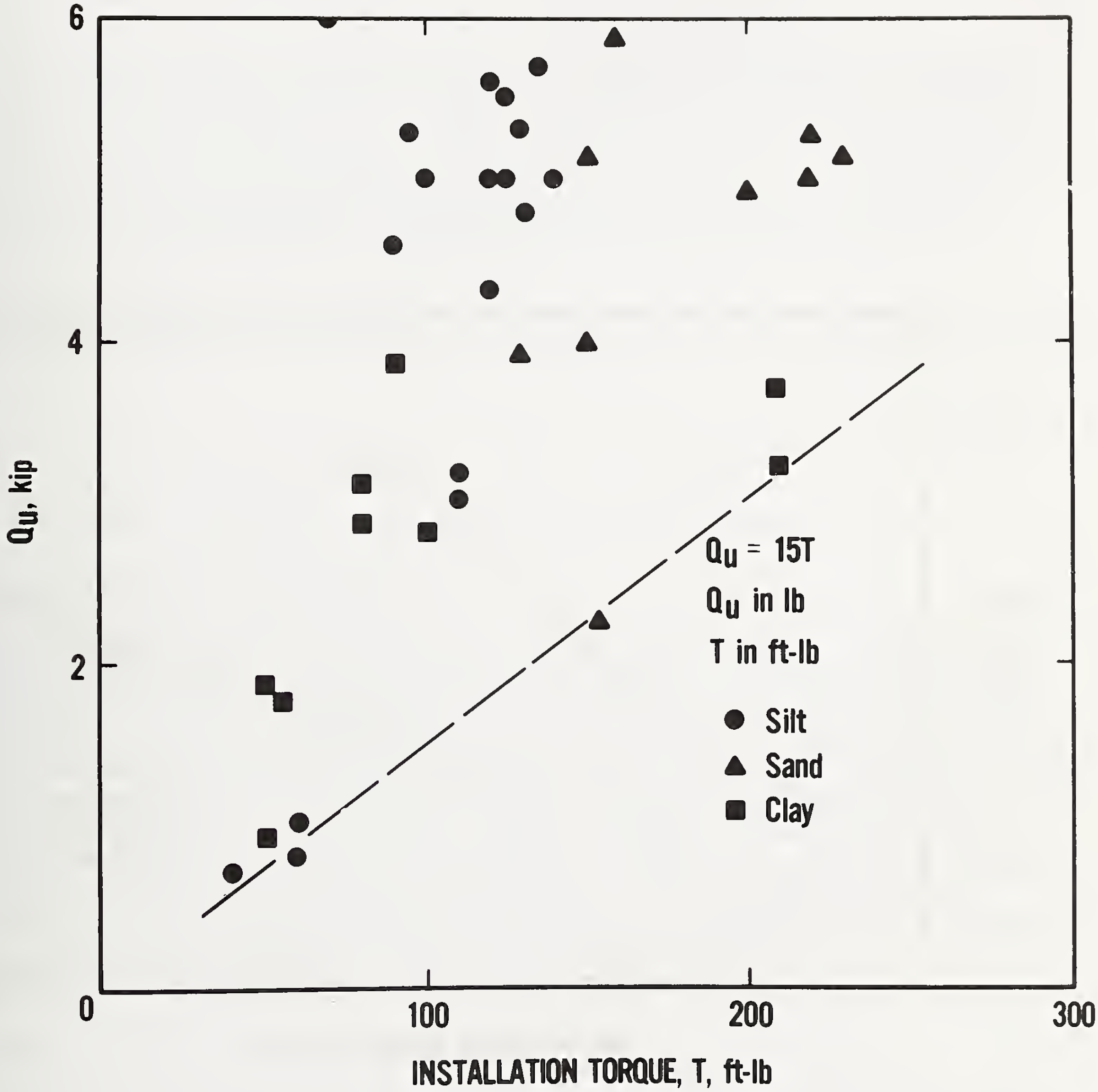

Figure 4.26 Relationship between installation torque and pullout strength for vertical, coaxially loaded 6-in single helix anchors

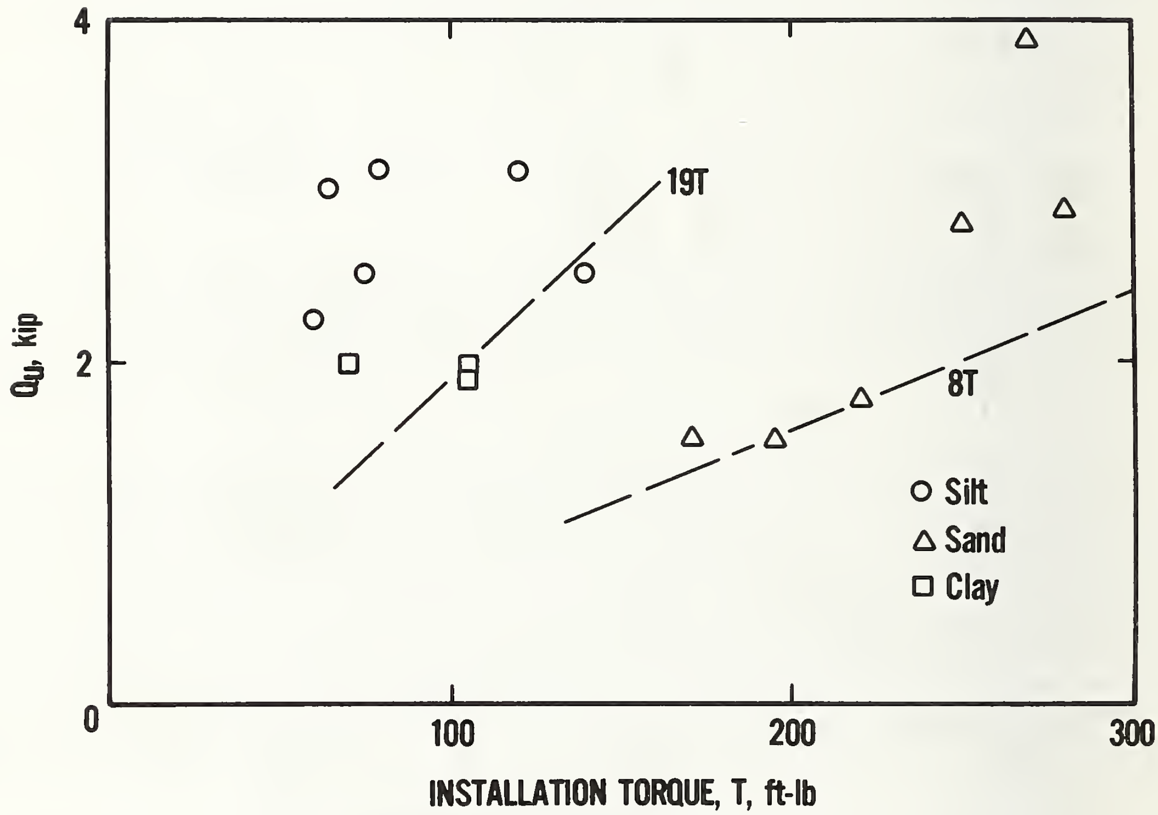

Figure 4.27 Relationship between installation torque and pullout strength for vertical, coaxially loaded 4-in double helix anchors

and there is no observed trend related to soil type. A reasonable lower bound is given by the equation

$$Q_u \geq 15T \qquad \text{(eq. 4.9)}$$

where: T = installation torque measured at maximum anchor penetration in ft-lb.

For the 4-in double helix there is a distinct difference between sand on one side, and silt and clay on the other side. A similar phenomenon was observed for the Soil Test Probe (figure 4.22) where the difference was eliminated when torque readings were averaged over the depth of the anchor. In the case of installation torque such a procedure would not be practical. Thus installation torque may be misleading as a strength measure for shallow anchors in soils in which shear strength increases rapidly with depth.

Equations for the lower bounds of T vs. Q_u are shown in figure 4.27. It should be noted that manufacturers recommend the equation:

$$Q_u \leq 10T \qquad \text{(eq. 10)}$$

where Q_u is in lb and T in ft-lb

which is considered conservative for the data presented herein.

(4) Standard Penetration Test (SPT)

The Standard Penetration Test is generally considered to correlate well with the shear strength of granular soils. However, in this instance, the exploration is shallow and drill stem lengths are therefore very short. It has been shown [15] that for drill stem lengths less than 10 ft. the energy delivered to the split spoon is extremely sensitive to the drill stem length. Thus, for this shallow exploration, one should expect erratic results from the SPT. The quantity of tests taken in this project does not permit a comparison of SPT counts with the strength of individual anchors. However, a comparison between STP readings and SPT blowcounts was made and is shown in figure 4.28. The scatter in the figure is considerable and no useful correlation can be derived.

4.6.3 Theoretical Determination of Anchor-Load Capacity

(1) General

Several hypotheses have been advanced which correlate anchor-load capacity with the in-situ shear strength and unit weight of the soil. All those hypotheses distinguish between "deep" and "shallow" anchors. In deep anchors the failure (slip) surface does not extend to the ground surface. In general, the ratio of depth below the surface to anchor-plate width (D/B) is used to determined whether an anchor is deep or shallow. The anchors tested in this project have D/B ratios at full penetration depth which puts them close to the dividing point between deep and shallow anchors. This somewhat complicates data interpretation.

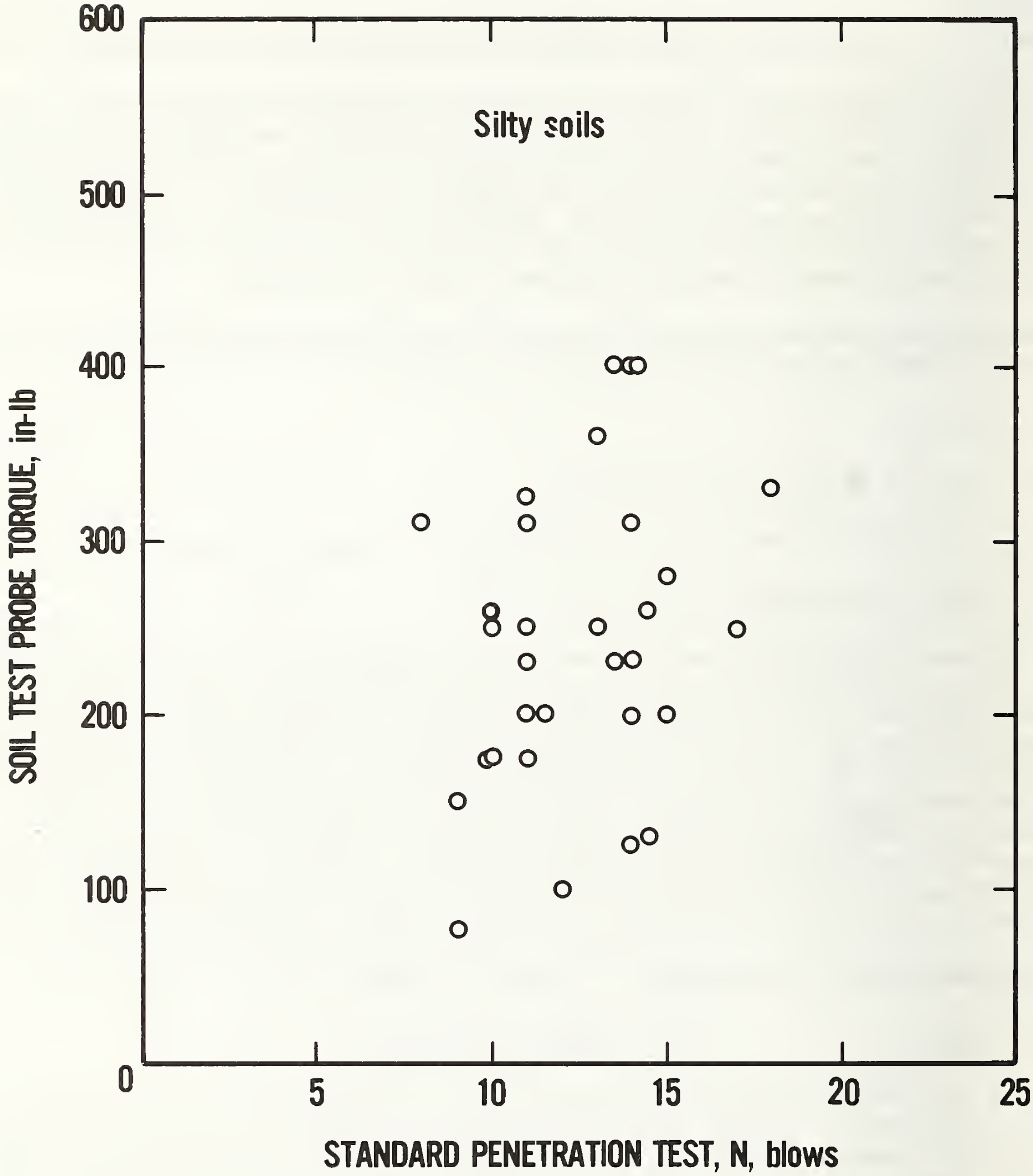

Figure 4.28 Correlation between SPT blowcount and STP torque readings for the silt site

Since all anchors are 4 ft or less, their strength is determined by the shear strength and unit weight of the soil between 0 and 4 ft depth. As already noted, soil shear strength in this depth range varies rapidly with depth and is difficult to measure. This further complicates the problem of comparing the test results with theoretical models.

(2) Comparison of Test Results with Uplift Capacity Equations

(a) Cohesive Soils

All the full-depth anchors will be considered in this section, even though it appears that all the 4-in double helix anchors (because of the upper helix) acted like shallow anchors, and the 6-in single helix anchors on the clay site may have been on the borderline between deep and shallow anchors.

The following equation was proposed to calculate the load capacity of anchors in cohesive soils [11]:

$$Q_u = N_u c A + S \qquad \text{(eq. 4.11)}$$

where A = projected anchor plate area
c = cohesive strength of soil
N_u = an uplift capacity factor
S = resistance of anchor shaft

S is assumed to be very small and therefore can be neglected. Strictly speaking, only the clay on Site C would act like a cohesive soil. The silt derives only part of its shear strength from cohesion (or apparent cohesion). The other part would be attributed to frictional resistance. However, due to the fact that the deepest anchors are only 4 ft deep the confining pressures and thus the frictional resistance should be small. This is further corroborated by the characteristics of the depth vs. shear strength profile evident from the STP readings and by the great ductility of the anchors tested in silt (confining pressures caused by overconsolidation would be relieved as the anchor is pulled out.)

The value of N_u is generally assumed to increase with depth until the anchor is a deep anchor and then to remain essentially constant. Typical values proposed for N_u are summarized by Davie and Sutherland [8, figure 7]. These values range from 5 to 10 and tend to become constant for D/B ratios greater than 6 (3 ft depth for a 6-in single helix anchor).

Implicit in equation 4.11 with a constant value of N_u is the assumption that for D/B ratios greater than 6 the anchor capacity should be essentially proportional to the area of the anchor plate (barring some shape factors related different plate geometries). In figure 4.29, the average load capactiy per unit area of anchor plate in psi is plotted against the size of the anchor plate in in^2 for the coaxial tests on full-depth vertical anchors and the tests on the self seating swivel anchors for the silt and the clay sites. These values

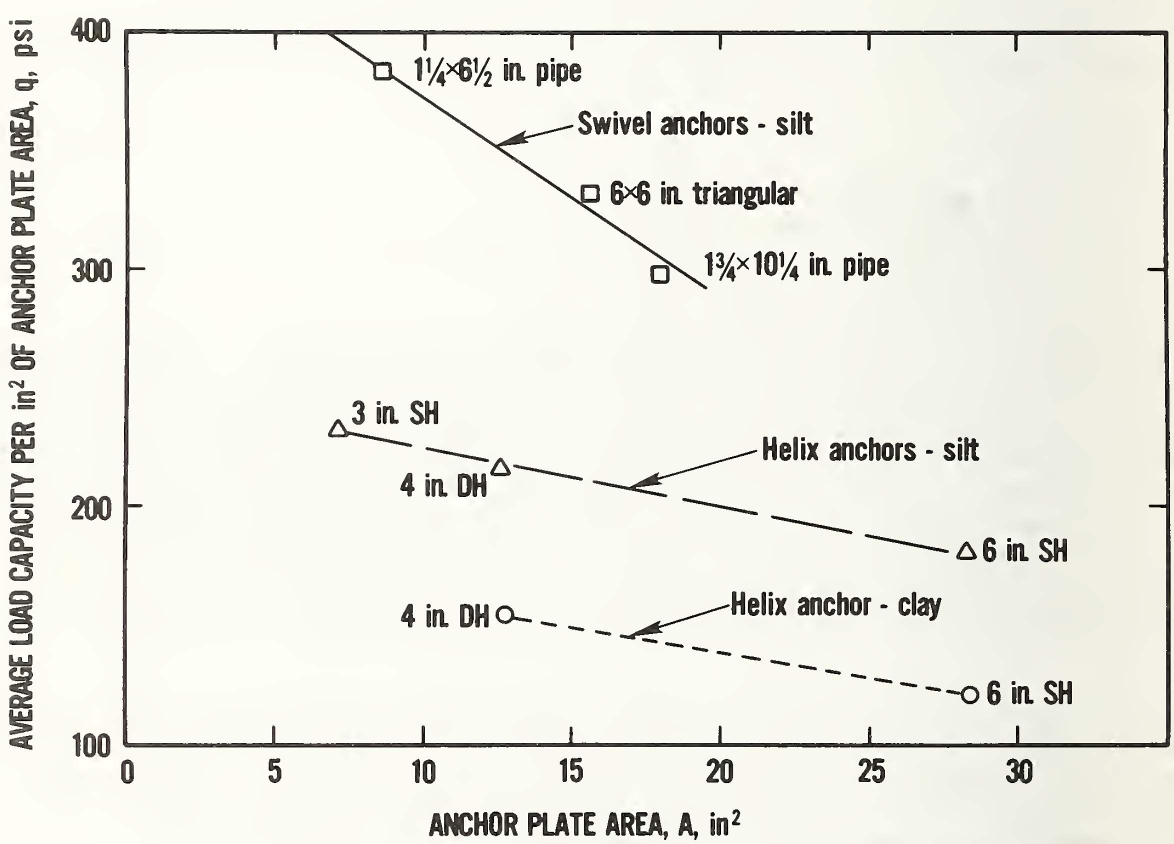

Figure 4.29 Effect of anchor plate size on q in the silt and clay sites

should be constant in accordance with accepted hypotheses. However, as can be seen from the figure, there is a consistent trend for q to increase with decreasing anchor-plate size. The trend is definitely not attributable to changes in the soil profile. The swivel anchors were all installed at the same depth of approximately 50 inches and had D/B ratios of 8 or more. The smaller helix anchors were shallower than the larger ones. If anything, this should produce the opposite effect, since soil strength tended to increase with depth. The fact that the 4-in double helix anchors acted like shallow anchors should also produce the opposite effect.

There is at this time no satisfactory explanation for the trend observed in figure 4.29. Similar trends have been observed by Tsangarides [18], pg. 186, for anchors in sand. However, in that case, plate diameters were 2 in or smaller. Another interesting trend that can be derived from figure 4.29 is that the value of N_u for the swivel anchors is greater than that for the helix anchors, and that the size effects for the swivel anchors are more pronounced.

Even though the shear strength of the soil changed with depth as well as location, it is of interest to try to determine N_u values for the anchors in the silt and clay sites.

On the basis of the laboratory tests, the c value of the clay on Site C is between 700 (U-U triaxial test) and 965 psf (unconfined compression test). Values calculated from the pullout tests of shallow anchors, using the simplified cylindrical surface are 570 psf for the 1 and 2 ft depths (6 in anchors), 720 psf for a 2.5 ft depth (4 in anchors), and 620 psf for a 3 ft depth (6 in anchors). These values are reasonably consistent with each other.

Based on 700 psf shear strength in the 3 to 4 ft depth range, the following N_u values are calculated:

Anchor C-6 Q_u = 2800 lb., N_u = 20
Anchor C-7 Q_u = 3800 lb., N_u = 28
Anchor C-8 Q_u = 3650 lb., N_u 27

These values of N_u, as well as the trend for N_u to increase with decreasing anchor plate area are not consistent with accepted anchor capacity hypotheses. There are two factors which may have increased anchor capacity: suction effects (negative porewater pressures) associated with the large pullout displacements and which did not dissipate during the test because of the low permeability of the clay (such effects have been observed by others [1]); and root systems in the soil.

For the silt site, the laboratory test results are not as consistent as those for the clay site. The unconfined compressive strength was 4000 psf, which would indicate a shear strength of 2000 psf. Shear strengths obtained from direct shear tests ranged from 400 to 800 psf. Lower bound shear strengths calculated from the pullout tests, using a cylindrical failure surface ranged from 430 to 1200 psf. Thus, it may be misleading to use any one value. Therefore, N_u is calculated in two ways:

1. Using 2000 psf on the basis of the unconfined compression test; and

2. Using a value of s = 6.4t for tests where t was measured.

The following results are obtained:

Using test probe readings:

> The average value of N_u for the 6-in single helix anchors is 19 with a coefficient of variation of 0.2.

Using s = 2000 psf:

> The average value of N_u for the 6-in single helix anchors is 12.6 with a coefficient of variation of 0.09. The average value of N_u for the 3-in single helix anchors is 16.8 with a coefficient of variation of 0.17 and the N_u values for the swivel anchors are 27.9 for the 6 1/2-in pipe, 23.6 for the 6 in arrowhead and 20.5 for the 10-in pipe.

If a cylindrical failure surface is assumed to be the failure mechanism it can be shown that

$$N_u = 4D/B^* \qquad \text{(eq. 4.12)}$$

where D/B* is the D/B ratio at which the failure surface ceases to extend to the ground surface.

It has been previously shown that for the 6-in single helix anchors on the silt site the critical depth[3/] where the anchors cease to be shallow anchors is between 2.83 ft and 3 ft (see figure 4.13) thus D/B* is somewhat smaller than 5.5, and N_u, calculated by eq. 4.12 would be somewhat less than 22.

Note that the N_u values for the silt site are not inconsistent with those obtained for the clay site. However, as in the case of the clay site, they are not consistent with hypotheses and data presented by others [8]. It should be noted, however, that in accordance with available data from engineering studies in the area the silt may have an angle of shearing resistance of as much as 30°, and thus the pullout capacity is not adequately predicted by eq. 4.11.

3/ Actually the concept of a clear demarcation between "deep" and "shallow" anchors has been questioned. Davie and Sutherland [8] distinguished three zones of D/B ratios: shallow - $0 < D/B \leq 2$; intermediate: $2 < D/B \leq 4.5$; deep: $D/B > 4.5$.

(b) Granular Soils

Anchor capacity on the sandy site should be compared with the pullout capacity equation proposed for sands [11]:

$$Q_u = \gamma D N_{qu} A$$

where γ = in-situ unit weight of soil
D = depth of anchor plate below surface
N_{qu} = uplift capacity factor for granular material which is a function of the angle of sheaing resistance (ϕ) and the D/B ratio.

The only "deep anchors" tested in sand were the 6-in single helix anchors. Thus size effect cannot be effectively explored. There is evidence [3] that N_{qu} increases with depth at least to a D/B ratio of 14. Thus, there is no sharp dividing line between "shallow" and "deep" anchors.

The N_{qu} values calculated on the basis of the test results are given in table 4.11 (tests conducted in the area overlain by the hard crust were not considered):

Table 4.11 Uplift Capacity Factors for Full-depth Anchors on the Sandy Site

Anchor Type	D/B	Number of Tests	Range of N_{qu}	(Average)	Coefficient of Variation of N_{qu}
H-6	7.5	5	74-84	75.6	0.07
D-4	7.5	3	71-84	77.5	0.08

The values in table 4.10 are quite consistent and the scatter is not very great. The N_{qu} values are high compared with other available data [3] (A ϕ value of 31° was used for the comparison). However, there is considerable scatter in the available data. The relatively high load capacity on the site is attributed to overconsolidation which increases the shear strength by increasing confining pressures (there was approximately 20 ft overburden which was recently removed). The rapid loss of load capacity as anchors are pulled out is also attributed to overconsolidation.

N_{qu} ratios were also calculated for the shallow anchor tests and are given in table 4.12. The values in table 4.11 can be compared with those for the full-depth anchors. All the results are for 6-in single helix anchors on the sandy site.

Table 4.12 Uplift Capacity Factors for 6-in Single Helix Anchors Installed to Less Than Their Full Depth in the Sandy Site

Anchor Depth	D/B	No. of tests	Range of N_{qu}	Average N_{qu}
2 ft	4	2	48-61	54
3 ft	6	2	72-88	80
3.75 ft	7.5	5	74-84	76

Unfortunately, there are not enough tests to determine whether the size effects observed on the silt and clay sites also occur in sands. However, the consistency of the N_{qu} values when comparing the full-depth 6-in single helix and 4-in double helix anchors indicates that there were probably no size effects for the anchors tested.

4.6.4 Determination of Load Capacity on the Basis of Pullout Tests in Similar Conditions

The tests presented herein were performed on reasonably uniform sites. Nevertheless, there were considerable variations in pullout strength on any one site. Much greater variations should be expected if an anchor is certified generically for some soil condition occurring over a larger region. The full-depth vertical coaxial pullout test results are summarized in table 4.13 below for the three sites to give an overview of the variability of test results encountered. All the numbers are for Q_u in lb.

Table 4.13 Range, Mean, and Coefficient of Variation of the Load Capacities of the Full-Depth Anchors

Site	Anchor Type	No. of Tests	Range, lb.	Mean, lb.	Coefficient of Variation
Silt	H-6	18	2800-6000	4740	0.18
	D-4	12	1900-3200	2700	0.18
Sand	H-6	10	2750-6825	5100	0.23
	D-4	6	1530-3890	2390	0.40
Clay	H-6	3	2800-3850	3430	
	D-4	3	1900-2000	1930	

Table 4.13 was compiled without regard to special local conditions such as the stiff crust covering part of the sand site and submerged areas, since such conditions should be expected to occur in practice. It can be seen that in most instances, even for one site which was considered uniform, there is considerable

strength variation. The effects of the strength variation were encountered during the cyclic tests on the clay site which was considered uniform. Cyclic load levels were set in advance at what was thought to be 75 percent of the load capacity as derived from adjacent static tests. However, in many instances the anchors failed before these load levels were reached. Typically, the coefficient of variation for various test results tended to be about 0.2. It can be seen that it increased to as much as 0.4 when local variations within the site are disregarded.

4.6.5 Effect of Submerged Conditions

The clay site was saturated, and therefore submerged conditions would not have had much effect on load capacity. On the other two sites, effects of submergence were explored. On the silt site, this was done in an area which was permanently under water. On the sandy site an area was temporarily submerged during some of the anchor tests. Results for the silty site are summarized in table 4.14.

Table 4.14 Comparison Between Regular and Submerged Anchor Tests on the Silty Site

Anchor Type	Condition	No. of Tests	Range of Q_u, lb	Average Q_u, lb	v
H-6	Unsubmerged	12	4200-6000	5090	0.11
H-6	Sumberged	6	2800-5700	3980	0.25
D-4	Unsubmerged	6	2250-3100	2730	0.13
D-4	Sumberged	6	1900-3200	2660	0.29
H-3	Unsubmerged	5	1300-2050	1650	0.17
H-3	Submerged	5	1000-2625	1895	0.36

It can be seen from the above summary that only the average strength of the 6-in single helix anchors was reduced by submergence. For all three anchor types, however, there was greater variation in the submerged test results and some individual submerged tests showed substantially reduced strength. It is suspected that the shear strength of the soil was actually reduced by submergence, but that some individual anchors had increased resistance because of the presence of some boulders in this area, and also possibly because of suction effects resisting the pullout.

The submergence tests in the sand did not result in strength reduction because it was impossible to submerge a large enough area to eliminate seepage forces (piezometric heads at anchor plate elevation extended only 0.5 ft above the anchor plate).

It is assumed that submergence should substantially reduce the resistance of anchors in granular soil, but that it does not necessarily affect the cohesive strength. However, no consistent trend emerges from the test data presented herein.

4.7 CYCLIC TESTS

4.7.1 Cyclic Tests on the Silt Site

Typical test results are shown in figure 4.30. Specimen ST111 was loaded to what was estimated to be 0.75 Q_u. Note that most of the displacement took place in the first load cycle and no further residual displacement occurred after 100 load cycles. This phenomenon is the result of gradual compaction causing the displacement to be entirely elastic after 100 load cycles. Similar load-displacement curves resulted at 50 and 25 percent of the estimated pullout load (tests ST116 and 118). Specimen ST123 was preloaded to 0.84 Q_u and subsequent load cycles were applied at 0.67 Q_u. Note that Q_u was overestimated for his specimen, resulting in a preload which was close or equal to the ultimate load (the pullout load after cyclic loading, Q_u, was less than the preload). Nevertheless, the total displacement after 200 cycles was only 1.2 in. However, unlike in the other tests plotted, the preloaded specimen had small residual displacements for each load cycle up to 200 cycles. The tests on the silt site also included two tests on preloaded 4-inch double helix anchors. These tests had no further residual displacements after 100 load cycles.

It is of interest to consider whether the load capacity of the anchors was diminished as a result of the cyclic loading. The five 6-inch single helix anchors which were subjected to high cyclic load (0.75 Q_u vs. 0.67 Q_u) had an averged pullout strength Q_u = 5240 lb. with a coefficient of variation v = 0.09. This compares with an average pullout strength of 5090 lb and v = .11 for the anchors which were not subjected to cyclic load. Thus loading of up to 300 cycles of 0.75 Q_u apparently had no significant effect on the pullout strength of the anchors. Indeed some of the anchors were subjected to cyclic loads as high as 0.9 Q_u, since the actual pullout strength was not known when the cyclic load was applied (specimen ST123 was probably preloaded to ultimate and thereby weakened).

The total cumulative displacement of the preloaded anchors was well within limits acceptable in present standards (see section 4.8).

4.7.2 Cyclic Tests on the Sandy Site

Typical test results from the sandy site are shown in figure 4.31. Tests SD34 and SD35 are unpreloaded and preloaded axial tests, and tests SD38 and SD40 are

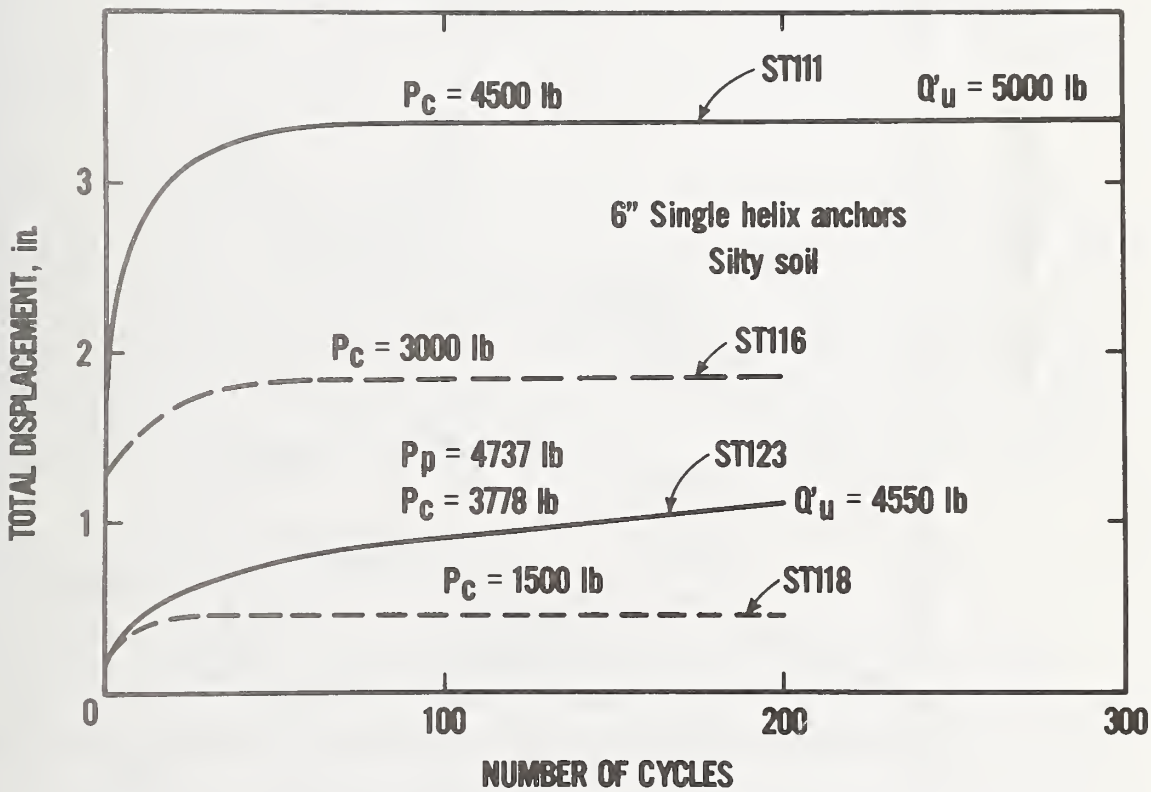

Figure 4.30 Cyclic load tests in silty soils

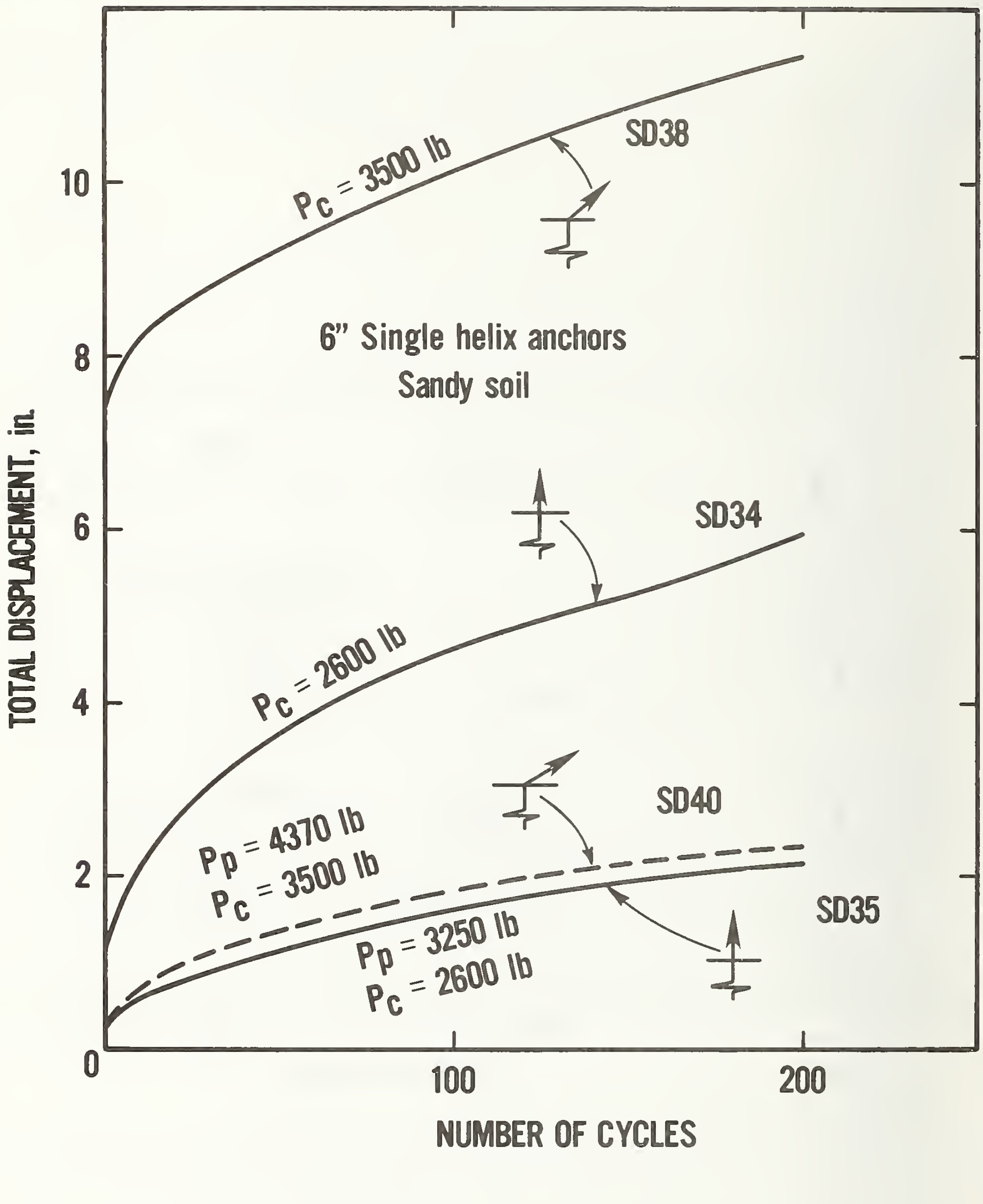

Figure 4.31 Cyclic load tests in sandy soils

unpreloaded and preloaded tests on vertical anchors pulled at 40° to the horizontal. Note that on the sandy site there was also some compaction effect, but there were small residual displacements in each load cycle up to the 200 load cycles applied in the test. The total cumulative displacement of the preloaded specimens after 200 load cycles of 2/3 the ultimate load was approximately 2 in. If the assumption is made that 2/3 of ultimate would be the maximum design load that can be reasonably permitted, and when the effect of these 200 cycles is compared with the hurricane history described in section 3.3.2(4), and the design load is compared with the "maximum" wind load, it is conservatively estimated that a similar hurricane would have resulted in a cumulative anchor head displacement for preloaded anchors of not more than 1 inch (see also section 4.7.3).

The effect of cyclic loading on anchor-load capacity is somewhat difficult to assess from the test data. For the vertical tests, four 6-inch single helix anchors had an average failure load Q_u of 4675 lb. with v = 0.07. This compares with Q_u = 5290 with v = 0.08 for the sand site if submerged tests are excluded. For the 4-inch double helix anchors, the average Q_u was 2325 lb. This compares with 2390 lb. with v = 0.4 for all the tests in sand, but only 1610 lb. with v = 0.08 if tests if the area of the dense crust are excluded. Thus no conclusive trend emerges from these tests.

Many of the inclined tests, when pulled out after cyclic loading failed by hardware failure rather than pullout (helixes broke off). The loads resisted before hardware failure tended to exceed the average static load capacity under this type of loading. Only in test SD13 was there a pullout as a result of strength deterioration by cyclic loading. The overall conclusion that can be drawn is that 200 cycles of 2/3 of the ultimate load are not likely to cause progressive anchor failure in either of the two loading modes used or to substantially weaken load capacity. However the anchor hardware will be weakened by the cyclic load in the inclined loading mode, and progressive soil failure could occur if the applied cyclic load approaches the load capacity of the anchors.

4.7.3 Cyclic Tests on the Clay Site

Typical test results are shown in figure 4.32. Specimens C24 and C25 are vertical anchors coaxially loaded to what was thought to be 0.75 Q_u. Both anchors experienced progressive failure. Specimen C26 is a preloaded specimen which performed well. However, its companion specimen, C27 (not shown) which was similarly preloaded, experienced progressive failure. An examination of the preloading curves of specimens C26 and C27, shown in figure 4.33, indicates that C27 experienced yielding during preloading. Thus the preload was very close to the ultimate load.

It is interesting to note, when comparing tests C25 and C26, that on the clay site the preloading effect did not occur in the initial load cycle, but rather tended to be gradual. This is attributed to the fast rate at which the cyclic load was applied. This loading rate did not permit enough time for the full

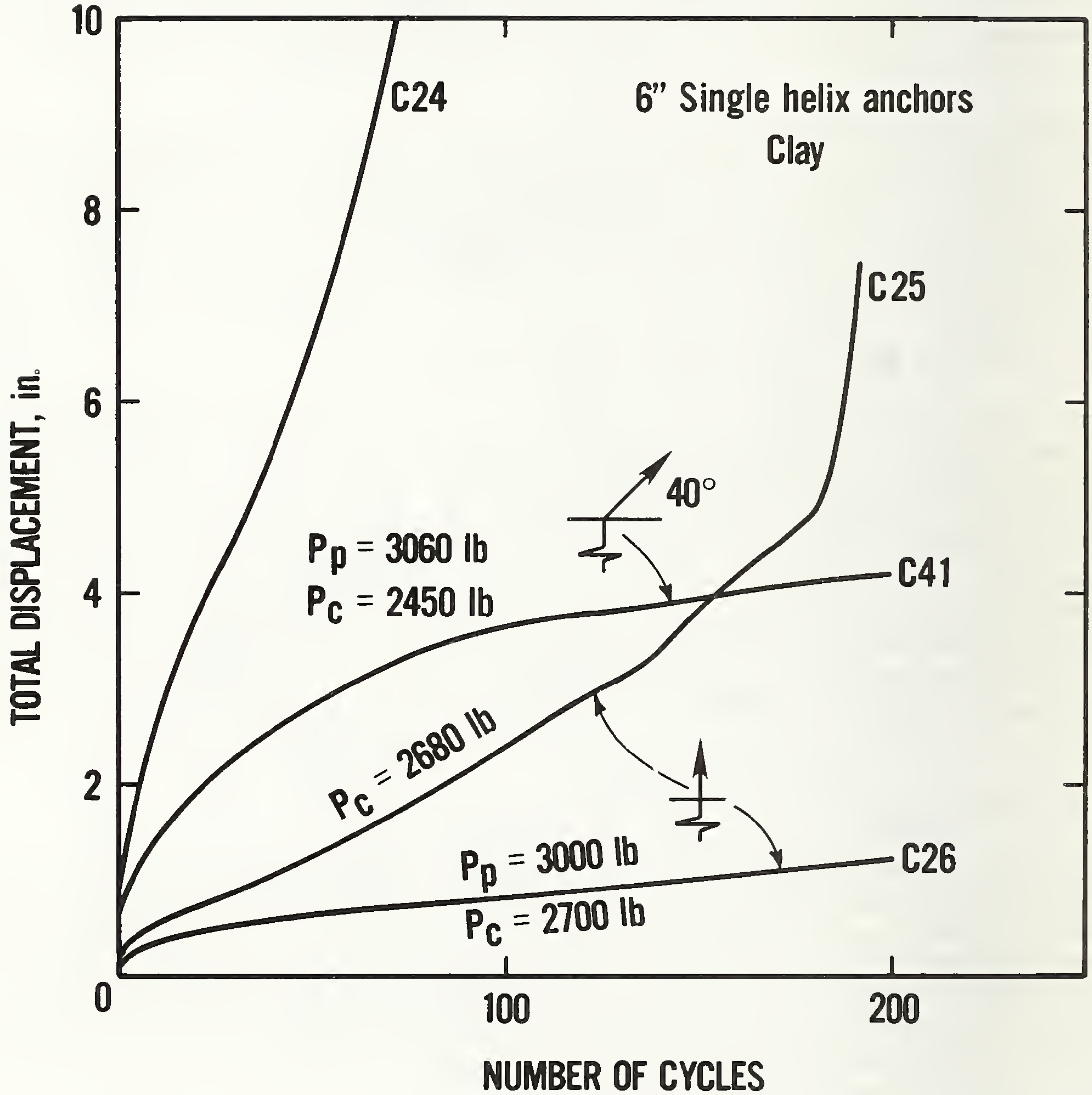

Figure 4.32 Cyclic load tests in clay

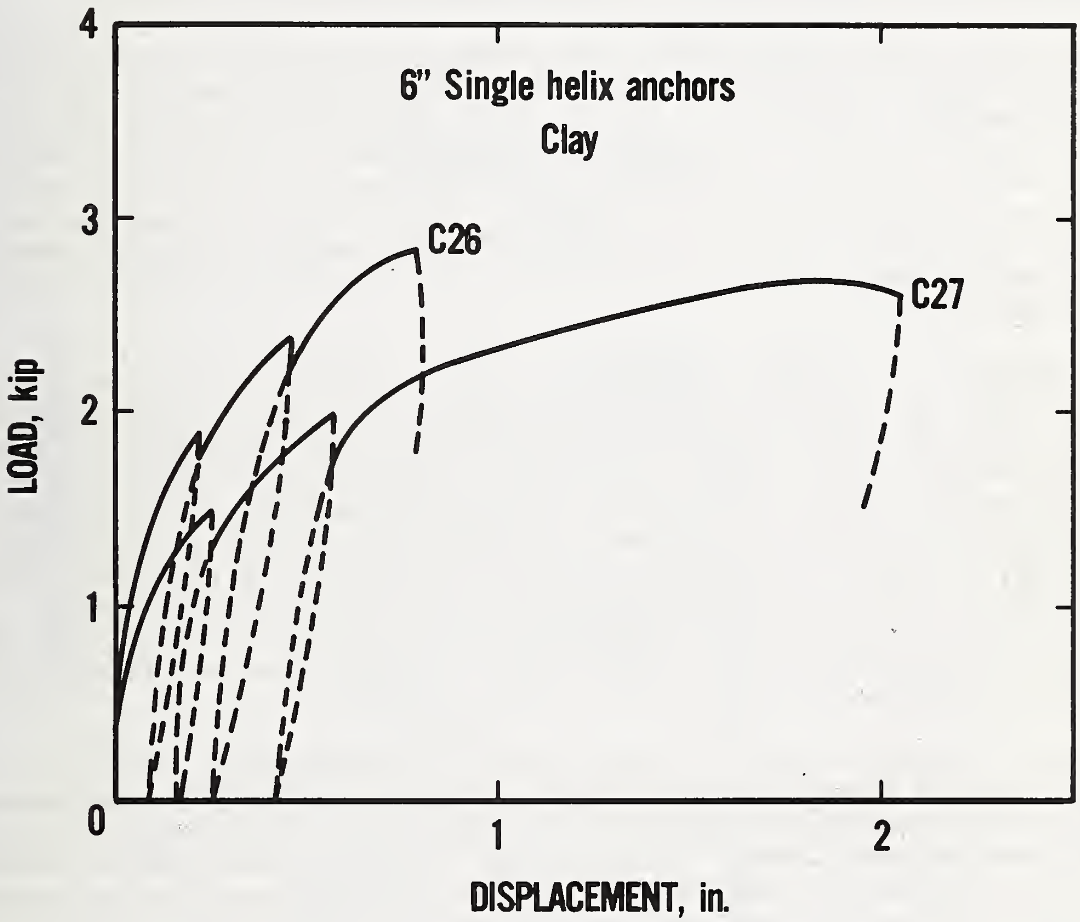

Figure 4.33 Comparison of the preloading curves of tests C26 and C27

displacement to occur in the first few load cycles. The load, rather than being resisted by the soil skeleton, induced porewater pressure gradients. The displacement occured gradually over many load cycles as these porewater pressure gradients dissipated.

Curve C41 in figure 4.32 is for a preloaded vertical anchor subjected to inclined pull. Note that, even though this specimen was preloaded, it experienced a movement of 4 inches during the 200 applied load cycles. Some of the anchors in clay failed before reaching 200 load cycles. Of eight 6-in single helix anchors, four failed before reaching 200 load cycles. If we divide these into preloaded and unpreloaded anchors, 3 of 4 unpreloaded, and 1 of 4 preloaded anchors failed. Of the eight 4-in double helix anhors tested, two of the four unpreloaded anchors and none of the four preloaded anchors failed.

Anchor C33, a 4-inch double helix, was loaded to 450 cycles in order to ascertain whether failure could be induced in anchors which perform satisfactorily for 200 load cycles. The results of test C33 are plotted in figure 4.34. Note that in the 10 to 200 cycle range the creep increment per cycle was about constant. After 200 cycles, the specimen deteriorated and failure occurred at 450 cycles. From the preloading curve, it appears that this specimen was loaded to 80 percent rather than 67 percent of ultimate. It is reasonable to assume that at these high cyclic loads all specimens would fail if enough load cycles are applied.

It is difficult to determine whether the load capacities of the specimens which did not fail were impaired by the application of 200 load cycles, since all the weaker specimens failed. Perhaps the best information can be derived from the coaxially-tested 4-in double helix anchors, which all survived the cyclic test (except that the test on anchor C33 was continued for 450 cycles until failure occurred). The three anchors tested had an average Q_u of 2067 lb This compares with an average load capacity of 1930 lb for the anchors tested statically. Since the variability of these test results is very small ($v = 0.07$ for the cyclic tests, and 0.03 for the static tests), this is taken as an indication that the load capacity of the anchors was not significantly affected by the cyclic tests.

The question should be asked whether cumulative displacement caused by windload effects would be within tolerable limits. Looking at the wind data in section 3.3.3(4) and assuming that increments of displacement would be approximately proportional to increments of load and that the cyclic load applied is equal to the design load, and thus the maximum windload experienced in the storm, the storm would cause a cumulative displacement of less than 100 cycles of the design load (this is a very conservative estimate). This would result in a cumulative displacement of 3 1/2 inches for anchor C41 and of less than 1 inch for anchor C26. Even though these displacements are considerable, they are not considered excessive for the extreme conditions assumed.

It is of interest to compare the anchor performance under cyclic load at the clay site with the results of other studies of the cyclic shear strength of

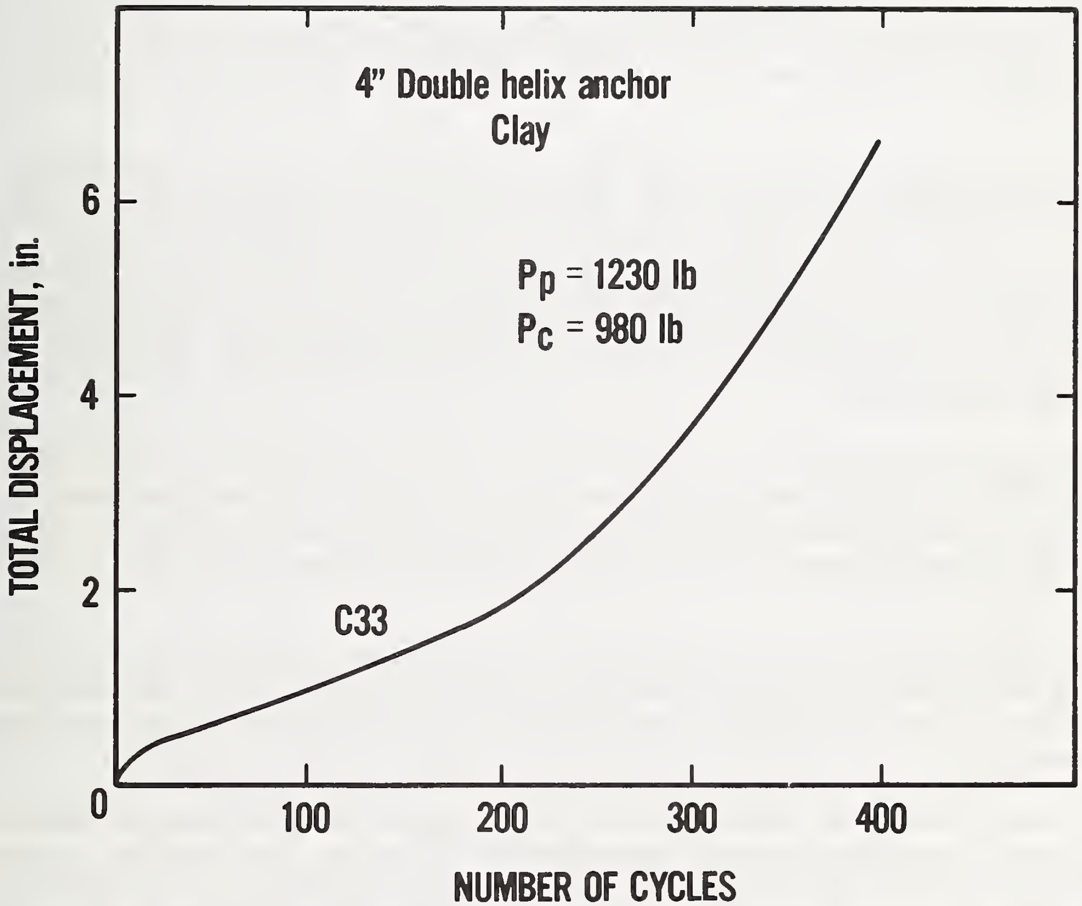

Figure 4.34 Results of the cyclic loading test of specimen C33 on the clay site

clays. Seed and Chan [16] studied three different clay types and found that, for loading conditions similar to those of the anchor tests (no shear stress reversals) the shear strength under 100 load cycles varied from 70 percent of the static shear strength for a soft sensitive clay to 80 percent of the static shear strength for compacted sandy clays. These findings are compatible with the results of this study and give further corroborations to the fiding that anchors in clays can survive 100 cycles of 67 percent of their failure load without failure.

4.8 COMPARISON OF ANCHOR PERFORMANCE WITH PRESENT STANDARD REQUIREMENTS

ANSI Standard A119.3 [2] requires that anchors resist a load of 4725 lb without failure, where failure is defined as a 2-inch displacement of the anchor head in the vertical direction or a 4-inch displacement in the horizontal direction. The HUD Mobile Home Construction and Safety Standard [9] sets even more conservative requirements by stipulating that loads be increased by 50 percent for the design of foundations. Hereafter these requirements are compared with the test results.

On the Silt Site (refers to table B.1), P_{2v} for the vertical full-depth 6-inch single helix anchors ranged from 3000 lb to 5750 lb and averaged at 4270 lb with v = 0.19. The average, as well as the lowest strength, are lower if the submerged tests are considered. The resistance of the diagonally loaded vertical anchors at 4-inch horizontal displacement and 45° pull ranged from 1100 to 1300 lb and averaged 1260 lb with v = 0.16. The values for the 4-inch double helix and the swivel anchors are not listed here since they fall far short of required capacities.

On the Sandy Site (refers to table B.3), P_{2v} for the vertical full-depth 6-inch single helix anchors ranged from 3900 lb to 6000 lb and averaged 4800 lb with v = 0.13. The resistance of the diagonally loaded vertical anchors at 4-inch horizontal displacement and 40° load inclination ranged from 2800 to 4000 lb and averaged 3200 lb.

On the Clay Site (refers to table B.5), P_{2v} for the vertical full-depth 6-inch single helix anchors ranged from 2300 lb to 3500 lb and averaged 3100 lb. The resistance of the diagonally loaded anchors at 4-inch horizontal displacement was negligible.

Thus, even though the sites selected were competent sites, anchor capacity fell far short of present standard requirements. Even on the sandy site, where anchors were relatively stiff and the average performance of the vertical anchors met standard requirements, many individual anchors did not meet the requirements. Not a single diagonally loaded specimen met the standard requirements.

It can be concluded from the test results that presently used anchor technology with present installation procedures did not deliver the performance required by ANSI Standard A119.3.

4.9 PERFORMANCE OF ANCHOR HARDWARE

In coaxial pullout, there were relatively few anchor hardware failures at loads lower than the stipulated 4725 lb capacity. In four instances, anchors failed below the 4725 load level, always by a break in the weld which connects the helix to the shaft. Two of the four failures occurred in cyclic tests. Many of the noncoaxially tested specimens failed because of anchor hardware failure. But these failures occurred at very high load levels and were in part caused by the fact that the soil resistance was extremely high.

Most anchors withdrawn had bent helixes (mushroom shaped). While the bending of the helix did not cause anchor failure, it may well have reduced the load capacity of the anchors and increased displacements. Almost all the anchors withdrawn had their paint stripped off. The paint stripping probably occurred during insertion. Thus it is concluded that painting does not provide effective corrosion protection. In the anchors which were subjected to non-coaxial pull, the anchor shaft was severely bent (see figure 4.7). Figure 4.35 shows typical anchor hardware failures.

The conclusions that can be drawn from the anchor hardware performance are that anchors should be galvanized or otherwise effectively protected against corrosion. The corrosion protection should not be damaged by installation and remain effective where yielding occurs during anchor installation or loading, i.e., on the anchor shaft and the helix. Another conclusion that can be drawn is that the load capacity of anchors could probably be improved by using a thicker helix plate that does not bend during withdrawal, and that during fabrication care should be exercised to insure the integrity of the weld between the anchor shaft and the helix plate.

FACING PAGE: *The "Soil Test Probe" used to predict anchor-load capacity is also a potentially useful device for the in-situ measurement of the shear strength of soils at a shallow depth.*

Figure 4.35 Typical anchor hardware failures

5. SUMMARY OF CONCLUSIONS

5.1 GENERAL

The findings presented herein are based on tests conducted in this project. Since soil is not a man-made material, the anchor behavior observed is not necessarily characteristic for all the sites that will be encountered in practice.

5.2 VIRGIN LOAD-DISPLACEMENT CURVES

All anchors tested had a unique virgin load-displacement curve which depended on the characteristics, installation depth and loading mode of the anchor and the soil conditions. The virgin load-displacement curve is a strength envelope which can not be changed by a limited number of intermediate unloading and reloading cycles, except that each unloading and reloading cycle will result in a small residual displacement. However, if a great number of intermediate load cycles was applied at a load level close to the pullout strength of the anchor they did in some instances cause incremental failure on the sand and the clay site. The virgin load-displacement curves observed were a unique function of the installation depth of the anchor. For instance, if an anchor is installed at the depth of 3 ft and withdrawn to the depth of 2.5 ft, unloaded and subsequently withdrawn, its load-displacement curve during withdrawal will differ from that of an anchor which is initially installed to a 2.5 ft depth and then withdrawn.

5.3 RELOADING CHARACTERISTICS

When an anchor is loaded to a certain load level and then unloaded and reloaded, the secant reloading modulus (reload/displacement caused by reload) will be several times larger (stiffer) than the secant modulus of the virgin loading curve. Thus, a preloaded anchor will have much more favorable load-displacement characteristics than an anchor which is loaded for the first time.

5.4 EFFECTS OF LOADING CONFIGURATION

Helix anchors installed vertically and pulled at an angle had a virgin load-displacement curve which exhibited much less stiffness than that of coaxially loaded anchors. However, the reloading characteristics of these anchors were superior to those required in present standards, even though they were adversely affected by elastic rebound of the anchor shaft. Helix anchors installed at an angle and withdrawn coaxially developed less load capacity than anchors installed vertically and pulled coaxially. The load capacity of coaxially loaded inclined anchors was roughly equal to that of coaxially loaded vertical anchors installed at the same helix depth below the ground surface (the difference in load capacity reflected the difference in embedment depth).

Vertically-installed helix anchors pulled at an angle to the vertical developed higher load capacities than coaxially loaded vertical anchors. Their load capacity increased as the angle of withdrawal with the horizontal was decreased. However, their resistance to displacement in the initial loading stages was very low.

Inclined helix anchors loaded at a 90° angle to the shaft had very low load capacities and low resistance to displacement.

Swivel anchors pulled at an angle to the vertical had either the same load capacity as vertically-pulled anchors (triangular anchors) or their load capacities decreased with a decrease of the angle of pull with the horizontal (pipe anchors).

5.5 EFFECTS OF SOIL TYPE

Anchors installed in the silt site had considerable ductility. (They could be withdrawn for a relatively large distance without a reduction in load capacity.) Anchors installed in clay had also considerable ductility, but their load capacity reached a peak and gradually decreased as they were further withdrawn. Anchors installed on the sand site lost their load capacity abruptly upon further withdrawal after their peak load capacity was reached.

5.6 PREDICTION OF ANCHOR LOAD CAPACITY BY IN-SITU TESTS

(1) Soil Test Probe

Soil Test Probe readings did correlate with anchor capacities, except that on the clay site some of the Test-Probe readings were abnormally low, apparently because of porewater pressure buildup. The capacity of the 6-inch single helix anchors can be reasonably correlated with Test Probe readings where the tip of the test probe was near the helix. The capacity of the 4-inch double helix anchors can be reasonably correlated with the average test probe reading over the depth of the anchor. There appears to be a good correlation between the in-situ shear strength of the soil and the test probe reading on the sand and the silt site. The results from the clay site were erratic.

(2) Installation Torque

It appears that it is possible to determine a lower bound for anchor pullout capacity from measurements of the installation torque at maximum penetration. However, the scatter of the data is considerable and the lower-bound prediction is too conservative to be of practical value.

(3) Standard Penetration Test

It does not appear that the Standard Penetration Test is a useful tool for predicting the strength of shallow anchors, mainly because of the short drill stem length used in shallow depths.

5.7 THEORETICAL PREDICTION OF ANCHOR-LOAD CAPACITY

Theoretical prediction of anchor-load capacity can only be as good as the estimate of the in-situ shear strength of the soil. Since in-situ shear strength at shallow depths is difficult to determine, the practical applicability of theoretical models is limited.

Correlation of the test results on the silt and clay sites with presently used theoretical pullout capacity models was poor. It was observed that anchor capacity per unit area of anchor plate increases as the anchor-plate area decreases. It was also observed that the swivel anchors have a higher load capacity per anchor-plate area than the helix anchors. In general, anchor load capacities were much higher than those that would be predicted on the basis of existing theoretical models and available data on soil-strength characteristics.

Correlation of the test results on the sandy site with presently used theoretical models was poor because the sand was overconsolidated and possibly cemented. Effects of depth and anchor plate size were similar to those predicted by theoretical models, but the anchor capacities were much higher than those that would be calculated on the basis of available data on soil-strength characteristics.

5.8 PREDICTION OF ANCHOR LOAD CAPACITIES ON THE BASIS OF TEST ON SIMILAR SITES

The coefficient of variation of anchor strength on the sites ranged from 0.18 to 0.40. There probably would be more variation if anchor test results from one site are used to predict anchor strength at another site.

5.9 ANCHORS SUBJECTED TO CYCLIC LOAD

On the silt site, anchors tended to stabilize after 100 load cycles and additional cycles caused no further creep. On the sand and clay site, creep displacement continued indefinitely and failure could be induced if enough load cycles are applied. Failure actually occurred in some specimen where the applied cyclic load was close to the anchor load capacity.

The cyclic-load performance of anchors, preloaded to a load higher than the applied cyclic load was superior to the performance of unpreloaded anchors. There was no evidence that the pullout strength of anchors was reduced by applying 200 cycles of about 2/3 of their pullout strength.

It appears that on all three sites anchors could survive the effects of a major hurricane with displacements smaller than those permitted in the present ANSI Standard (2-inch vertical and 4-inch horizontal), provided that the maximum wind load effect does not exceed 2/3 of the pullout strength of the anchor and the anchors are preloaded.

5.10 COMPARISON OF ANCHOR PERFORMANCE WITH PRESENT STANDARD REQUIREMENTS

Presently-used anchoring technology did not provide the anchor performance required by ANSI Standard A119.3, neither in terms of load capacity, nor in terms of load-displacement characteristics.

5.11 PERFORMANCE OF ANCHOR HARDWARE

Anchor hardware generally developed the required load resistance. After installation, painted anchors do not seem to have effective corrosion protection because of paint stripping. Most anchor helixes were bent after anchor withdrawal, indicating that anchor performance could probably be improved by thicker helix plates.

The shafts of anchors installed vertically and withdrawn at an angle were severely bent before the anchors reached their maximum load capacity.

Most anchor hardware failures occurred by a failure of the weld between the shaft and the helix plate. Some of these failures were induced by cyclic loading.

5.12 USE OF SOIL ANCHORS IN PERMANENT MOBILE HOME FOUNDATIONS

The possibility of using anchors in permanent mobile home foundations has recently received some consideration.

It is evident from the test results, that if anchors are to be included as part of a permanent foundation they must have adequate corrosion protection to retain their structural integrity throughout the service life of the mobile home and they should be preloaded to insure adequate performance under anticipated extreme loads.

Such anchors would also have to be adequately protected against potential effects of frost heave.

FACING PAGE: *Pullout test of submerged anchor installed in swail crossing Site B.*

6. RECOMMENDATIONS

6.1 REQUIRED LOAD CAPACITY

Recommendation:

It is recommended that the requirement for a 4725 lb load capacity for anchors be abandoned. Instead, it is recommended to stipulate the required total working load that the anchoring system must resist, and then determine the number of anchors required to achieve this performance on the basis of anchor capacity that can be achieved at particular sites. This requirement will have to be coupled with a maximum allowable spacing requirement to avoid unreasonably wide spacing in dense soils. The implementation of this approach will require an initial estimate of the anchor capactiy at an installation site either by a load test or by previous experience. To avoid unnecessary costs the allowable working load for the anchors could be established during the pre-loading of the first anchor installed on the site.

Commentary:

As a result of this test program, it was determined that on the sites selected, which had competent soils, existing anchor technology did not provide the required 4725 lb load capacity required by ANSI A119.3.

6.2 INSTALLATION REQUIREMENTS

Recommendations:

The following procedure is recommended:

1. Each anchor installed must be preloaded to 1.25 its working load.

2. One anchor per mobile home, or three anchors per site where the soil conditions are uniform, must be preloaded to 1.5 the working load.

3. The working load (P_w) is defined as the anchor load induced by the design wind pressure (without the 50 percent increase required by HUD) [9].

A suggested preloading procedure for diagonally loaded anchors is shown schematically in figure 6.2. Loading devices for vertically loaded anchors are commercially available.

Commentary:

Figure 6.1 illustrates the intent of the recommended procedure. The dashed curve is the loading curve of specimen C7. The load capacity Q_u = 3.8k, preload P_p = 3.17k and working load P_w = 2.53k. If the specimen is preloaded in accordance with the recommended procedure, the reloading modulus will be high and the anchor performance, accordingly, excellent. Had the specimen been preloaded to Q_u, it is conceivable that the load capacity upon reloading would be less than Q_u. The 1.5 safety margin is intended to provide sufficiently high probability that the anchor will resist the working load. The 1.25 P_w preload will insure good anchor performance. Note that the reloading modulus would be much lower if the preload were only 1 P_w.

The provision that some of the anchors should be loaded to 1.5 P_w is to provide some assurance that the preload will not approach the anchor-load capacity since this could weaken the anchors. Another way in which this could be accomplished

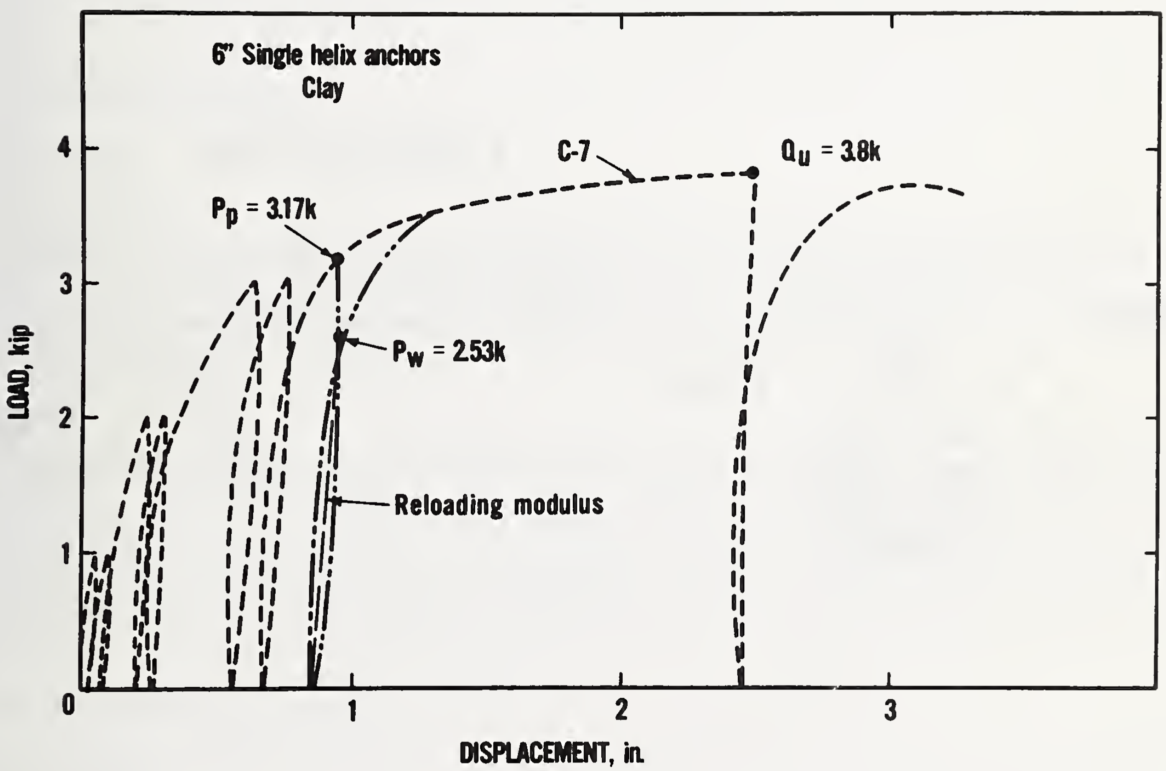

Figure 6.1 Illustration of the recommended preloading requirement and the resulting anchor performance

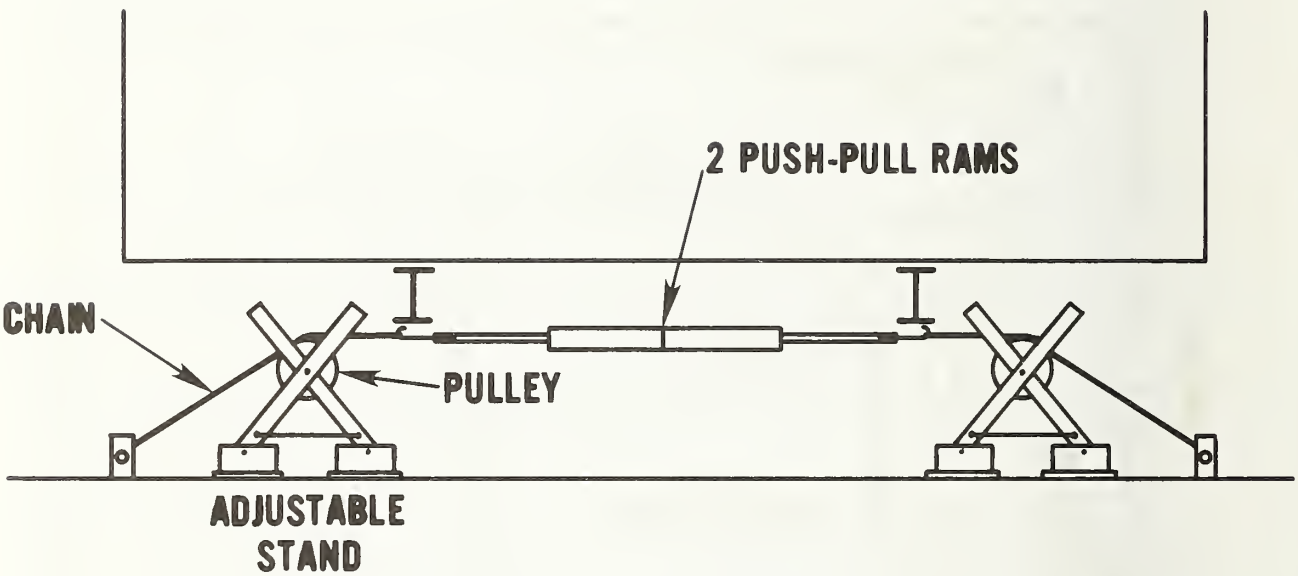

Figure 6.2 Suggested preloading procedures for diagonally loaded anchors

would be to stipulate that the preload be held for five minutes without measurable creep (say not more than 1/4 inch). However such a procedure is considered too cumbersome.

It is conceivable on sites where the variability of anchor capacity is high that in some instances P_p will be as high as Q_u. A good installer would probably decide in such a case that he needs more anchors and would reduce his preload. However, if he keeps pulling in order to try to reach the preload, he may pull out his anchors too much during the preloading process. This could be forestalled by stipulating a maximum allowable displacement during the preloading. However, it is difficult to come up with a displacement magnitude which could apply to all anchor types and soil conditions. A stipulation that preloading should be discontinued if an anchor moves more than two inches without an increase in applied load would probably provide adequate protection against overloading.

The test results also indicate that anchors preloaded by the recommended procedure would perform adequately under wind-induced cyclic loading.

6.3 CORROSION PROTECTION

Recommendation

Anchors should have adequate corrosion protection for the service life of the mobile home. The corrosion protection should remain effective after the anchor is subjected to inelastic deformations similar to those anticipated during preloading. Painting or any other coating that could be damaged by installation and preloading is not acceptable.

Commentary:

Most anchors tested lost much of their coat of paint during installation. Galvanized anchors would probably perform adequately, but there is concern that the anchor may become vulnerable to corrosion because of deformations induced by preloading or service load. All diagonally loaded anchors experienced large deformations in the shaft, and most anchors tested experienced deformations in the helix.

6.4 ANCHOR HARDWARE CAPACITY

Recommendations:

Anchor hardware should resist two times the service load without rupture. Inelastic deformations are permitted if it can be demonstrated that the durability of the anchor will not be impaired.

Commentary:

It is unrealistic to expect that anchor hardware should not experience inelastic deformations as the anchor adjusts to the applied load. The safety margin of two is to insure that welds will not fail during anticipated cyclic loads.

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8. ACKNOWLEDGMENTS

The contribution of the following persons is gratefully acknowledged: Mr. James McCollom of HUD monitored the NBS contract and made many constructive suggestions; Dr. William D. Kovacs of NBS conducted the state-of-the-art study which recommended this work and contributed to the planning and interpretation of the tests; Mr. Christopher L. Mullen participated in the design of the test rig; Mr. J. T. Odom of A. B. Chance Co. reviewed the design of the test rig; Mr. Erik D. Anderson and Mr. Mike P. Glover participated in the testing work in the field; MHA corporation, Minuteman Corporation, A. B. Chance, Tiedown Engineering and Abema, Inc., supplied anchor hardware and installation devices, gave advice, and installed some of the anchors. Some of this assistance was rendered at no cost to the government. Schnabel Engineering Corporation identified two of the test sites and conducted the soil exploration on all the test sites.

APPENDIX A

Test Sites

A.1. Introduction

Appendix A Contains information on anchor location and subsurface information on three test sites. Site A is in silty soil; Site B is in sandy soil; and Site C is in clay soil.

Figures A1 through A4 show the location of anchor tests, test borings and test pits. The anchor-location coordinates are given for each test identified in Appendix B. Tests W1 through W17 on Site A are located in a drainage swail where the soil is permanently submerged.

There are two separate test areas on Site B. Area B2 was chosen because part of Area B1 was overlain by a dense crust and thus some of the test locations were not utilized. A test pit was dug on Site B2 to explore soil conditions.

The appended soil exploration reports contain the logs of the borings shown in the location maps, as well as laboratory test results. Information on Soil Test Probe readings is included in the tables in Appendix B.

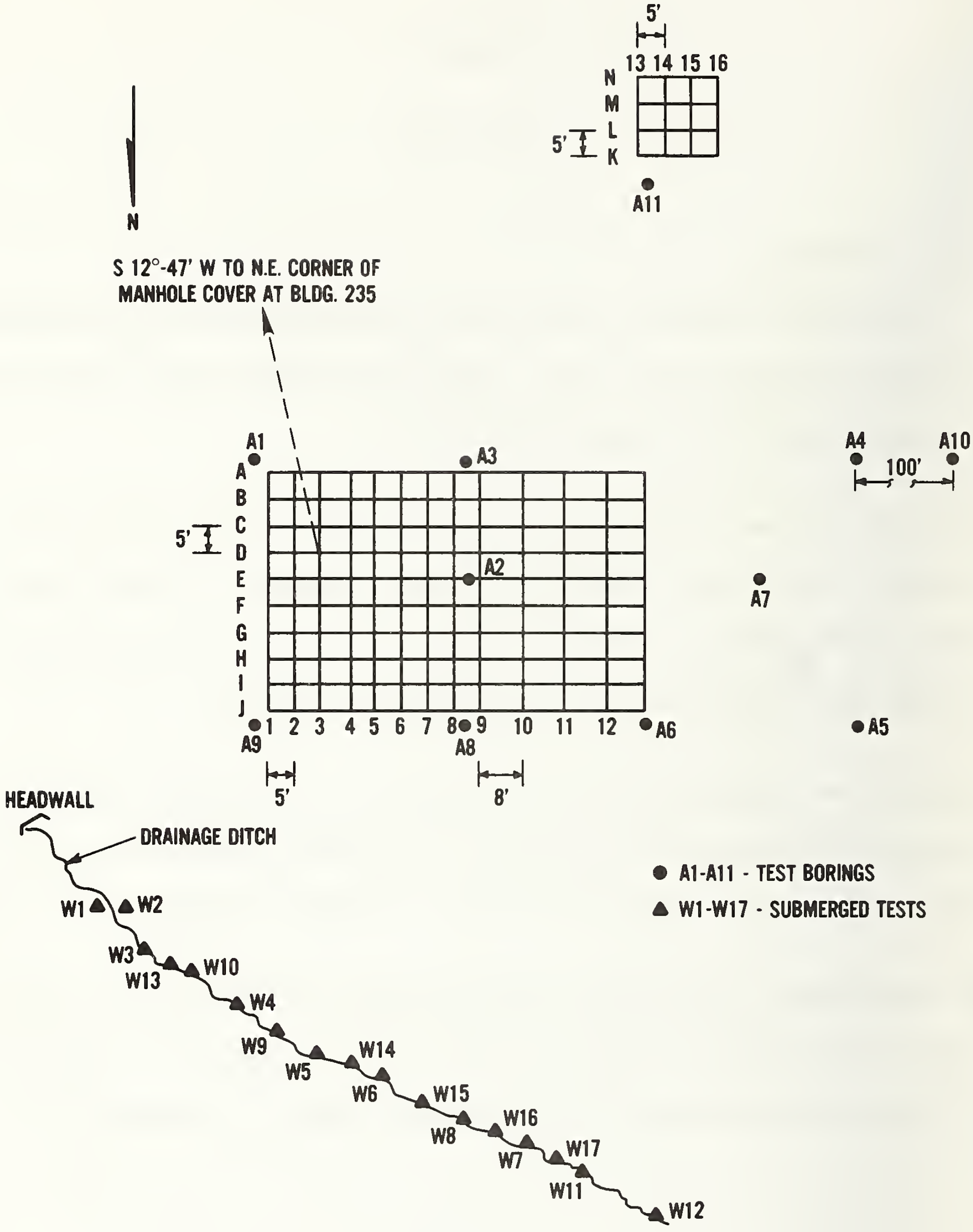


Figure A.1 Anchor Test and Boring Locations on Site A

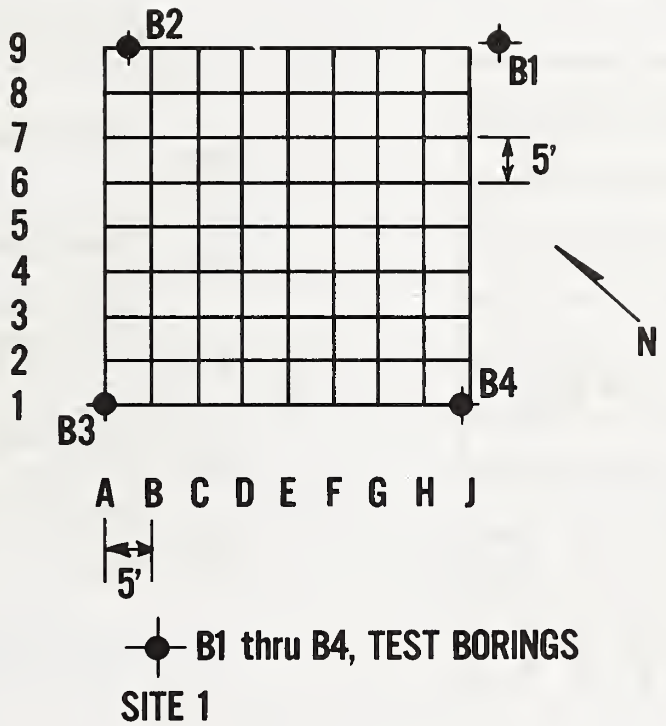


Figure A.2 Anchor Test and Boring Locations on Site B.1

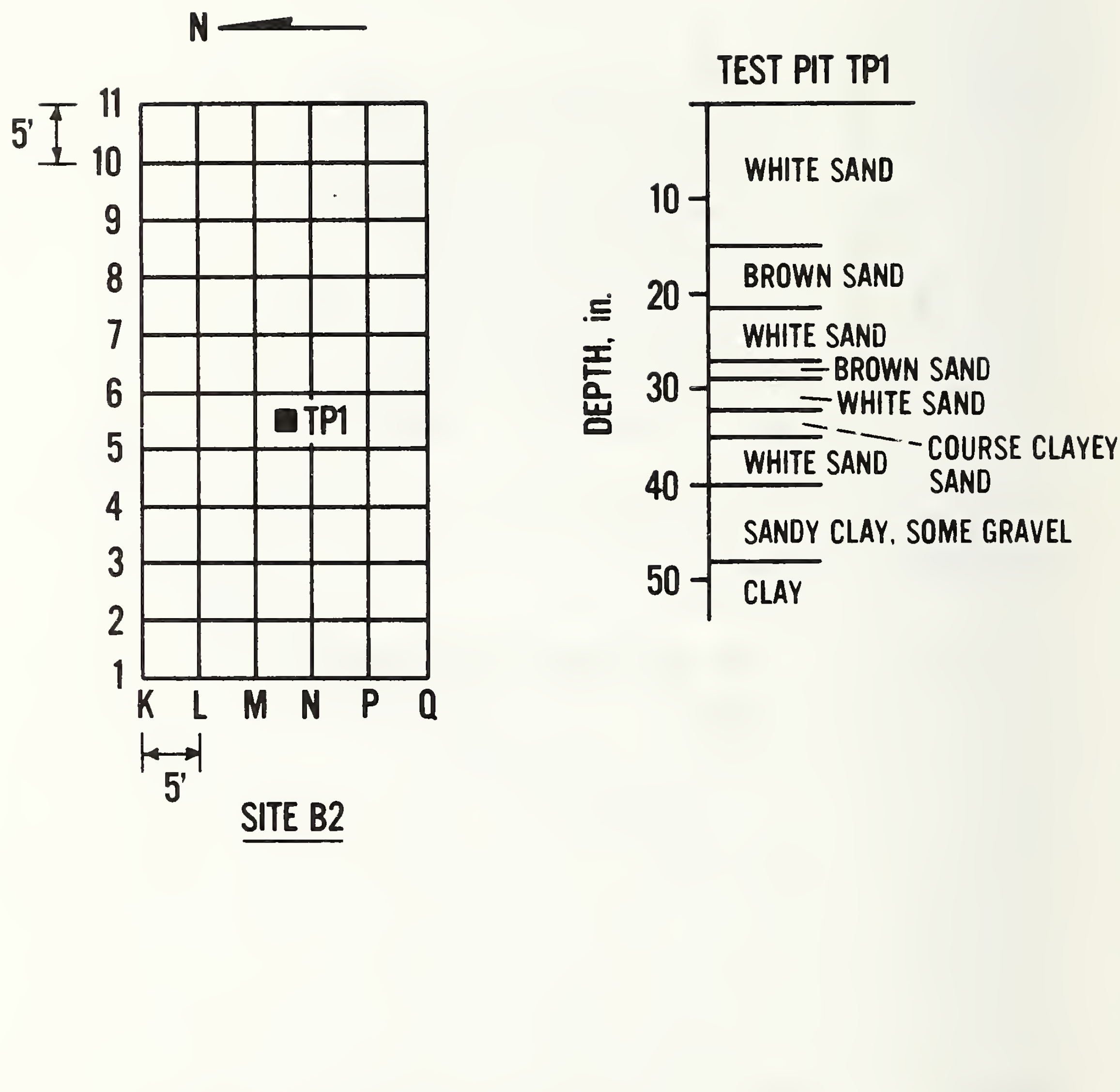


Figure A.3 Anchor Test and Test Pit Locations on Site B.2

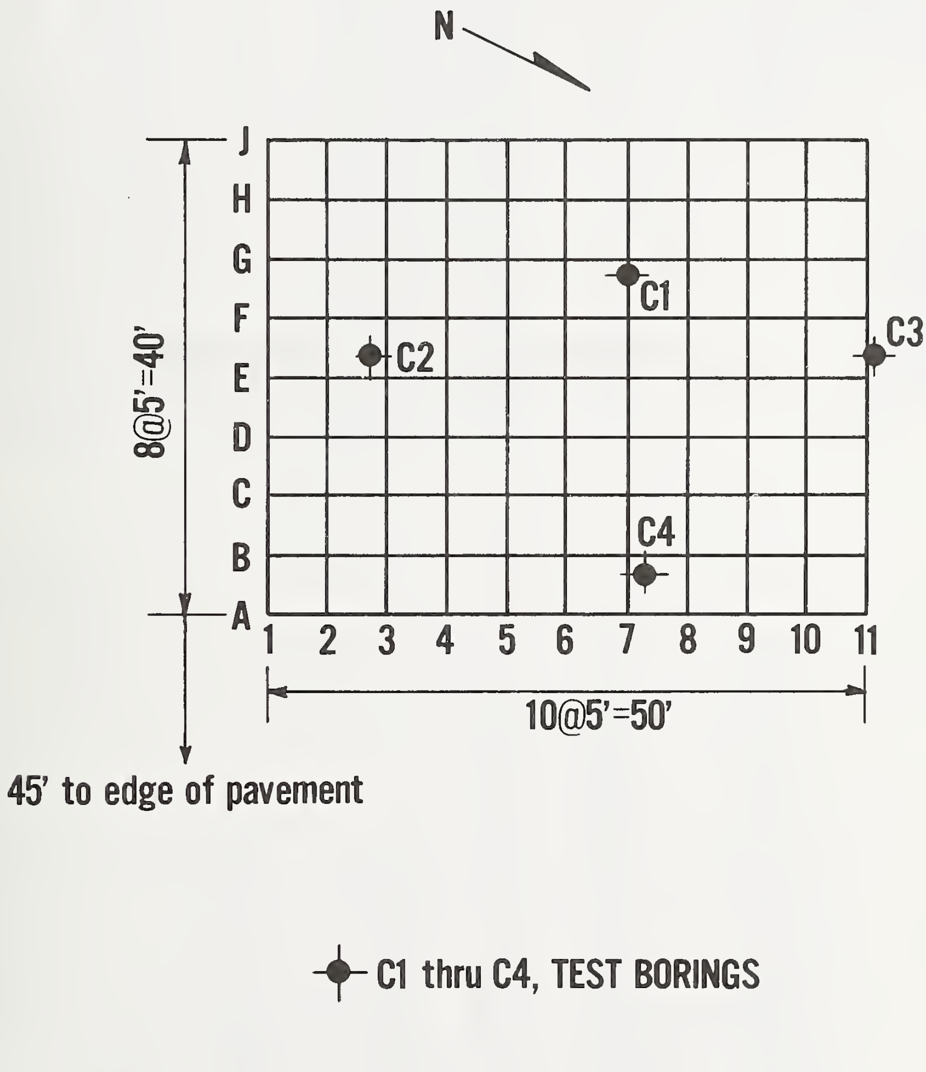


Figure A.4 Anchor Test and Boring Locations on Site C

A.2 Soil Exploration Reports

SCHNABEL ENGINEERING ASSOCIATES

CONSULTING ENGINEERS

SOIL MECHANICS AND FOUNDATIONS

JAMES J. SCHNABEL, P.E.
ERNEST WINTER, P.E.

4800 CORDELL AVENUE
BETHESDA, MARYLAND 20014
301/652-4130

November 5, 1979

Dr. Felix Yokel
National Bureau of Standards
Center for Building Technology
Building 226, Room B-168
Washington, D.C. 20234

Project: Testing Services for Mobile Home Anchorage Project, Phase I, Various Locations, Washington, D.C. area (M79043)

Report for Site "A"
National Bureau of Standards
Reservation, Gaithersburg, MD

Gentlemen:

Submitted herewith are four copies of our report of subsoil exploration and soil laboratory testing for the first site on this project, designated Site "A".

Site

The test site for this series of tests was on the National Bureau of Standards reservation in Gaithersburg, MD. Soils encountered were artificial fills, some with foreign matter detected, overlying natural residual silts and sands derived from the in-place weathering of the underlying schist rock of the Wissahickon Formation.

Field Testing

Eleven soil test borings were drilled by our subsidiary, Foundation Test Service, Inc., at locations selected by your representatives in about the pattern shown on the enclosed drawing, to a depth of 5 ft each. Six borings were planned but additional borings were drilled at your direction to explore a new test area while we were on the site. Two undisturbed tube samples were recovered in an alternate boring adjacent to A-7.

Dr. Felix Yokel
November 5, 1979
Page Two

Group symbols after the description of soils are in accordance with the Unified Soil Classification System and were based on visual examination of samples. The system used by this firm for identifying soils is included in Enclosure (1), Identification of Soil.

The standard penetration resistance, or N value, indicates the penetration resistance in blows per foot of a 2 in O.D., 1-3/8 in I.D. sampling spoon driven with a 140 lb hammer falling 30 in per ASTM D-1586. After an initial set of 6 in to assure the sampler is in undisturbed material, the number of blows required to drive the sampler an additional 12 in is usually taken as the N value. The penetration for each 6 in of drive is shown in the right hand column of the test boring logs.

No groundwater was observed in any of the borings during drilling or up to 5 hours after drilling. Long term water observations were not made.

Soil Laboratory Testing

Natural moisture content tests were conducted on jar samples and a natural density test was performed on one tube sample. Atterberg Limits and sieve analyses were conducted to determine physical properties of soils, and an unconfined compression and direct shear test were performed to determine strength characteristics of natural soils. These soil laboratory test results are summarized in Enclosure 2, and indicate that the natural soils tested are typically a clayey silt, with some fine sand, or ML according to ASTM D-2487. Visual classifications indicate that soils described as fine sandy silt are also present. Traces of quartz fragments were detected in some samples. Moisture contents ranged from 12.5% to 30.8%. The natural dry density of one specimen was 97.6 pcf. Strength testing indicated an unconfined compressive strength of about 4000 psf on one sample, while a direct shear test yielded $\phi = 30^{o}$ and c = 200 psf. Direct shear test results may have been influenced by the presence of quartz fragments in the soil specimen.

Soil samples remaining will be retained until December 20, 1979 and then discarded unless other disposition is requested.

Please call if there are questions concerning this initial submittal.

Very truly yours,

SCHNABEL ENGINEERING ASSOCIATES, P.A.

J. A. Padgett, MECE
Project Manager

James J. Schnabel, P.E.
State of Maryland

Encl:

(1) Identification of Soil
(2) Summary of Soil Laboratory Tests
Gradation Curve (1)
Unconfined Compression Curve (1)
Direct Shear Curve (1)
(3) Subsoil Investigation Report

Enclosure (1)
Contract No. M79043

SCHNABEL ENGINEERING ASSOCIATES

Consulting Geotechnical Engineers

IDENTIFICATION OF SOIL

I. DEFINITION OF SOIL COMPONENTS				II. DEFINITION OF COMPONENT PROPERTIES		
Major Material Component	Material Fraction	Sieve Size	Plasticity	Component	Proportions of Soil Components	Approximate Percentage by Weight
GRAVEL, GM, GC, GP, GW	Coarse Fine	3/4 to 3" No 4 to 3/4	– –	Major	Noun Form Gravel, Sand, Silt, Clay, etc.	50 or more
SAND, SM, SC, SP, SW	Coarse Medium Fine	No 10 to No 4 No. 40 to No 10 No. 200 to No. 40	– – –	Minor	Adjective Form Gravelly, Sandy, Silty, Clayey Silty, Clayey, Silty Clayey	35 to 50
SILT, ML	–	Passing No. 200	Non-plastic		Some Some Gravel, Some Silt, etc.	12 to 35
CLAYEY SILT, ML, MH, CL-ML	–	Passing No. 200	Slight to High			
SILTY CLAY, CL	–	Passing No. 200	Medium to High		Trace Trace Gravel, trace sand, etc.	1 to 12
CLAY, CH	–	Passing No. 200	Very High		With with rock fragments, with organic matter, etc.	indicates presence only
ORGANIC SILT, OH, OL	–	Passing No. 200	Slight to High			
PEAT, Pt	Partially decomposed fibrous organic matter with or without silt or sand filter					

III. GLOSSARY OF MISCELLANEOUS TERMS

SYMBOLS – Unified Soil Classification Symbols are shown in major material component column. Use A Line Chart for laboratory identification.

BOULDERS – Rounded pieces of rock larger than 3 inches

DISINTEGRATED ROCK – Residual soil with a standard penetration resistance of at least 60 blows or more per foot

ROCK FRAGMENTS – Angular pieces of rock, distinguished from transported gravel, which have separated from original vein or strata and are present in a soil matrix.

QUARTZ – A hard silica mineral often found in residual soils

IRONITE – Iron oxide deposited within a soil layer forming cemented deposits

CEMENTED SAND – Usually localized rock-like deposits within a soil stratum composed of sand grains cemented by calcium carbonate or other minerals

MICA – A soft silica mineral found in many rocks, and in residual or transported soils derived therefrom

FISSURED CLAYS – Cohesive soils exhibiting a joint structure

ORGANIC MATERIAL (Excluding Peat): Top Soil – Surface soils that support plant life and which contain considerable amounts of organic matter; Decomposed Vegetation – Partially decomposed organic matter which retains its original character, Lignite – Decomposed organic matter with low fixed carbon content frequently exhibiting distinct texture of wood

FILL – Man made deposit containing soil, rock and often foreign matter

PROBABLE FILL – Soils which contain no visually detectable foreign matter but which are suspect with respect to origin

LENSES – 0 to 1/2 inch layer of minor soil component

LAYERS – 1/2 to 12 inch layers of minor soil component

POCKET – Discontinuous pocket of minor soil component

COLOR SHADES – Light or dark to indicate substantial differences in color

MOISTURE CONDITIONS – Wet, moist, or dry to indicate visual appearance of specimen

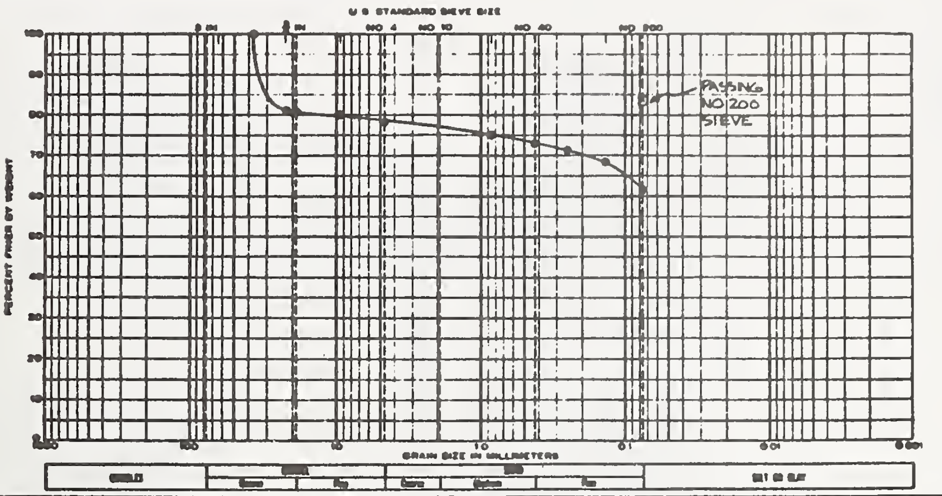
Enclosure (2)
Contract No. M79043

SUMMARY OF SOIL LABORATORY TESTS

Boring No.	Sample Depth	Sample Type	Description of Soil Specimen	Percent Passing No. 200 Sieve	Atterberg Limits LL	PL	PI	Natural Moisture (%)	Natural Density (pcf) Wet	Dry	Soil Parameters	Remarks
A-5	1.5'	Jar	CLAYEY SILT, some fine sand, brown (ML)	-	39	27	12	17.6	-	-	-	Bottom of Sample
A-7a	0.5'-2.5'	3" Tube	CLAYEY SILT, some fine sand with quartz fragments, brown (ML)	61.9	-	-	-	19.8	-	-	-	See Gradation Curve
A-7a	3'-5'	3"	CLAYEY SILT, some fine sand with quartz fragments, brown (ML)	83.0	-	-	-	21.6	118.7	97.6	ϕ = 30° c = 200 psf q_u= 4029 psf @ e = 4.0%	See Gradation, Unconfined Compression & Direct Shear Curves

Notes:

1. Soil test in accordance with applicable ASTM Standards.
2. Soil classification symbols are in accordance with Unified Soil Classification System, based on testing indicated and visual identification.
3. Visual identification of samples is in accordance with the system used by this firm.
4. Key to abbreviations: LL=Liquid Limit; PL=Plastic Limit; PI=Plasticity Index; q_u=Unconfined Compressive Strength; c= Apparent Cohesion; ϕ=Soil Friction Angle; e=Strain.
5. Natural moisture content determinations were run on jar samples from all borings. Results are shown on Test Boring Report.
6. Soil tests were conducted by L. Trulio, K. Santill, E. Sonnenberg & T. Charlton.



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASSIF.	M C	LL	PI
⊙	A-7a	.5-2.5	CLAYEY SILT, SOME FINE SAND WITH QUARTZ FRAGMENTS, BROWN	ML	19.8	—	—
⊡	A-7a	3-5	CLAYEY SILT, SOME FINE SAND WITH QUARTZ FRAGMENTS, BROWN	ML	21.6	—	—

SCHNABEL ENGINEERING ASSOCIATES

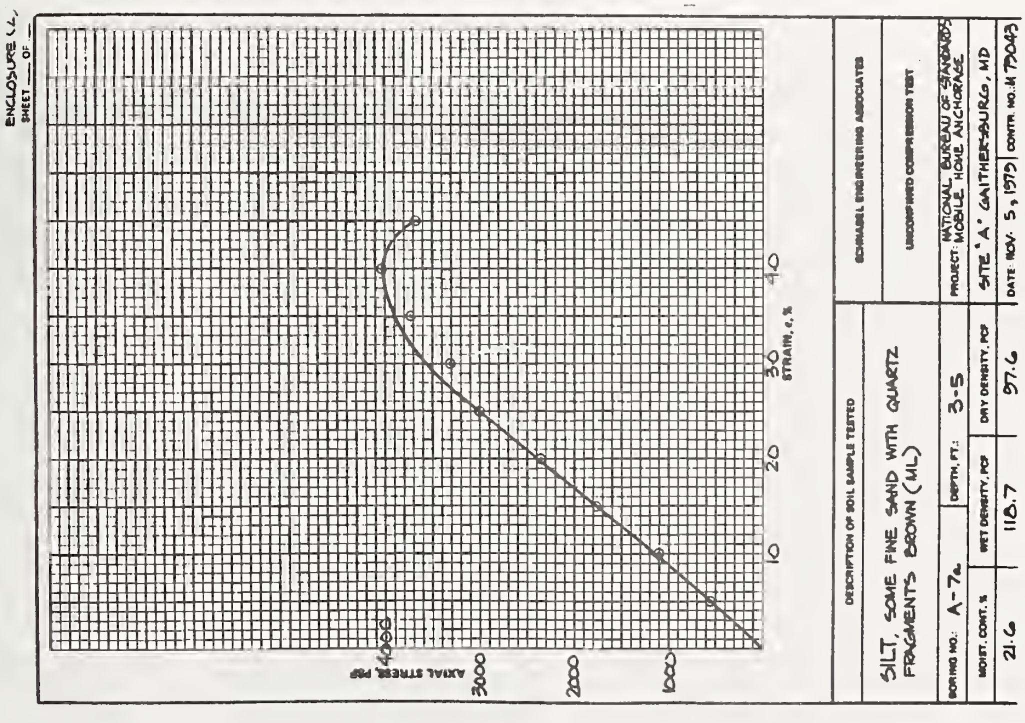
GRADATION CURVES

PROJECT NATIONAL BUREAU OF STANDARDS MOBILE HOME ANCHORAGE

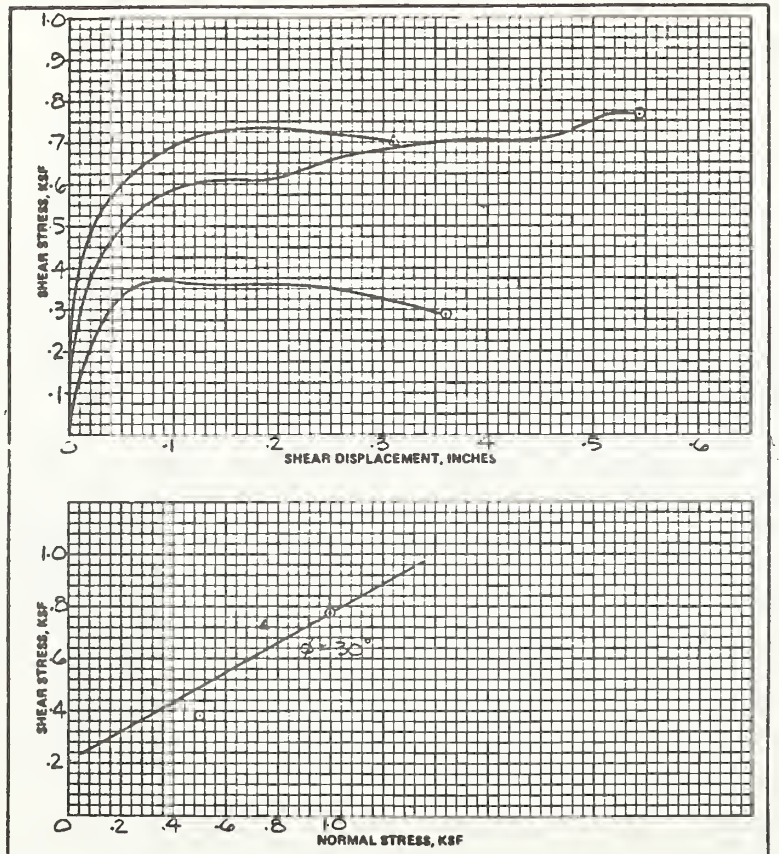
SITE "A" GAITHERSBURG, MD

DATE NOV. 5, 1979 CONTR. NO M75043

ENCLOSURE (2) SHEET 10



ENCLOSURE (2)
SHEET ___ OF ___



DESCRIPTION OF SOIL SAMPLE TESTED							
SILT, SOME FINE SAND, WITH QUARTZ FRAGMENTS, BROWN (ML)							
KEY	BORING NO.	DEPTH FT.	NORMAL STRESS	MOIST. CONT. % INITIAL	MOIST. CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
⊙	A7a	3.5'	500	21.6	–	97.6	118.7
△	A7a	3.5'	750				
⊙	A7a	3.5'	1000				

SCHNABEL ENGINEERING ASSOCIATES

DIRECT SHEAR TEST

TYPE OF TEST: CU
RATE OF SHEAR: 0.06"/MIN.
PROJECT: NATIONAL BUREAU OF STANDARDS MOBILE HOME ANCHORAGE
SITE "A" GAITHERSBURG, MD
DATE: NOV. 5, 1979 CONTR. NO.: M 79043

Enclosure (3)
Contract No. M79043

SUBSOIL INVESTIGATION REPORT

Test Boring Report

Description of Subsoil Investigation Procedures:

1. Test Borings

Hollow Stem Augers

The borings are advanced by turning an auger with a center opening of 2-1/4 to 3-1/4 inch. A plug device blocks off the center opening while augers are advanced. Cuttings are brought to the surface by the auger flights. Sampling is performed through the center opening in the hollow stem auger, by standard methods, after removal of the plug. No water was introduced into the boring using this procedure.

2. Standard Penetration Tests

Testing is performed by driving a 2 inch O.D., 1-3/8 inch I.D. sampling spoon through three 6 inch intervals or as indicated, using a 140 pound hammer falling 30 inches, according to ASTM D-1586.

3. Boring Locations and Grades

The borings were drilled at locations selected in the field. No plans available for surveying and elevations were not obtained.

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CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 1 OF 8
DATE 11-5-79
CONTRACT NO. M79043
ENCL. 3

BORING NUMBER A1
GROUND SURFACE ELEVATION

Depth	Description	Blows	Moisture
0.2'	Topsoil		
	sandy silt, FILL, with quartz fragments, moist, brown	3+5+7	
		4+7+9	18.9%
3.5'	fine sandy SILT (ML), trace quartz fragments, moist, brown	2+4+5	18.5%
5.0'			

bottom of boring @ 5.0'

BORING COMPLETED 7-19-79
WATER LEVEL READINGS
ENCOUNTERED none
UPON COMPLETION none
WATER none AFTER 4.5 hrs
CAVED AT -

BORING NUMBER A2
GROUND SURFACE ELEVATION

Depth	Description	Blows	Moisture
0.2'	Topsoil		
	sandy silt, FILL, with quartz fragments, moist, brown	3+6+8	
		7+7+8	21.7%
2.5'	CLAYEY SILT (ML), some fine sand, moist, brown	3+4+5	22.6%
5.0'			

bottom of boring @ 5.0'

BORING COMPLETED 7-19-79
WATER LEVEL READINGS
ENCOUNTERED none
UPON COMPLETION none
WATER none AFTER 3 hrs
CAVED AT -

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SHEET NO. 2 OF 8
DATE 11-5-79
CONTRACT NO. M79043
ENCL. 3

BORING NUMBER A3
GROUND SURFACE ELEVATION

Depth	Description	Blows	Moisture
0.2'	topsoil		
	sandy silt, FILL, moist, brown	6+9+9	
1.5'	CLAYEY SILT, some fine sand with quartz fragments, moist, brown	4+5+6	23.3%
3.0'	fine sandy SILT, moist brown	3+3+4	20.5%
5.0'			

bottom of boring @ 5.0'

BORING COMPLETED 7-19-79
WATER LEVEL READINGS
ENCOUNTERED none
UPON COMPLETION none
WATER none AFTER 2.5 hrs
CAVED AT -

BORING NUMBER A4
GROUND SURFACE ELEVATION

Depth	Description	Blows	Moisture
0.2'	topsoil		
	sandy silt, FILL, with quartz fragments, moist, brown	5+8+8	
1.5'	CLAYEY SILT (ML), some fine sand, moist, brown	4+7+9	18.0%
3.5'	fine sandy SILT (ML), moist, brown	4+5+5	17.7%
5.0'			

bottom of boring @ 5.0'

BORING COMPLETED 7-19-79
WATER LEVEL READINGS
ENCOUNTERED none
UPON COMPLETION none
WATER none AFTER 2½ hrs
CAVED AT -

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SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 3 OF 8
DATE 11-5-79
CONTRACT NO. M79043
ENCL. 3

BORING NUMBER A5
GROUND SURFACE ELEVATION

Depth	Description	Blows	Moisture
0.2'	topsoil		
	CLAYEY SILT(ML), some fine sand, moist, brown	2+4+4	
		4+5+6	17.6%
3.0'	fine sandy SILT(ML), moist, brown	5+7+9	16.1%
5.0'			

bottom of boring @ 5.0'

BORING COMPLETED 7-19-79

WATER LEVEL READINGS

ENCOUNTERED none
UPON COMPLETION none
WATER none to 4.9', AFTER 4 hrs
CAVED AT 4.9' and dry

BORING NUMBER A6
GROUND SURFACE ELEVATION

Depth	Description	Blows	Moisture
0.2'	topsoil		
	sandy silt, FILL, moist brown	3+5+5	
		6+7+7	22.6%
2.5'	CLAYEY SILT(ML), some fine sand, moist, brown	3+6+8	15.3%
5.0'			

bottom of boring @ 5.0'

BORING COMPLETED 7-19-79

WATER LEVEL READINGS

ENCOUNTERED none
UPON COMPLETION none
WATER non AFTER 5 hrs
CAVED AT -

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SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 4 OF 8
DATE 11-5-79
CONTRACT NO. M79043
ENCL. 3

BORING NUMBER A7 and A7a
GROUND SURFACE ELEVATION

Depth	Description	Blows	Sample / Moisture
0.2'	topsoil		
	sandy silt, FILL, moist, brown	6+8+9	P-24"
1.5'	CLAYEY SILT(ML), some fine sand, moist, brown	4+4+8	R-24"
			19.8%
3.5'	fine sandy SILT(ML), with quartz fragments, moist, brown	4+6+8	P-24"
			R-22"
5.0'			21.6%

bottom of boring @ 5.0'

BORING COMPLETED 7-19-79

WATER LEVEL READINGS

ENCOUNTERED none
UPON COMPLETION none
WATER none AFTER 1.5 hr
CAVED AT -
Alternate boring A7a drilled to recover undisturbed samples

BORING NUMBER A8
GROUND SURFACE ELEVATION

Depth	Description	Blows	Moisture
0.2'	topsoil		
	sandy silt, FILL, with quartz fragments & organic matter, moist, brown	2+4+5	
		4+8+8	16.9%
2.8'	fine sandy SILT(ML), moist, brown	3+4+5	26.2%
5.0'			

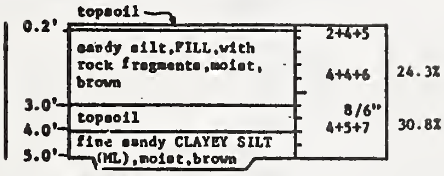
bottom of boring @ 5.0'

BORING COMPLETED 7-19-79

WATER LEVEL READINGS

ENCOUNTERED none
UPON COMPLETION none
WATER none AFTER 3 hrs
CAVED AT -

BORING NUMBER A9
GROUND SURFACE ELEVATION



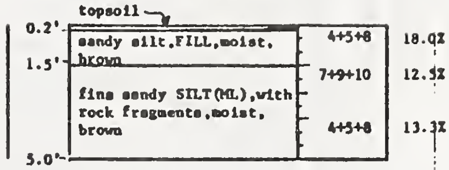
bottom of boring @ 5.0'

BORING COMPLETED 7-19-79

WATER LEVEL READINGS

ENCOUNTERED none
UPON COMPLETION none
WATER none **AFTER** 3 hrs
CAVED AT -

BORING NUMBER A10
GROUND SURFACE ELEVATION



bottom of boring @ 5.0'

BORING COMPLETED 7-19-79

WATER LEVEL READINGS

ENCOUNTERED none
UPON COMPLETION none
WATER - **AFTER** -
CAVED AT -
backfill on completion

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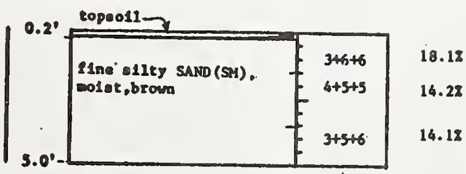
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 5 OF 8
DATE 11-5-79
CONTRACT NO. M79043
ENCL. 3

BORING NUMBER A11
GROUND SURFACE ELEVATION



bottom of boring @ 5.0'

BORING COMPLETED 7-19-79

WATER LEVEL READINGS

ENCOUNTERED none
UPON COMPLETION none
WATER - **AFTER** -
CAVED AT -
backfill on completion

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 6 OF 8
DATE 11-5-79
CONTRACT NO. M79043
ENCL. 3

Sheet 7 of 8
M79043 ENCL 3

GENERAL NOTES

1. Number in right hand column indicate the number of blows required to drive a 2 inch O.D., 1-3/8 inch I.D. sampling spoon through three 6 inch intervals or as indicated, using a 140 pound hammer falling 30 inches, according to ASTM D-1586.

2. Classification of soil is by visual inspection and is in accordance with the unified soil classification system. Symbols in parentheses are estimated Unified Soil Classification Symbols by visual inspection.

3. Estimated groundwater levels indicated by ▼; these are only estimates from available data and may vary with precipitation, porosity of the soil, site topography, etc.

4. Refusal at the surface of rock, boulder, or obstruction is defined as a penetration resistance of 100 blows for 2 inches penetration or less.

5. Boring foreman: R. Stidham

6. Key to abbreviations and symbols:

   P-24" = 3 inch tube sample pressed 24 inches

   R-20" = 20 inch sample recovery

7. Borings made by mechanical auger without use of drilling water.

8. Boring layout was provided by others. No elevations were obtained.

9. The boring logs and related information depict subsurface conditions only at these specific locations and at the particular time when drilled. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the subsurface soil and groundwater conditions at these boring locations.

10. The stratification lines represent the approximate boundary between soil and rock types as determined in the drilling and sampling operation. Some variation may also be expected vertically between samples taken. The soil profile, water level observations and penetration resistances presented on this drawing have been made with reasonable care and accuracy and must be considered only an approximate representation of subsurface conditions to be encountered at the particular location.

11. Percentages to right of boring logs indicate natural moisture content of the soil.

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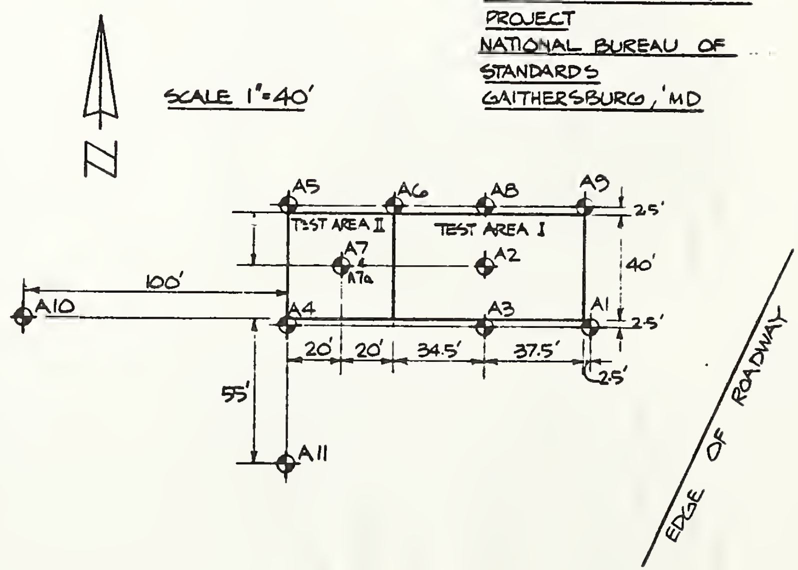
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 8 OF 8
DATE 11-5-79
CONTRACT NO. M 79043 ENCL. 3

BORING LOCATION SKETCH
TEST SITE "A"
MOBILE HOME ANCHORAGE PROJECT
NATIONAL BUREAU OF STANDARDS
GAITHERSBURG, MD

SCALE 1"=40'



SCHNABEL ENGINEERING ASSOCIATES
A PROFESSIONAL CORPORATION
CONSULTING GEOTECHNICAL ENGINEERS

May 29, 1980

JAMES J. SCHNABEL P.E.
ERNEST WINTER P.E.
GERALD C. DAVIT P.E.
FRANK D. GREENHEIM P.E.

5505 CORNELL AVENUE
BETHESDA, MARYLAND 20014
301-652-8922

National Bureau of Standards
Center for Building Technology
Building 226, Room B-168
Washington, D.C. 20234

Project: Testing Services for Mobile Home Anchorage Project, Report for Site "B"
Odenton, Maryland
(M79043)

Attn: Dr. Felix Y. Yokel

Gentlemen:

Submitted herewith are four copies of our report of subsoil exploration and soil laboratory testing for Site "B". These services have been provided under your purchase order NB 79 NAAA 9922 dated April 18, 1979.

Site

The test site for this series of tests was in Odenton, Maryland. Soils encountered were shallow silty sand fills overlying sands of the Potomac Group of the Cretaceous geologic age.

Field Testing

Four soil test borings were drilled by our subsidiary, Foundation Test Service, Inc., to a depth of 10 feet each. Borings were located in a roughly square pattern of approximately 40 feet on a side, but no layout, vicinity map or plans are available. Two undisturbed tube samples of sand were recovered in an alternate boring adjacent to B-2. Your in-situ testing program was conducted in the same general area but we understand it was not within the bounds of the locations drilled due to difficulty with penetrating anchors through very compact soils found at the surface.

Group symbols after the description of soils are in accordance with the Unified Soil Classification System and were based on visual examination of samples. The system used by this firm for identifying soils is included in Enclosure (1), Identification of Soil.

National Bureau of Standards
May 29, 1980
Page Two

The standard penetration resistance, or N value, indicates the penetration resistance in blows per foot of a 2 inch O.D., 1-3/8 inch I.D. sampling spoon driven with a 140 pound hammer falling 30 inches per ASTM D-1586. After an initial set of 6 inches to assure the sampler is in undisturbed material, the number of blows required to drive the sampler an additional 12 inches is usually taken as the N value. The penetration for each 6 inches of drive is shown in the right hand column of the test boring logs.

No groundwater was observed in any of the borings during drilling. Long term water observations were not made.

Soil Laboratory Testing

Soil laboratory test results are summarized in Enclosure (2). Natural moisture content tests and natural density tests were conducted on two tube samples. Sieve analyses were conducted to determine the grain size gradation, and a triaxial compression and direct shear test were performed on remolded specimens to determine strength characteristics of soils.

The test results indicate that the natural soils tested are typically a fine to medium or fine to coarse sand with traces of silt and gravel corresponding to SP and SM classifications according to ASTM D-2487. Visual classifications indicate that soils described as, fine sandy silty clay, fine to coarse gravelly sand, and fine sand are also present.

Natural moisture contents ranged from about 2% to 4%, with natural dry densities between about 92 and 118 pcf. Several natural density and moisture tests were conducted, with soils reconstituted to about the same condition for strength testing. Triaxial compression and direct shear tests yielded $\phi = 31^{\circ}$ and 29° and $c = 400$ psf and $c = 0$, respectively. As requested, a 50 psi back pressure was applied during triaxial testing.

Soil samples remaining will be retained until July 11, 1980 and then discarded unless other disposition is requested.

National Bureau of Standards
May 29, 1980
Page Three

Please call if there are questions concerning this submittal.

Very truly yours,

SCHNABEL ENGINEERING ASSOCIATES, P.A.

J. A. Padgett, MECE
Project Manager

James J. Schnabel, P.E.
State of Maryland

Encls:
(1) Identification of Soil
(2) Summary of Soil Laboratory Tests
Gradation Curves (2)
Triaxial Compression Curve (1)
Direct Shear Curve (1)
(3) Subsoil Investigation Report
Test Boring Report, Sheets 1 - 5

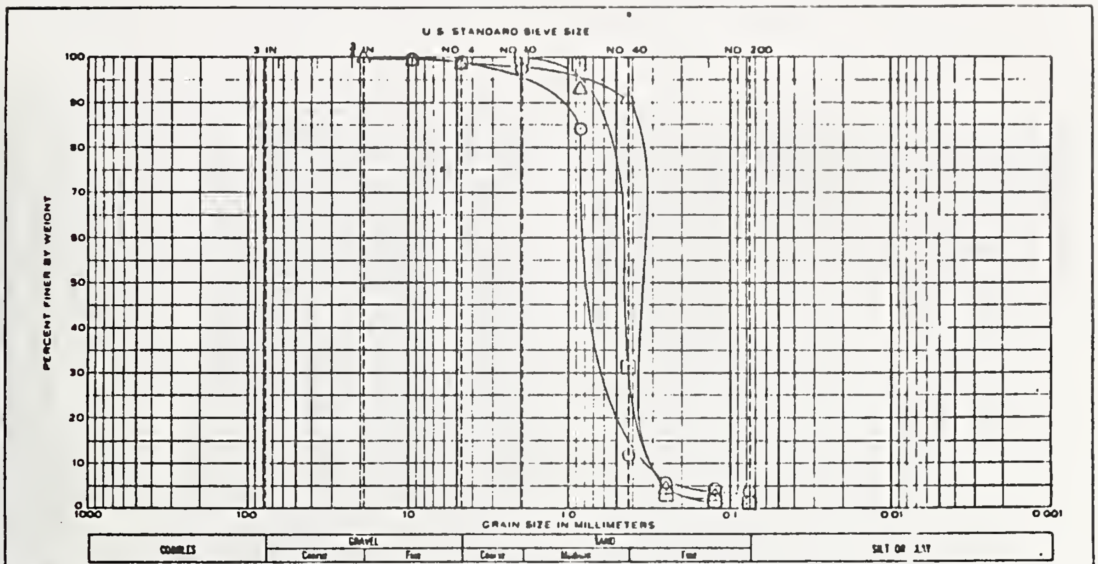
Enclosure (2)
Contract No. M79043

SUMMARY OF SOIL LABORATORY TESTS

Boring No.	Sample Depth	Sample Type	Description of Soil Specimen	Percent Passing No. 200 Sieve	Natural Moisture (%)	Natural Density (PCF) Wet	Natural Density (PCF) Dry	Remolded Moisture (%)	Remolded Density (PCF) Wet	Remolded Density (PCF) Dry	Remolded Soil Parameters	Remarks
B1	6'	Jar	Fine to coarse SAND (SP), trace silt & gravel, brown & white	3.7								See: Gradation Curve
B2	1' to 3'	3" Tube	Fine to coarse SAND (SM), some silt, light brown	26.5	3.3 4.0	108.1 122.5	104.7 117.8					See: Gradation Curve
	3' to 5'	3" Tube	Fine to medium SAND (SP), trace silt, light brown & white	1.8	2.0 2.2 2.1	94.0 96.1 95.5	92.2 94.0 93.6	1.5 3.1 4.7	96.4 95.6 96.7	95.0 92.7 92.3	ø=31° c=0.4 ksf	See: Gradation and Triaxial Compression Curve Notes 6 & 7
	do	do	do	do				1.9 3.1 2.4	94.3 96.7 95.2	92.5 93.7 93.0	ø=29° c=D	See: Direct Shear Curves Note 6
B3	4'	Jar	Fine to coarse SAND (SP), trace silt & gravel, brown	2.2								See: Gradation Curve
B4	2'	Jar	Fine to medium SAND (SP), trace silt, light brown	4.6								See: Gradation Curve

Notes:

1. Soil test in accordance with applicable ASTM Standards.
2. Soil classification symbols are in accordance with Unified Soil Classification System, based on testing indicated and visual identification.
3. Visual identification of samples is in accordance with the system used by this firm.
4. Key to abbreviations: do = ditto, c = apparent cohesion ø = soil friction angle
5. Soil tests were conducted by L. Trulio, T. Charlton & E. Sonnenberg.
6. Specimens remolded to approximate natural density and moisture for strength testing.
7. A 50 psi back pressure was applied to triaxial specimens.



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASSIF	MC	LL	PI
⊙	B1	6'	FINE TO COARSE SAND, TRACE SILT & FINE GRAVEL, BROWN & WHITE	SP	—	—	—
⊡	B2	3'	FINE TO MEDIUM SAND, TRACE SILT, LIGHT BROWN & WHITE	SP	2.2	—	—
△	B3	4'	FINE TO COARSE SAND, TRACE FINE GRAVEL & SILT, BROWN	SP	—	—	—

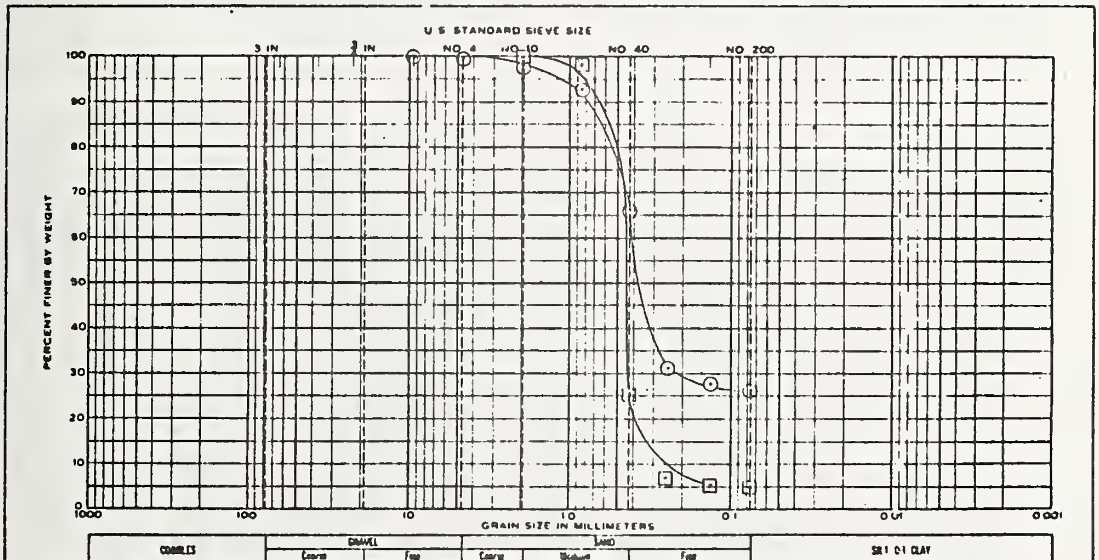
SCHNABEL ENGINEERING ASSOCIATES

GRADATION CURVES

PROJECT NBS MOBILE HOME ANCHORAGE PROJECT

DATE 5-29-80 CONTR. NO M 7204[illegible]

ENCLOSURE (2) SHEET 1 OF [illegible]



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASSIF	MC	LL	PI
⊙	B2	1'	FINE TO COARSE SAND, SOME SILT LIGHT BROWN	SM	3.3	~	~
⊡	B4	2'	FINE TO MEDIUM SAND, TRACE SILT, LIGHT BROWN	SP	4.6	~	~

SCHNABEL ENGINEERING ASSOCIATES

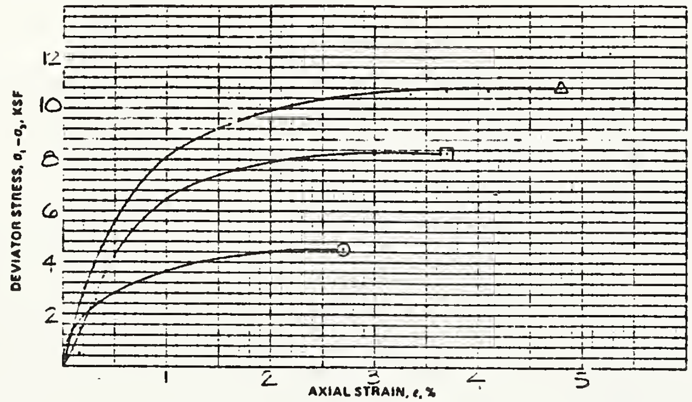
GRADATION CURVES

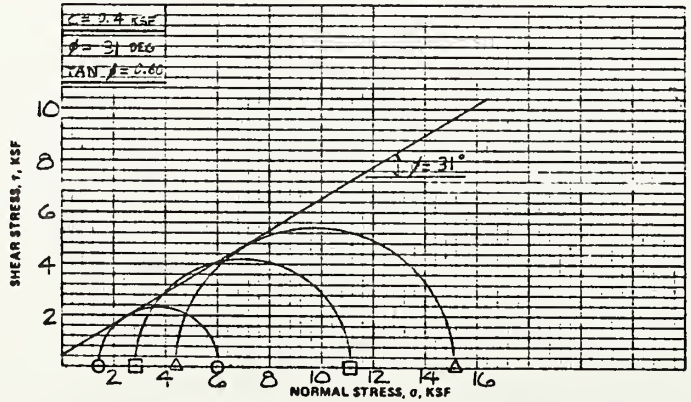
PROJECT NBS MOBILE HOME ANCHORAGE PROJECT

DATE 5-29-80 CONTR. NO M 7[illegible]

ENCLOSURE (2) SHEET 2 OF 4

ENCLOSURE (2)
SHEET 3 OF 4





50 psi BACKPRESSURE APPLIED

DESCRIPTION OF SOIL SAMPLE TESTED							
FINE TO MEDIUM SAND (SP), TRACE SILT, LIGHT BROWN & WHITE							
KEY	BORING NO.	DEPTH FT.	LATERAL PRESS	REMOLDED MOIST. CONT. % INITIAL	REMOLDED MOIST. CONT. % FINAL	REMOLDED DENSITY, PCF DRY	REMOLDED DENSITY, PCF WET
⊙	B2	3'	1.44 KSF	1.5	23.8	93.0	96.4
⊡	B2	3'	2.88 KSF	3.1	23.7	92.7	95.6
△	B2	3'	4.32 KSF	4.7	25.4	92.3	96.7

SCHNABEL ENGINEERING ASSOCIATES

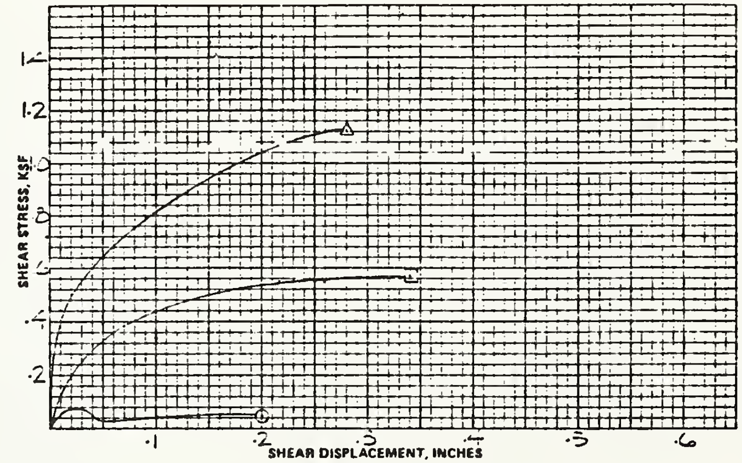
TRIAXIAL COMPRESSION TEST

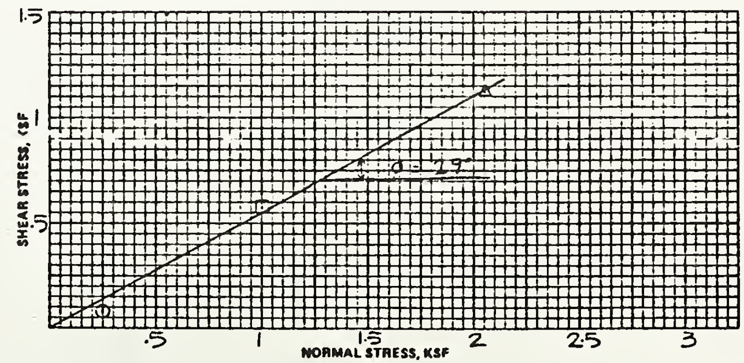
TYPE OF TEST: CONSOLIDATED-UNDRAINED
RATE OF SHEAR: .005"/MIN.

PROJECT: NBS MOBILE HOME ANCHORAGE PROJECT, SITE B

DATE: 5-29-80 CONTR. NO. M 73043

ENCLOSURE (2)
SHEET 4 OF 4





DESCRIPTION OF SOIL SAMPLE TESTED							
FINE TO MEDIUM SAND (SP), TRACE SILT, LIGHT BROWN & WHITE							
KEY	BORING NO.	DEPTH FT.	NORMAL STRESS	REMOLDED MOIST. CONT. % INITIAL	REMOLDED MOIST. CONT. % FINAL	REMOLDED DENSITY, PCF DRY	REMOLDED DENSITY, PCF WET
⊙	2B	3'	.25 KSF	1.9		92.5	94.3
⊡	2B	3'	1 KSF	2.4	27.2	93.0	95.2
△	2B	3'	2 KSF	3.1		93.7	96.7

SCHNABEL ENGINEERING ASSOCIATES

DIRECT SHEAR TEST

TYPE OF TEST: CONSOLIDATED-UNDRAINED
RATE OF SHEAR: .04" PER MINUTE

PROJECT: NBS MOBILE HOME ANCHORAGE PROJECT

DATE: 5-29-80 CONTR. NO.: M 73043

Enclosure (3)
Contract No. M79043

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 1 OF 5
DATE MAY 29 1980
CONTRACT NO. M 79043
Encl. (3)

SUBSOIL INVESTIGATION REPORT

Test Boring Report

Description of Subsoil Investigation Procedures

1. Test Borings

Continuous Flight (Mechanical) Augers

Test borings are advanced using 4-1/2 in continuous flight solid augers which rotate into the soil and bring cuttings to the surface. The augers are withdrawn from the borehole at each sampling depth, and samples are obtained using standard methods. Augers are used only when the borehole sidewalls will stand without support. No water is introduced into the boring using this procedure.

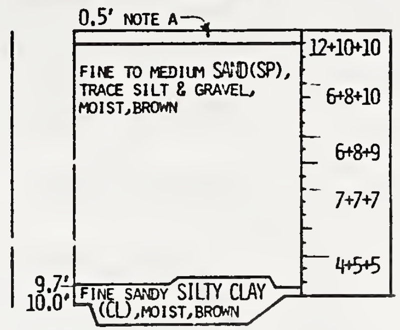
2. Standard Penetration Tests

Testing is performed by driving a 2 in O.D., 1-3/8 in I.D. sampling spoon through three 6 in intervals or as indicated, using a 140 lb hammer falling 30 in, according to ASTM D-1586.

3. Boring Locations and Grades

The borings were drilled at locations selected by others as noted on the Boring Location Sketch. Elevations were not provided.

BORING NUMBER B1
GROUND SURFACE ELEVATION —



BOTTOM OF BORING @ 10.0'

NOTE A: SILTY SAND, FILL
WITH GRAVEL, MOIST, BROWN

BORING COMPLETED 10-30-80

WATER LEVEL READINGS

ENCOUNTERED NONE
UPON COMPLETION NONE TO 6.5' (A)
WATER — AFTER —
CAVED AT —
BACKFILLED ON COMPLETION

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 2 OF 5
DATE MAY 29 1980
CONTRACT NO. M 78043
ENCL. (3)

BORING NUMBER B2
GROUND SURFACE ELEVATION —

Depth	Description	Blows	Sample
	NOTE A	18+19+17	
1.0'	FINE TO COARSE SAND(SM), SOME SILT, MOIST, BROWN	8+9+10	3" P-24" R-24"
3.0'	FINE TO MEDIUM SAND(SP), TRACE SILT, MOIST, LIGHT BROWN	4+7+8	3" P-24" R-24"
		6+9+10	
8.0'	FINE TO COARSE GRAVELLY SAND(SP), TRACE SILT, MOIST, BROWN	5+8+10	
10.0'			

BOTTOM OF BORING @ 10.0'

NOTE A: SILTY SAND, FILL WITH GRAVEL, MOIST, BROWN

BORING COMPLETED 10-30-80

WATER LEVEL READINGS

ENCOUNTERED NONE
UPON COMPLETION NONE TO 6.0' (A)
WATER — AFTER —
CAVED AT —
BACKFILLED ON COMPLETION

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 3 OF 5
DATE MAY 29 1980
CONTRACT NO. M 78043
ENCL. (3)

BORING NUMBER B3
GROUND SURFACE ELEVATION —

Depth	Description	Blows
0.5'	NOTE A	
	FINE TO COARSE SAND(SP), TRACE GRAVEL & SILT, MOIST, BROWN	22+20+26
		11+10+11
		4+6+8
6.0'	FINE TO COARSE GRAVELLY SAND(SP-SM), TRACE CLAYEY SILT, MOIST, BROWN	10+12+10
8.0'	FINE SAND(SP), MOIST, BROWN	4+5+6
10.0'		

BOTTOM OF BORING @ 10.0'

NOTE A: SILTY SAND, FILL WITH GRAVEL, MOIST, BROWN

BORING COMPLETED 10-30-80

WATER LEVEL READINGS

ENCOUNTERED NONE
UPON COMPLETION NONE TO 5.0' (A)
WATER — AFTER —
CAVED AT —
BACKFILLED ON COMPLETION

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 4 OF 5
DATE MAY 29 19[illegible]
CONTRACT NO. M 79043
Encl. (3)

BORING NUMBER B4
GROUND SURFACE ELEVATION —

Depth	Description	Blows
	FINE TO MEDIUM SAND(SP), TRACE SILT, MOIST, BROWN	10+15+14
		5+6+8
		5+6+7
6.0'	FINE TO COARSE SAND(SP-SM), SOME GRAVEL, TRACE CLAYEY SILT, MOIST, BROWN	8+9+10
8.5'	FINE SAND(SM), SOME CLAYEY SILT, MOIST, BROWN	7+11+11
10.0'		

BOTTOM OF BORING @ 10.0'

BORING COMPLETED 10-30-80

WATER LEVEL READINGS

ENCOUNTERED NONE
UPON COMPLETION NONE TO 6.0' (A)
WATER — AFTER —
CAVED AT —
BACKFILLED ON COMPLETION

GENERAL NOTES

Number in right hand column indicates the number of blows required to drive a 2 inch O.D., 1-3/8 inch I.D. sampling spoon through three 6 inch intervals or as indicated, using a 140 pound hammer falling 30 inches, according to ASTM D-1586.

Classification of soil is by visual inspection and is in accordance with the unified soil classification system. Symbols in parentheses are estimated Unified Soil Classification Symbols by visual inspection.

Boring foreman: B. Spierenburg

Key to abbreviations and symbols:

3 P-24" = 3 inch tube sample pressed 24 inches
R-20" = 20 inch sample recovery

(A) = Augers pulled

Borings made by hollow stem auger.

Boring locations selected in the field. No layout plans available. No elevations were obtained.

The stratification lines represent the approximate boundary between soil and rock types as determined in the drilling and sampling operation. Some variation may also be expected vertically between samples taken. The soil profile, water level observations and penetration resistances presented on this drawing have been made with reasonable care and accuracy and must be considered only an approximate representation of subsurface conditions to be encountered at the particular location.

SCHNABEL ENGINEERING ASSOCIATES

A PROFESSIONAL CORPORATION

CONSULTING GEOTECHNICAL ENGINEERS

September 22, 1980

JAMES J. SCHNABEL P. E.
ERNEST WINTER P. E.
GERALD C. DAVIT P. E.
FRANK D. GREFSHEIM P. E.

4909 CORDELL AVENUE
BETHESDA, MARYLAND 20014
301-652-8922

National Bureau of Standards
Center for Building Technology
Building 226, Room B-168
Washington, D.C. 20234

Project: Testing Services for Mobile Home, Anchorage Project, Report for Site "C", Brown Station Road, Upper Marlboro, Maryland (Our Contract M79043)

Attn: Dr. Felix Y. Yokel

Gentlemen:

Submitted herewith are four copies of our report of subsoil exploration and soil laboratory testing for Site "C". These services have been provided under your Purchase Order NB 79 NAAA 9922, dated April 18, 1979.

Site

The test site for this series of tests was a wooded tract along Brown Station Road near Upper Marlboro, Maryland. Soils encountered were silty clays overlying sands, both believed to be river terrace deposits of the Western Branch of the Patuxent River. These soils are believed to be of Pleistocene geologic age.

Field Testing

Four soil test borings were drilled by our subsidiary, Foundation Test Service, Inc., to a depth of about 5 to 5.5 feet each. Two 3 inch undisturbed tube samples of silty clay were recovered in Borings 1C and 2C. Borings were located in a roughly diamond pattern, with a long axis of about 75 feet, generally perpendicular to Brown Station Road. No layout plan is available but we assume as-drilled locations were recorded in the field by your personnel during field pull out testing. A vicinity site plan indicating the general area of work was previously provided to you.

Group symbols after the description of soils are in accordance with the Unified Soil Classification System and were based on visual examination of samples. The system used by this firm for identifying soils is included in Enclosure (1), Identification of Soil.

National Bureau of Standards
September 22, 1980
Page Two

The standard penetration resistance, or N value, indicates the penetration resistance in blows per foot of a 2 inch O.D., 1-3/8 inch I.D. sampling spoon driven with a 140 pound hammer falling 30 inches per ASTM D-1586. After an initial set of 6 inches to assure the sampler is in undisturbed material, the number of blows required to drive the sampler an additional 12 inches is usually taken as the N value. The penetration for each 6 inches of drive is shown in the right hand column of the test boring logs.

Groundwater was observed in all of the borings upon completion of drilling at depths of about 1 to 4 feet. Long term water observations were not made.

Soil Laboratory Testing

The laboratory testing program was assigned by your office. Soil laboratory test results are summarized in Enclosure (2). Natural moisture content, natural density and specific gravity tests were conducted on two tube samples. Sieve analyses were conducted to determine the grain size gradation, and a triaxial compression and unconfined compression test were performed to determine strength characteristics of soils.

The test results indicate that the natural soil tested are typically a silty clay or fine sandy silty clay with trace to some fine or fine to coarse sand, with organic matter, corresponding to CL classification according to ASTM D-2487. Visual classifications indicate that soils described as fine silty sand are also present.

Natural moisture contents ranged from about 23% to 40% with natural dry densities between about 81 and 100 pcf. Specific gravities of 2.56 and 2.54 were recorded. An unconsolidated, undrained triaxial compression test yielded $\phi = 19^{o}$ and c = 700 psf. An unconfined compression test recorded an unconfined compressive strength of q_u = 1,933 psf at 6.7% strain.

Soil samples remaining will be retained until November 6, 1980 and then discarded unless other disposition is requested.

We appreciate the opportunity to be of continued service for this project. Please call if there are questions concerning this submittal.

Very truly yours,

SCHNABEL ENGINEERING ASSOCIATES, P.A.

J. A. Padgett, MECE
Project Manager

James J. Schnabel, P.E.
State of Maryland

STATE OF MARYLAND
James Joseph Schnabel
REGISTERED PROFESSIONAL ENGINEER

Encls:
(1) Identification of Soil
(2) Summary of Soil Laboratory Tests
Gradation Curves (2)
Triaxial Compression Curve (1)
Unconfined Compression Curve (1)
(3) Subsoil Investigation Report
Test Boring Report, Sheets 1 - 5

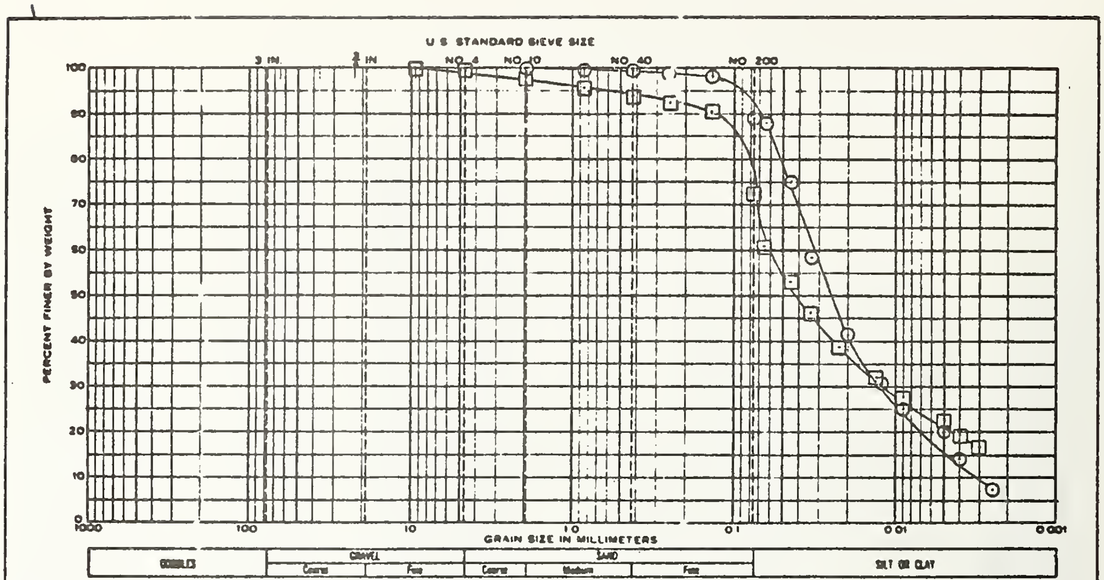
Enclosure (2)
Contract No. M79043

SUMMARY OF SOIL LABORATORY TESTS

Boring No.	Sample Depth	Sample Type	Description of Soil Specimen	Percent Passing No. 200 Sieve	Atterberg Limits LL	PL	PI	Natural Moisture (%)	Natural Density (pcf) Wet	Dry	Specific Gravity	Soil Parameters	Remarks
1C	2'-4'	3" Tube	SILTY CLAY (CL), trace fine sand with organic matter, gray	89.1	39	20	19	40.2	113.6	81.1	2.56	q_u=1933 psf @ ϵ =6.7% Brittle Failure	See: Gradation and Unconfined Compression Curves
2C	2'-4'	3" Tube	SILTY CLAY (CL), some fine to coarse sand with organic matter, gray	72.2	35	20	15	23.1	123.5	100.3	2.54	$\emptyset = 19^{o}$ c = 0.7 ksf	See: Gradation and Triaxial Compression Curves
3C	2'	Jar	SILTY CLAY (CL), some fine sand with organic matter, gray	82.6	38	22	16	27.6	-	-	-	-	-
4C	3.5'	Jar	Fine sandy CLAYEY SILT (CL-ML), with organic matter, gray	61.1	27	20	7	23.9	-	-	-	-	See: Gradation Curve

Notes:

1. Soil test in accordance with applicable ASTM Standards.
2. Soil classification symbols are in accordance with Unified Soil Classification System, based on testing indicated and visual identification.
3. Visual identification of samples is in accordance with the system used by this firm.
4. Key to abbreviations: LL=Liquid Limit; PL=Plastic Limit; PI=Plasticity Index; q_u=Unconfined Compressive Strength; c=Apparent Cohesion; Ø=Soil Friction Angle; ϵ =Axial Strain.
5. Soil tests were conducted by L. Trulio, T. Charlton and E. Sonnenberg.



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASSIF	M.C.	LL	PI
⊙	1C	2-4'	SILTY CLAY, TRACE FINE SAND WITH ORGANIC MATTER, GRAY	CL	40.2	33	19
⊡	2C	2-4'	SILTY CLAY, SOME FINE TO COARSE SAND WITH GRAVEL & ORGANIC MATTER, GRAY	CL	23.1	35	15

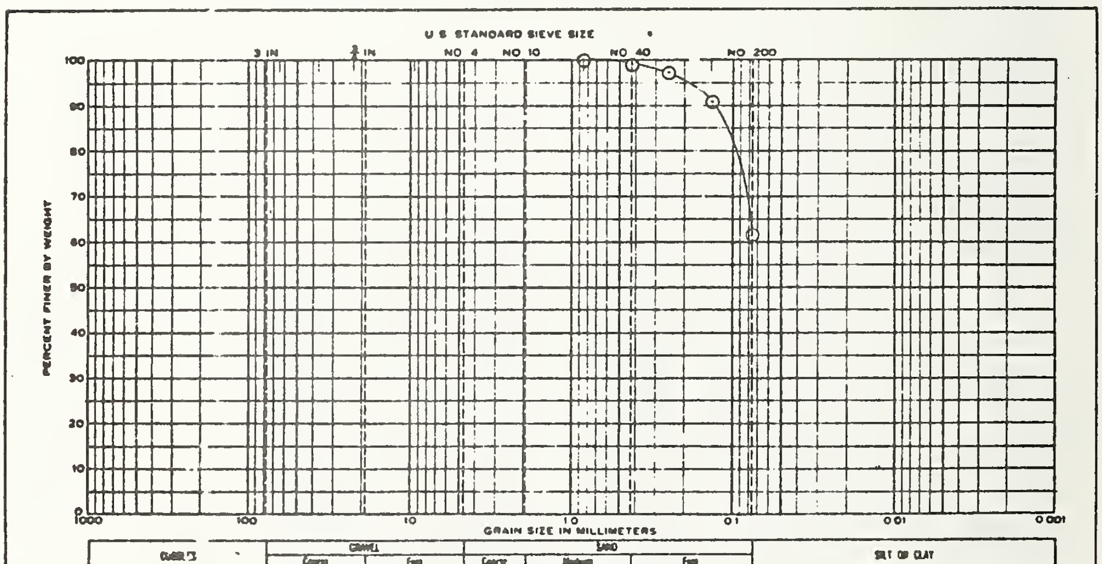
SCHNABEL ENGINEERING ASSOCIATES

GRADATION CURVES

PROJECT NBS MOBILE HOME ANCHORAGE PROJECT

DATE SEPT. 22, 1980 CONTR. NO. M79043

SHEET 1 OF 4



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASSIF	M C	LL	PI
⊙	4C	3-5	FINE SANDY CLAYEY SILT WITH ORGANIC MATTER, GRAY	CL-ML	23.9	27	7

SCHNABEL ENGINEERING ASSOCIATES

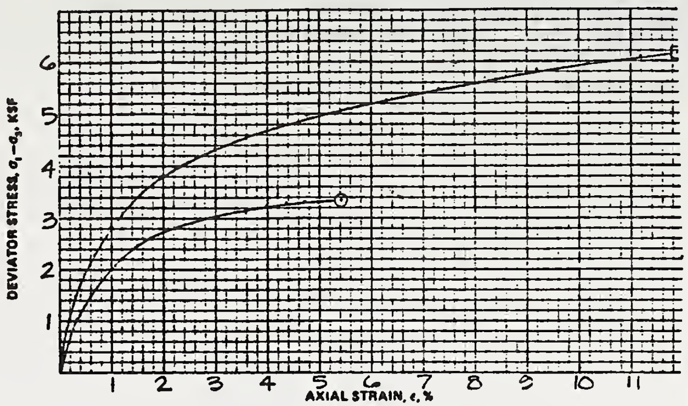
GRADATION CURVES

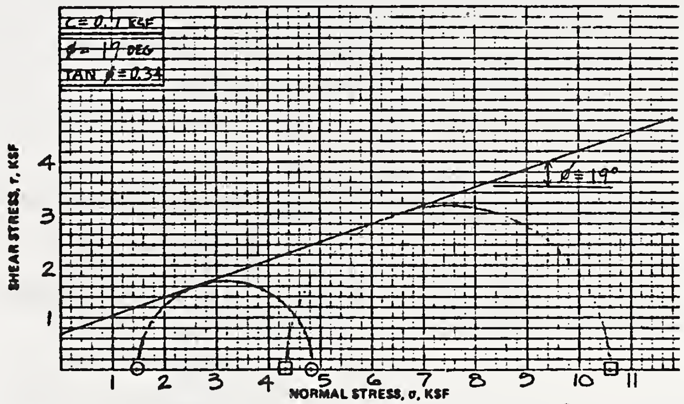
PROJECT NBS MOBILE HOME ANCHORAGE PROJECT

DATE SEPT. 22, 1980 CONTR. NO. M79043

SHEET 2 OF 4

SHEET 3 of





DESCRIPTION OF SOIL SAMPLE TESTED

SILTY CLAY (CL), SOME FINE TO COARSE SAND WITH GRAVEL & ORGANIC MATTER, GRAY

KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST. CONT. % INITIAL	MOIST. CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
⊙	2C	2-4	1.44 KSF	22.3	32.4	97.8	119.6
⊡	1C	2-4	4.32 KSF	28.6		92.2	118.6

SCHNABEL ENGINEERING ASSOCIATES

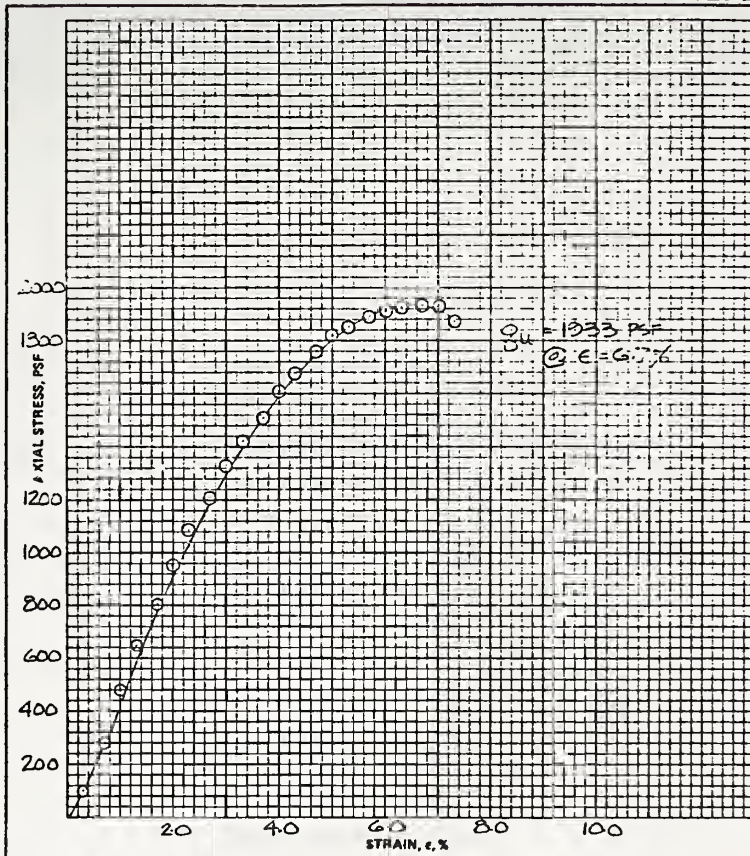
TRIAXIAL COMPRESSION TEST

TYPE OF TEST: UNCONSOLIDATED-UNDRAIN
RATE OF SHEAR: 0.05"/MIN.

PROJECT: NBS MOBILE HOME ANCHORAGE PROJECT

DATE: SEPT. 22, 1980 CONTR. NO.: M 79

SHEET 4 OF 4



DESCRIPTION OF SOIL SAMPLE TESTED

SILTY CLAY (CL) TRACE FINE SAND WITH ORGANIC MATTER, GRAY

BORING NO.: 1C		DEPTH, FT.: 2'-4'
MOIST. CONT. %	WET DENSITY, PCF	DRY DENSITY, PCF
40.2	113.6	81.1

SCHNABEL ENGINEERING ASSOCIATES

UNCONFINED COMPRESSION TEST

PROJECT: NBS MOBILE HOME ANCHORAGE PROJECT

DATE: SEPT. 22, 1980 CONTR. NO. M 79043

Enclosure (3)
Contract No. M79043

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 1 OF 5
DATE SEPT. 22, 1980
CONTRACT NO. M 79043
ENCL. (3)

SUBSOIL INVESTIGATION REPORT

Test Boring Report
Borings 1C - 4C, Sheets 1 - 4
General Notes, Sheet 5

Description of Subsoil Investigation Procedures:

1. Test Borings Using Continuous Flight (Mechanical) Augers

Test borings were advanced using 4-1/2 inch continuous flight solid augers which rotate into the soil and bring cuttings to the surface. The augers are withdrawn from the borehole at each sampling depth, and samples are obtained using standard methods. No water was introduced into the boring using this procedure.

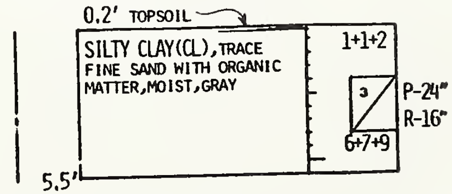
2. Standard Penetration Tests

Testing is performed by driving a 2 inch O.D., 1-3/8 I.D. sampling spoon through three 6 inch intervals or as indicated, using a 140 pound hammer falling 30 inches, according to ASTM D-1586.

3. Boring Locations and Grades

The borings were drilled at locations selected in the field. No plans were available for surveying and elevations and were not obtained.

BORING NUMBER 1C
GROUND SURFACE ELEVATION



BOTTOM OF BORING @ 5.5'

BORING COMPLETED 11-21-79

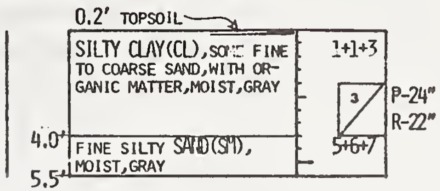
WATER LEVEL READINGS

ENCOUNTERED NONE
UPON COMPLETION 1.5'
WATER — AFTER —
CAVED AT —

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 2 OF 5
DATE SEPT 22, 1980
CONTRACT NO. M 79043
ENCL. (3)

BORING NUMBER 2C
GROUND SURFACE ELEVATION



BOTTOM OF BORING @ 5.5'

BORING COMPLETED 11-21-79

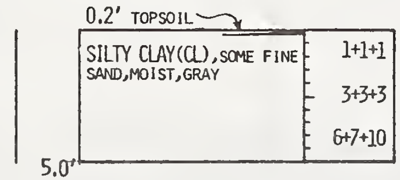
WATER LEVEL READINGS

ENCOUNTERED NONE
UPON COMPLETION 1.0'
WATER — AFTER —
CAVED AT —

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 3 OF 5
DATE SEPT 22, 1980
CONTRACT NO. M 72043
ENCL. (3)

BORING NUMBER 3C
GROUND SURFACE ELEVATION



BOTTOM OF BORING @ 5.0'

BORING COMPLETED 11-21-79

WATER LEVEL READINGS

ENCOUNTERED NONE
UPON COMPLETION 4.2'
WATER — AFTER —
CAVED AT —

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

SHEET NO. 4 OF 5
DATE SEPT. 22, 1980
CONTRACT NO. M 79043
ENCL. (3)

BORING NUMBER 4C
GROUND SURFACE ELEVATION

Depth	Description	Blows
0.2'	TOPSOIL	
	SILTY CLAY(CL),TRACE FINE SAND,MOIST,GRAY	1+1+1
2.0'	FINE SANDY SILTY CLAY (CL),MOIST,GRAY	1+1+1
		1+2+2
5.0'		

BOTTOM OF BORING @ 5.0'

BORING COMPLETED 11-21-79

WATER LEVEL READINGS

ENCOUNTERED NONE
UPON COMPLETION 1.0'
WATER — AFTER —
CAVED AT —

Sheet 5 of 5
Enclosure (3)
Contract No. M79043

GENERAL NOTES

1. Number in right hand column indicates the number of blows required to drive a 2 inch O.D., 1-3/8 inch I.D. sampling spoon through three 6 inch intervals or as indicated, using a 140 pound hammer falling 30 inches, according to ASTM D-1586.

2. Classification of soil is by visual inspection and is in accordance with the Unified Soil Classification System. Symbols in parentheses are estimated Unified Soil Classification Symbols by visual inspection.

3. Boring foreman: R. Stidham

4. Key to abbreviations and symbols:

   3 — P-24" = 3 inch tube sample pressed 24 inches
   R-20" = 20 inch sample recovery

5. Borings made by mechanical auger (without the use of drilling water).

6. Boring locations selected in the field. No layout plans available. No elevations were obtained.

7. The stratification lines represent the approximate boundary between soil and rock types as determined in the drilling and sampling operation. Some variation may also be expected vertically between samples taken. The soil profile and penetration resistances presented on this drawing have been made with reasonable care and accuracy and must be considered only an approximate representation of subsurface conditions to be encountered at the particular location.

APPENDIX B

Test Results

B.1. Introduction

The test results are presented in a series of six tables. The data in the tables were taken from x-y plots produced electronically in the field, except that in tests ST1 to ST12 the data were recorded manually. Anchors of several makes were used in the study. Since it was not the intent of this study to compare the performance of different products, anchors of different makes are identified by letters only. The swivel anchors were inserted by a percussion tool. The time it took to install the anchors is identified in the footnotes.

B.2 Symbols Used in the Tables

Static test results:

Test Number Designations:

ST = Silty Site

SD = Sandy Site

C = Clay Site

Test Location:

Coordinates in location maps in Appendix A

Anchor Type:

H-6 = 6-inch single helix anchor

D-4 = 4-inch double helix anchor

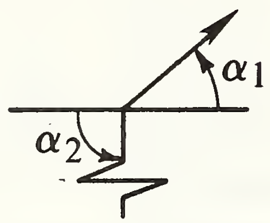
H-3 = 3-inch single helix anchor

P-10 = 10 x 1 3/4 in. pipe anchor

P-6 = 6 1/2 x 1 1/4 in. pipe anchor

AH-6 = 6-inch arrowhead anchor

Anchor Inclination:



Loading:

SM = Static monotonic

SUR = Static monotonic with several unloading and reloading cycles

CP = Creep test

Soil Condition:

M = Moist

W = Wet

S = Submerged

Other Symbols:

STP = Soil Test Probe reading, in-lb

T_i = Installation torque, ft-lb

P_{2v} = Load at 2-inch vertical displacement, lb

P_{4H} = Load at 4-inch horizontal displacement, lb

Q_u = Ultimate load capacity, lb

Δ_u = Anchor head displacement in the direction of pull at Q_u, in

R_{85} = Reloading modulus at 85 percent of Q_u, lb/in

P_c = Cyclic load, lb

n = Number of cycles

Δ_1, Δ_{10}, Δ_{100}, Δ_u = total anchor head displacement in 1st, 10th, 100th and last cycle

R_{10} = Reloading modulus in 10th cycle, lb/in

Q_u' = Ultimate load capacity determined by anchor pullout after completion of cyclic tests

Table B.1. Static Test Results on Site A (Silty Soils)

Test No.	Test Location	Anchor Type	Make	Shaft Size (in)	Pull	α_1 (degrees)	α_2 (degrees)	Depth	Loading	Soil Conditions	STP Reading (in/lb)	Test Results T_1 (ft/lb)	P_{2v} (lbs)	P_{4h} (lbs)	Q_u (lbs)	Δ_u (in)	R_{85} (lb/in)	Notes
ST-1	1-C	H-6	A	3/4	A	90	90	4'	SM	M	-	70	5250	-	6000	3.95	-	-
ST-2	1-E	H-6	A	3/4	A	90	90	4'	SM	M	-	-	5750	-	6000	2.30	-	-
ST-3	G-1	H-6	B	3/4	A	90	90	4'	SM	M	175	90	3000	-	4600	6.00	-	Non-symmetrical radial crack pattern (longest of about 46") around embedded anchor.
ST-4	J-9	H-6	B	3/4	A	90	90	4'	SM	M	175	125	3500	-	5000	8.00	-	Non-symmetrical radial crack pattern (longest of about 26") around embedded anchor.
ST-5	A-5	H-6	A	3/4	A	90	90	4'	SM	M	275	120	4500	-	5000	3.66	-	Helix broke off at 5000 lbs.
ST-6	A-3	H-6	A	3/4	A	90	90	4'	SM	M	250	120	-	-	4300	12.00	-	-
ST-7	A-1	H-6	C	3/4	A	90	90	4'	SM	M	200	130	3500	-	5300	12.00	-	-
ST-8	C-1	H-6	C	3/4	A	90	90	4'	SM	M	210	125	4250	-	5500	7.00	-	-
ST-9	E-1	D-4	A	3/4	A	90	90	2'6"	SM	M	200	60	-	-	2250	2.50	-	12" diameter mound of soil around embedded anchor.
ST-10	G-1	D-4	A	3/4	A	90	90	2'6"	SM	M	260	75	-	-	3050	2.50	-	12" diameter mound of soil around embedded anchor.
ST-11	J-1	D-4	C	3/4	A	90	90	2'9"	SM	M	270	65	2400	-	3000	2.50	-	18" diameter mound (4" hump) of soil around embedded anchor.
ST-12	J-3	D-4	C	3/4	A	90	90	2'9"	SM	M	200	75	-	-	2500	2.50	-	1. 12" diameter mound of soil around embedded anchor. 2. 4" diameter cylinder of soil pulled out with anchor.
ST-13	J-5	H-6	B	5/8	A	90	90	4'	SM	M	225	135	4800	-	5200	6.50	-	First test conducted using X-Y plotter.
ST-14	J-7	H-6	B	3/4	A	90	90	4'	CP	M	-	140	4500	-	5000	4.00	-	Creep Test.
ST-15	H-9	D-4	B	3/4	A	90	90	2'9"	SM	M	-	140	2400	-	2500	1.85	-	18" diameter mound of soil around embedded anchor.
ST-16	F-9	D-4	B	3/4	A	90	90	2'9"	SM	M	-	120	3100	-	3100	2.00	-	18" diameter mound of soil around embedded anchor.
ST-17	B-4	H-6	B	3/4	A	90	90	4'	SM	M	-	100	3900	-	5000	13.50	-	-
ST-18	A-4	H-3	A	3/8	A	90	90	2'6"	SM	M	-	30	1800	-	2050	3.50	-	-
ST-19	A-8	H-3	A	3/8	A	90	90	2'6"	SM	M	-	10	950	-	1300	9.50	-	-
ST-20	C-8	H-3	A	3/8	A	90	90	2'6"	SM	M	-	15	1450	-	1700	4.00	-	-
ST-21	B-9	H-3	A	3/8	A	90	90	2'5"	SM	M	-	15	1300	-	1500	5.50	-	Ground is very wet with water on surface.
ST-22	D-9	H-3	A	3/8	A	90	90	2'5"	SM	M	-	20	1350	-	1700	5.50	-	Ground is very wet with water on surface.
ST-23	W-1	H-6	A	3/4	A	90	90	4'	SM	S	125	150	1250	-	2800	15.00	-	-
ST-24	W-2	H-6	B	5/8	A	90	90	4'	SM	S	90	70	1350	-	3300	14.70	-	-
ST-25	W-3	H-6	C	3/4	A	90	90	4'	SM	S	450	160	3000	-	4200	6.00	-	-
ST-26	W-4	H-6	A	3/4	A	90	90	4'	SM	S	130	70	3100	-	5700	13.00	-	-
ST-27	W-5	H-6	B	5/8	A	90	90	4'	SM	S	140	115	3100	-	3700	9.00	-	-
ST-28	W-6	H-6	C	5/8	A	90	90	4'	SM	S	200	100	2900	-	4200	10.50	-	-
ST-29	W-7	D-4	A	3/4	A	90	90	2'6"	SM	S	-	80	2900	-	3150	2.90	-	-
ST-30	W-8	D-4	B	5/8	A	90	90	2'9"	SM	S	-	90	3150	-	3200	1.50	-	-
ST-31	W-11	D-4	B	3/4	A	90	90	2'9"	SM	S	-	260	2950	-	3250	2.50	-	-
ST-32	W-12	D-4	C	3/4	A	90	90	2'9"	SM	S	-	80	2500	-	2500	2.00	-	-
ST-33	W-9	D-4	C	5/8	A	90	90	2'9"	SM	S	-	60	1850	-	1900	1.50	-	-
ST-34	W-10	D-4	A	3/4	A	90	90	2'6"	SM	S	-	135	1850	-	1950	0.80	-	-
ST-35	W-13	H-3	A	3/8	A	90	90	2'6"	SM	S	-	45	1200	-	1400	3.80	-	-
ST-36	W-14	H-3	A	3/8	A	90	90	2'6"	SM	S	-	70	2050	-	2090	1.80	-	-
ST-37	W-15	H-3	A	3/8	A	90	90	2'6"	SM	S	-	30	900	-	1000	3.00	-	-
ST-38	W-16	H-3	A	3/8	A	90	90	2'6"	SM	S	-	100	2400	-	2625	3.20	-	-
ST-39	W-17	H-3	A	3/8	A	90	90	2'6"	SM	S	-	30	1800	-	2360	3.96	-	-
ST-40	J-2	H-6	A	3/4	A	90	90	1'	SM	M	75	45	600	-	700	1.30	-	1. 26" diameter mound of soil around embedded anchor at 2.5" displacement and 600 lb. 2. 1/4" hump at 625 lb.
ST-41	H-2	H-6	A	3/4	A	90	90	1'	SM	M	75	60	700	-	800	0.35	-	1. 1/4" hump at 750 lb. and 0.8" displacement. 2. 24" diameter mound of soil around embedded anchor at pullout.
ST-42	G-2	H-6	A	3/4	A	90	90	1'	SM	M	-	60	980	-	1020	1.60	-	-
ST-43	F-2	H-6	A	3/4	A	90	90	2'	SM	M	160	110	2750	-	3120	1.20	-	1. 1/4" hump at 3100 lb. 2. 38" diameter mound of soil around embedded anchor.

Table B.1. Static Test Results on Site A (Silty Soils) (continued)

Test No.	Test Location	Anchor Type	Make	Shaft Size (in)	Pull	α_1 (degrees)	α_2 (degrees)	Depth	Loading	Soil Conditions	STP Reading (in/lb)	Test Results: T_1 (ft/lb)	P_{2v} (lbs)	P_{4h} (lbs)	Q_u (lbs)	Δ_u (in)	R_{85} (lb/in)	Notes
ST-44	E-2	H-6	A	3/4	A	90	90	2'	SM	M	190	85	3150	-	3300	1.75	-	1. 1/4" hump at 3300 lb. 2. 36" diameter mound of soil around embedded anchor.
ST-45	D-2	H-6	A	3/4	A	90	90	2'	SM	M	-	110	3000	-	3180	1.65	-	1. 1/4" hump at 3100 lb. 2. 36" diameter mound of soil around embedded anchor at 9" pullout.
ST-46	B-2	H-6	A	3/4	A	90	90	3'	SM	M	280	95	3900	-	5250	6.00	-	1. 1" hump at 12-1/2" pullout. 2. 36" to 40" diameter mound of soil around embedded anchor.
ST-47	B-3	H-6	A	3/4	A	90	90	3'	SM	M	250	120	3900	-	5600	9.34	-	42" diameter mound of soil (1" hump) around embedded anchor.
ST-48	C-3	H-6	A	3/4	A	90	90	3'	SM	M	-	132	4000	-	4800	6.92	-	1. 1/2" hump at 4500 lb. 2. 32" diameter mound of soil around embedded anchor.
ST-49	D-3	H-6	A	3/4	A	15	15	4'	SM	M	-	85	1210	1180	1280	2.75	-	1. Soil was quite wet due to heavy rain fall in the weekend. 2. Soil mounded up along the axis of anchor (about 22" wide and 32" long).
ST-50	E-3	H-6	A	3/4	A	15	15	4'	SM	M	250	50	1500	1490	1610	2.60	-	Soil mounded up along the axis of the embedded anchor.
ST-51	F-3	H-6	A	3/4	A	15	15	4'	SM	M	450	120	1375	1450	1450	2.40	-	-
ST-52	G-3	H-6	A	3/4	A	30	30	4'	SM	M	320	50	3380	2950	3420	3.80	-	24" by 24" mound of soil around embedded anchor.
ST-53	H-3	H-6	A	3/4	A	30	30	4'	SM	M	325	85	2960	2700	3120	1.75	-	24" by 24" mound of soil around embedded anchor.
ST-54	J-4	H-6	A	3/4	A	30	30	4'	SM	M	260	70	1920	1990	2090	1.85	-	28" by 40" mound of soil around embedded anchor.
ST-55	H-4	H-6	A	3/4	A	45	45	4'	SM	M	330	250	4160	2850	4220	3.10	-	1. Anchor was raised vertically about 2" at 5000 lbs. 2. 32" by 32" mound of soil around embedded anchor.
ST-56	G-4	H-6	A	3/4	A	45	45	4'	SM	M	-	-	3900	3750	4000	3.40	-	1. Soil was very wet after heavy rain. 2. 30" by 30" mound of soil around embedded anchor.
ST-57	E-4	H-6	A	3/4	A	45	45	4'	SM	M	-	-	4350	4400	4520	4.30	-	24" by 30" mound of soil around embedded anchor.
ST-58	D-4	H-6	A	3/4	A	60	60	4'	SM	M	-	150	4450	5090	5280	9.55	-	24" by 42" mound of soil around embedded anchor.
ST-59	C-4	H-6	A	3/4	A	60	60	4'	SM	M	260	90	3740	4200	4300	10.80	-	20" by 24" mound of soil around embedded anchor.
ST-60	B-5	H-6	A	3/4	A	60	60	4'	SM	M	250	100	3800	3870	4110	9.25	-	24" by 24" mound of soil around embedded anchor.
ST-61	C-5	D-4	B	3/4	A	60	60	2'9"	SM	M	265	150	2140	1110	2610	1.02	-	18" to 24" diameter mound of soil around embedded anchor.
ST-62	D-5	D-4	B	3/4	A	60	60	2'9"	SM	M	260	140	2320	1000	2700	1.40	-	1. Soil was very wet after rain. 2. 20" diameter mound of soil around embedded anchor.
ST-63	E-5	D-4	B	3/4	A	60	60	2'9"	SM	M	-	120	2400	1400	2720	1.70	-	1. Soil was wet following rain. 2. 18" diameter mound of soil around embedded anchor.
ST-64	F-5	H-3	A	7/16	A	45	45	2'6"	SM	M	325	10	1050	1130	1130	5.60	-	Soil was very wet following rain.
ST-65	G-5	H-3	A	7/16	A	45	45	2'6"	SM	M	325	15	1040	950	1100	3.50	-	Soil was very wet following rain.
ST-66	H-5	H-3	A	7/16	A	45	45	2'6"	SM	M	-	15	1050	800	1075	3.10	-	1. Soil was wet second day after rain. 2. 20" diameter mound of soil at 7-1/2" pullout.
ST-67	B-6	D-4	B	3/4	A	45	45	2'9"	SM	M	175	110	1900	1420	2350	1.00	-	-
ST-68	C-6	D-4	B	3/4	A	45	45	2'9"	SM	M	-	135	1420	1000	1730	0.60	-	-
ST-69	D-6	D-4	B	3/4	A	45	45	2'9"	SM	M	350	123	1700	1270	2010	0.95	-	-
ST-70	E-6	D-4	B	3/4	A	30	30	2'9"	SM	M	260	83	980	920	1440	1.67	-	-
ST-71	F-6	D-4	B	3/4	A	30	30	2'9"	SM	M	225	80	1230	1100	1430	0.90	-	-
ST-72	G-6	D-4	B	3/4	A	30	30	2'9"	SM	M	-	98	1400	1250	1680	1.20	-	-
ST-73	H-6	D-4	B	3/4	A	15	15	2'9"	SM	M	200	65	640	1020	1300	1.00	-	-
ST-74	H-7	D-4	B	3/4	A	15	15	2'9"	SM	M	225	50	580	990	1400	1.20	-	-
ST-75	G-7	D-4	B	3/4	A	15	15	2'9"	SM	M	-	71	625	830	1220	1.00	-	-
ST-76	E-7	D-4	B	3/4	I	15	90	2'9"	SUR	M	-	155	-	850	5780	16.00	3700	-
ST-77	D-7	D-4	B	3/4	I	15	90	2'9"	SUR	M	375	133	-	750	6000	17.30	5230	-
ST-78	C-7	D-4	B	3/4	I	15	90	2'9"	SUR	M	260	164	-	650	4550	14.40	3400	-
ST-79	B-7	H-6	B	3/4	I	15	90	4'	SUR	M	220	180	-	870	8900	25.60	7800	Head of anchor came off at 8000 lb.
ST-80	A-10	H-6	B	3/4	I	15	90	4'	SUR	M	260	112	-	800	5900	15.00	5340	-
ST-81	B-10	H-6	B	3/4	I	15	90	4'	SUR	M	-	150	-	700	5400	15.80	3520	-
ST-82	C-10	H-6	B	3/4	I	30	90	4'	SUR	M	-	180	-	700	8200	19.70	2800	-
ST-83	D-10	H-6	B	3/4	I	30	90	4"	SUR	M	260	210	-	1000	6820	13.80	5030	Anchor head came off at 6800 lb.
ST-84	E-10	H-6	B	3/4	I	30	90	4'	SUR	M	230	222	-	680	4280	12.00	2100	-

Table B.1. Static Test Results on Site A (Silty Soils) (continued)

Test No.	Test Location	Anchor Type	Make	Shaft Size (in)	Pull	α_1 (degrees)	α_2 (degrees)	Depth	Loading	Soil Conditions	STP_3 Reading (in/lb)	Test Results: T_1 (ft/lb)	P_{2v} (lbs)	P_{4h} (lbs)	Q_u (lbs)	Δ_u (in)	R_{85} (lb/in)	Notes
ST-85	F-10	D-4	A	3/4	1	30	90	2'6"	SUR	M	-	102	-	1000	4500	12.00	2320	-
ST-86	G-10	D-4	A	3/4	1	30	90	2'6"	SUR	M	210	100	-	900	3700	10.80	2000	Anchor bent over and pulled out.
ST-87	H-10	D-4	A	3/4	1	30	90	2'6"	SUR	M	200	105	-	750	3800	12.00	2440	Anchor bent over and pulled out.
ST-88	J-10	D-4	A	3/4	I	45	90	2'6"	SUR	M	300	88	-	1150	3350	9.40	6200	Anchor bent over and pulled out.
ST-89	J-11	D-4	A	3/4	1	45	90	2'6"	SUR	M	275	128	-	1300	3720	9.55	5440	Anchor bent over and pulled out.
ST-90	H-11	D-4	A	3/4	1	45	90	2'6"	SUR	M	-	188	-	1050	3800	10.00	3360	Anchor bent over and pulled out.
ST-91	G-11	H-6	B	3/4	1	45	90	4'	SUR	M	250	170	-	1600	7860	12.60	3575	Anchor bent over and pulled out.
ST-92	F-11	H-6	B	3/4	1	45	90	4'	SUR	M	265	185	-	1350	8130	13.95	3400	Anchor bent over and pulled out.
ST-93	C-11	H-6	B	3/4	I	45	90	4'	SUR	M	-	172	-	1100	7810	15.00	2700	Anchor bent over and pulled out.
ST-94	B-11	H-6	B	3/4	I	60	90	4'	SUR	M	425	148	-	2000	8825	13.90	1700	Anchor bent over and pulled out.
ST-95	A-12	H-6	B	3/4	I	60	90	4'	SUR	M	315	110	-	3300	7070	7.10	2300	Anchor bent over and pulled out.
ST-96	C-12	H-6	B	3/4	I	60	90	4'	SUR	M	-	200	-	3600	6800	9.20	2030	Anchor bent over and pulled out.
ST-97	D-12	D-4	A	3/4	1	60	90	2'6"	SUR	M	250	100	-	2850	4300	5.70	3710	Anchor bent over and pulled out.
ST-98	E-12	D-4	A	3/4	1	60	90	2'6"	SUR	M	190	88	-	3050	3880	4.90	2160	Anchor bent over and pulled out.
ST-99	F-12	D-4	A	3/4	1	60	90	2'6"	SUR	M	-	100	-	3500	3660	4.58	2203	Anchor bent over and pulled out.
ST-100	G-12	H-6	B	3/4	1	60	135	4'	SUR	M	250	128	-	400	3840	24.50	760	Shaft bent back towards the pulling rig before it started to pull out.
ST-101	H-12	H-6	B	3/4	I	60	135	4'	SUR	M	250	155	-	400	2980	24.75	510	Shaft bent back towards the pulling rig before it started to pull out.
ST-102	J-12	H-6	B	3/4	I	60	135	4'	SUR	M	-	150	-	500	3340	26.20	1500	Shaft bent back towards the pulling rig before it started to pull out.
ST-103	J-13	H-6	B	3/4	I	45	135	4'	SUR	M	225	168	-	450	4280	32.40	800	-
ST-104	H-13	H-6	B	3/4	I	45	135	4'	SUR	M	200	122	-	380	4090	34.26	2050	-
ST-105	G-13	H-6	B	/34	1	45	135	4'	SUR	M	-	112	-	410	4360	34.74	1800	Shaft bent back towards the pulling rig before it started to pull out.
ST-106	F-13	H-6	B	3/4	1	15	135	4'	SUR	M	220	90	-	400	4100	55.00	1350	Shaft bent back towards the pulling rig before it started to pull out.
ST-107	E-13	H-6	B	3/4	1	15	135	4'	SUR	M	260	100	-	450	5425	56.00	-	Shaft bent back towards the pulling rig before it started to pull out.
ST-108	D-13	H-6	8	3/4	1	15	135	4'	SUR	M	-	155	-	450	4800	51.00	-	Shaft bent back towards the pulling rig before it started to pull out.
ST-122	M-16	H-6	O	3/4	A	90	90	4'	SUR	M	300	170	4000	-	4200	2.75	17000	
ST-201		P-10	E		A	90	90	4'	SM				1000	-	4700	12.3	-	Installation Time 60 sec.
ST-208		P-6	E		A	90	90	4'	SUR				700	-	3300	21.0	6000	Installation Time 45 sec.
ST-209		P-6	E		1	30	90	4'	SUR				1150	1200	2800	14.9	4860	Installation Time 28 sec.
ST-204		P-10	E		I	30	90	4'	SUR				650	700	4400	19.5	10625	Installation Time 50 sec.
ST-206		P-10	E		I	45	90	4'	SUR				1100	1200	2500	17.5	7730	Ioatallation Time 50 sec.
ST-207		P-10	E		I	45	90	4'	SUR				1200	1400	4200	18.0	10600	Installation Time 47 sec.
ST-210		P-6	E		1	45	90	4'	SUR				870	1350	2500	12.2	8920	Installation Time 25 sec.
ST-203		P-10	E		A	30	30	4'	SUR				500	500	4600	19.0	11300	Installation Time 53 sec.
ST-202		P-10	E		A	90	90	4'	SUR				900	-	6000	22.5	10200	Installation Time 45 sec.
ST-205		P-10	E		1	30	90	4'	SUR				1650	1850	5800	18.5	11200	Installation Time 48 sec.
ST-211		AH-6 1/	F		A	45	45	4.5'	SUR				300	550	5000	24.0	7730	Installation Time 95 sec.
ST-212		AH-6 1/	F		A	30	30	4.5'	SUR				600	650	5300	26.0	7080	Installation Time 51 sec.
ST-213		AH-6 2/	F		A	90	90	4.5'	SUR				500	-	5100	26.0	7080	Installation Time 41 sec.

1/ Steel anchor plate

2/ Aluminum anchor plate

Table B.2. Cyclic Test Results on Site A (Silty Soils)

Test No.	Test Location	Anchor Type	Make	Shaft Size (in)	Pull	α_1 (degrees)	α_2 (degrees)	Depth	Q_u (lbs)	P_c/Q_u	n	P_p/Q_u	Cumulative Displacement (in) Δ_1	Δ_{10}	Δ_{100}	Δ_u	R_{10} (lb/in)	Q'_u (lbs)	Notes
ST-111	M-13	H-6	B	3/4	A	90	90	4'	6000	0.75	300	0	1.90	2.80	3.35	3.35	47000	5000	-
ST-112	N-13	H-6	B	3/4	A	90	90	4'	6000	0.75	180	0	1.23	3.42	4.35	4.45	20500	5850	-
ST-113	N-14	H-6	B	3/4	A	90	90	4'	6000	0.75	200	0	1.50	3.05	4.15	4.25	30000	5450	-
ST-114	M-14	H-6	B	3/4	A	90	90	4'	6000	0.50	200	0	0.45	0.72	0.90	0.90	≐200000	5900	Soil was quite wet due to recent rain.
ST-115	L-14	H-6	B	3/4	A	90	90	4'	6000	0.50	200	0	1.12	1.55	2.66	2.75	≐200000	5500	-
ST-116	K-14	H-6	B	3/4	A	90	90	4'	6000	0.50	200	0	1.30	1.54	1.85	1.85	≐200000	5750	-
ST-117	K-15	H-6	B	3/4	A	90	90	4'	6000	0.25	200	0	0.18	0.22	0.35	0.35	>200000	6000	-
ST-118	L-15	H-6	B	3/4	A	90	90	4'	6000	0.25	200	0	0.21	0.35	0.41	0.42	>200000	6000	-
ST-119	M-15	H-6	B	3/4	A	90	90	4'	6000	0.25	200	0	0.19	0.30	0.32	0.35	>200000	5700	-
ST-123	L-16	H-6	D	3/4	A	90	90	4'	5640	0.67	200	0.84	0.13	0.41	0.90	1.10	180000	4550	Helix broke off of anchor shaft.
ST-124	K-16	H-6	D	3/4	A	90	90	4'	5640	0.67	200	0.84	0.15	0.31	0.74	0.92	50000	5350	Helix broke off of anchor shaft.
ST-125	ϕ-14	D-4	D	3/4	A	90	90	2'9"	2710	0.67	200	0.84	0.06	0.08	0.13	0.15	>100000	2500	-
ST-126	ϕ-13	D-4	D	3/4	A	90	90	2'9"	2710	0.67	200	0.84	0.10	0.14	0.20	0.21	>100000	2550	-

Table B.3. Static Test Results on Site B (Sandy Soils)

Test No.	Test Location	Anchor Type	Make	Shaft Size (in)	Pull	α_1 (degrees)	α_2 (degrees)	Depth	Loading	Soil Condition	STP Reading (in-lb)	Test Results T_i (ft/lb)	P_{zv} (lbs)	P_{4h} (lbs)	Q_u (lbs)	Δ_u (in)	R_{85} (lb/in)	Notes
SD-1	H-1	H-6	D	3/4	A	90	90	2'	SUR	M	350	300	4000	-	4160	2.16	15110	1. 30" diameter radial crack pattern around embedded anchor. 2. 6" diameter cylinder of soil pulled out with anchor.
SD-2	G-1	D-4	D	3/4	A	90	90	2'9"	SUR	M	-	280	1200	-	2870	0.75	21250	1. 24"-28" diameter radial crack pattern around embedded anchor. 2. 4" diameter cylinder of soil pulled out with anchor.
SD-3	F-1	D-4	D	3/4	A	90	90	2'9"	SUR	M	-	270	3500	-	3890	0.98	51000	1. 15"-28" diameter radial crack pattern around embedded anchor. 2. 4" diameter cylinder of soil pulled out with anchor.
SD-4	H-9	H-6	D	3/4	A	90	90	2'	SUR	M	-	300	4600	-	4880	1.50	22700	Helix broke off at 2-1/4".
SD-5	J-2	D-4	D	3/4	A	90	90	2'9"	SUR	M	-	250	2600	-	2770	1.25	34000	4" diameter cylinder of soil pulled out with anchor.
SD-6	J-3	D-4	D	3/4	I	40	90	2'9"	SUR	M	-	210	-	2800	3730	7.38	6800	16" diameter mound of soil around embedded anchor.
SD-7	J-4	D-4	D	3/4	I	40	90	2'9"	SUR	M	275	270	-	4000	5160	7.48	13900	1. 20" diameter mound of soil around embedded anchor. 2. Anchor bent over and pulled out.
SD-8	J-5	D-4	D	3/4	I	40	90	2'9"	SUR	M	-	300	-	2760	4650	11.00	16700	1. 20" diameter mound of soil around embedded anchor. 2. Anchor bent over and pulled out.
SD-9	J-6	D-4	D	3/4	A	40	45	2'9"	SUR	M	-	290	1050	510	1680	1.13	>50000	1. 16" diameter mound of soil around embedded anchor. 2. Anchor pulled out without a cylinder of soil.
SD-10	J-7	D-4	D	3/4	A	40	45	2'9"	SUR	M	350	200	720	410	1460	0.30	>50000	1. Small amount of mounding. 2. Anchor pulled out without a cylinder of soil.
SD-11	J-8	D-4	D	3/4	A	40	45	2'9"	SUR	M	-	210	1110	630	2300	0.79	>50000	1. 20" diameter mound of soil around embedded anchor. 2. Anchor pulled out without a cylinder of soil.
SD-20	K-1	H-6	D	3/4	A	90	90	2'	SUR	M	360	155	2040	-	2260	2.83	8500	24" diameter radial crack pattern around embedded anchor.
SD-21	K-2	H-6	D	3/4	A	90	90	2'	SUR	M	-	200	1750	-	1780	1.64	6800	24" diameter radial crack pattern around embedded anchor.
SD-22	K-3	H-6	D	3/4	A	90	90	3'	SUR	M	360	150	3560	-	4040	2.83	13260	Helix broke off at 3" displacement and at 4,000 lbs.
SD-23	K-4	H-6	D	3/4	A	90	90	3'	SUR	M	-	200	4860	-	4925	2.34	16190	3" diameter radial crack pattern around embedded anchor.
SD-24	K-5	H-6	D	3/4	A	90	90	4'	SUR	M	200	220	4500	-	5000	2.97	11780	30"-36" diameter radial crack pattern around embedded anchor.
SD-25	K-6	M-6	D	3/4	A	90	90	4'	SUR	M	-	230	4825	-	5130	3.53	13900	24"-30" diameter radial crack pattern around embedded anchor.
SD-26	K-7	H-6	D	3/4	A	90	90	4'	SUR	M	160	130	3600	-	3930	2.78	9800	Helix broke off at 3-1/4" displacement and at 3,900 lbs.
SD-27	K-8	H-6	D	3/4	A	40	45	4'	SUR	M	-	-	2380	1430	2750	2.10	>50000	-
SD-28	K-9	H-6	D	3/4	A	40	45	4'	SUR	M	360	-	2610	2480	2690	1.50	34000	Small amount of mounding.
SD-29	K-10	M-6	0	3/4	A	40	45	4'	SUR	M	-	-	2300	1775	2500	1.50	14200	Small amount of mounding.
SD-30	K-11	N-6	D	3/4	I	40	90	4'	SUR	M	475	240	-	2750	6000	8.35	6070	Anchor bent over and helix broke off at 9.3".
SD-31	L-11	H-6	D	3/4	I	40	90	4'	SUR	M	-	250	-	3000	6560	9.50	6240	Anchor bent over and helix broke off at 10.3".
SD-32	L-10	H-6	D	3/4	I	40	90	4'	SUR	M	380	240	-	2000	6000	9.40	5950	Anchor bent over and helix broke off at 10.1".
SD-41	L-1	D-4	D	3/4	I	40	90	2'9"	SUR	M	-	180	-	870	3000	7.90	16400	12" diameter radial crack pattern around embedded anchor.
SD-42	M-1	D-4	D	3/4	I	40	90	2'9"	SUR	M	370	190	-	925	2230	7.65	11330	1. 12" diameter radial crack pattern around embedded anchor. 2. Anchor bent over and pulled out.
SD-43	M-2	D-4	D	3/4	I	40	90	2'9"	SUR	M	-	200	-	1000	2210	7.23	10100	15" diameter radial crack pattern around embedded aochor.
SD-44	M-3	D-4	D	3/4	A	90	90	2'9"	SUR	M	325	220	1000	-	1750	0.47	42500	12" diameter radial crack pattern around embedded anchor.
SD-45	M-4	D-4	D	3/4	A	90	90	2'9"	SUR	M	-	195	940	-	1540	0.34	28330	12" diameter radial crack pattern around embedded anchor.
SD-46	M-5	D-4	D	3/4	A	90	90	2'9"	SUR	M	275	170	950	-	1530	0.41	11700	12" diameter radial crack pattern around embedded anchor.
SD-47	M-6	M-6	B	3/4	A	90	90	4'	SUR	M	230	150	4000	-	5150	6.47	22900	24" diameter radial crack pattern around embedded anchor.
SD-48	M-7	H-6	B	3/4	A	90	90	4'	SUR	M	-	160	5000	-	5880	3.68	23100	24" diameter radial crack pattern around embedded anchor.
SD-49	M-8	H-6	8	3/4	A	90	90	4'	SUR	M	-	220	5000	-	5290	3.51	19800	20" diameter radial crack pattern around embedded anchor.
SD-50	M-10	H-6	8	3/4	A	90	90	4'	SUR	W	300	250	6000	-	6825	4.18	20900	20" diameter radial crack pattern around embedded anchor.
SD-51	M-11	H-6	8	3/4	A	90	90	4'	SUR	W	-	250	5000	-	6275	5.50	22800	24" diameter radial crack pattern around embedded anchor.
SD-52	N-11	H-6	8	3/4	A	90	90	4'	SUR	W	300	250	4300	-	4760	3.38	20720	-

Table 8.4. Cyclic Test Results on Site 8 (Sandy Soils)

Test No.	Test Location	Anchor Type	Make	Shaft Size (in)	Pull	α_1 (degrees)	α_2 (degrees)	Depth	Q_u (lbs)	P_c/Q_u	n	P_p/Q_u	Cumulative Displacement (in) δ_1	δ_{10}	δ_{100}	δ_u	R_{10} (lb/in)	Q_u' (lbs)	Notes
SD-12	H-9	D-4	O	3/4	I	40	90	2'9"	4643	0.75	280	0	3.97	4.61	5.46	6.16	20000	4900	Anchor bent over and pulled out with 4" diameter cylinder of soil.
SD-13	H-8	D-4	D	3/4	I	40	90	2'9"	4643	0.75	10	0	5.20	7.20	-	-	7400	-	1. Anchor pulled out with 20" diameter mound of soil. 2. Only completed 10 cycles before failure.
SD-14	H-7	D-4	D	3/4	I	40	90	2'9"	4643	0.67	-	0.84	-	-	-	-	-	-	Anchor pulled out before preload was reached.
SD-15	H-6	D-4	D	3/4	I	40	90	2'9"	4643	0.67	-	0.84	-	-	-	-	-	-	Anchor pulled out before preload was reached.
SD-16	H-5	D-4	D	3/4	I	40	90	2'9"	3627	0.75	300	0	4.80	5.60	6.60	7.40	18000	3400	Anchor bent over and pulled out.
SD-17	H-4	D-4	D	3/4	I	40	90	2'9"	3627	0.75	200	0	4.27	4.82	5.45	5.73	18000	4200	Anchor bent over and pulled out with 20" diameter mound of soil.
SD-18	H-3	D-4	D	3/4	I	40	90	2'9"	3770	0.67	200	0.84	0.75	0.95	1.40	1.67	17000	4700	Anchor bent over and pulled out with a cylinder of soil.
SD-19	H-2	D-4	D	3/4	I	40	90	2'9"	3770	0.67	180	0.84	0.80	1.10	1.55	1.72	20000	3850	Anchor bent over and pulled out with a cylinder of soil.
SD-33	L-9	H-6	O	3/4	A	90	90	4'	3475	0.75	300	0	0.85	1.37	3.30	5.43	23000	4700	24" diameter radial crack pattern around embedded anchor.
SD-34	L-8	H-6	O	3/4	A	90	90	4'	3475	0.75	200	0	1.52	2.18	4.63	5.90	20500	4500	24" diameter radial crack pattern around embedded anchor.
SD-35	L-7	H-6	D	3/4	A	90	90	4'	3860	0.67	200	0.84	0.25	0.60	1.67	2.14	12360	5100	36" diameter radial crack pattern around embedded anchor.
SD-36	L-6	H-6	O	3/4	A	90	90	4'	3860	0.67	200	0.84	0.20	0.57	1.97	2.60	20600	4400	36" diameter radial crack pattern around embedded anchor.
SD-37	L-5	H-6	D	3/4	I	40	90	4'	4675	0.75	121	0	8.85	9.85	12.27	-	14600	-	Helix broke off at n = 121.
SD-38	L-4	H-6	O	3/4	I	40	90	4'	4675	0.75	200	0	7.52	8.22	10.22	11.57	18700	-	Load was increased to 7000 lb when helix broke off.
SD-39	L-3	H-6	O	3/4	I	40	90	4'	5200	0.67	200	0.84	0.25	0.45	1.15	1.47	20700	-	Load was increased to 6400 lb when helix broke off.
SD-40	L-2	H-6	D	3/4	I	40	90	4'	5200	0.67	200	0.84	0.33	0.75	1.82	2.29	13500	-	Load was increased to 5800 lb when helix broke off.
SD-53	N-9	D-4	O	3/4	A	90	90	2'9"	1627	0.67	200	0.84	0.01	0.07	0.24	0.35	>50000	2330	12" diameter radial crack pattern around embedded anchor.
SD-54	N-8	D-4	D	3/4	A	90	90	2'9"	1627	0.67	200	0.84	0.03	0.08	0.37	0.25	>50000	2320	12" diameter radial crack pattern around embedded anchor.

Table B.5. Static Test Results on Site C (Clayey Soils)

Test No.	Test Location	Anchor Type	Make	Shaft Size (in)	Pull	α_1 (degrees)	α_2 (degrees)	Depth	Loading	Soil Condition	STP Reading (in/lb)	Test Results T_i (ft/lb)	P_{2v} (lbs)	P_{4h} (lbs)	Q_u (lbs)	Δ_u (in)	R_{85} (lb/in)	Notes
C-1	A-11	H-6	B	3/4	A	90	90	1'	SM	W	50	50	750	-	910	0.75	-	24" diameter mound of soil around embedded anchor
C-2	A-10	H-6	B	3/4	A	90	90	2'	SUR	W	-	55	1700	-	1750	2.75	12140	Very slight amount of mounding.
C-3	A-9	H-6	B	3/4	A	90	90	2'	SUR	M	-	50	1750	-	1850	3.50	10650	Very slight amount of mounding.
C-4	B-11	H-6	B	3/4	A	90	90	3'	SUR	M	-	80	2750	-	3100	4.50	13700	30" diameter mound of soil and anchor pulled out at 12" displacement.
C-5	B-10	H-6	B	3/4	A	90	90	3'	SUR	M	-	80	2500	-	2850	4.00	12140	Small smount of mounding.
C-6	B-9	H-6	B	3/4	A	90	90	4'	SUR	M	100	100	2300	-	2800	9.00	12140	Small amount of mound and anchor pulled out at 24" displacement
C-7	C-11	H-6	B	3/4	A	90	90	4'	SUR	M	-	90	3400	-	3850	5.00	7612	Small amount of mounding and anchor pulled out at 15" displacement
C-8	C-9	H-6	B	3/4	A	90	90	4'	SUR	M	-	210	3500	-	3650	3.50	13700	Small amount of mounding and anchor pulled out at 20" displacement
C-9	D-11	D-4	D	3/4	A	90	90	2'9"	SUR	M	-	105	1850	-	1900	1.00	13500	Small amount of mounding and anchor pulled out at 8" displacement.
C-10	E-11	D-4	D	3/4	A	90	90	2'9"	SUR	M	75	105	1850	-	1900	2.50	42500	Small amount of mounding and anchor pulled out at 8" displacement.
C-11	F-11	D-4	D	3/4	A	90	90	2'9"	SUR	M	-	70	1750	-	2000	4.00	16150	Very Small amount of mounding.
C-12	G-11	D-4	D	3/4	I	50	90	2'9"	SUR	M	150	90	-	2500	2500	6.22	14700	Anchor bent over and pulled out.
C-13	G-10	D-4	D	3/4	I	50	90	2'9"	SUR	M	-	110	-	2100	2100	4.73	9000	Anchor bent over and pulled out.
C-14	F-9	D-4	D	3/4	I	40	90	2'9"	SUR	M	125	75	-	1050	1800	8.00	1700	Anchor bent over and pulled out.
C-15	G-9	D-4	D	3/4	A	45	45	2'9"	SUR	M	-	50	1450	1300	1500	2.55	8500	Slight amount of mounding.
C-16	H-10	D-4	D	3/4	A	45	45	2'9"	SUR	M	-	75	1750	1400	1750	2.25	21250	Slight amount of mounding.
C-17	H-11	D-4	D	3/4	A	45	45	2'9"	SUR	M	120	-	1700	1400	1800	2.40	21250	Slight amount of mounding and anchor pulled out.
C-18	J-10	H-6	B	3/4	I	40	90	4'	SUR	M	-	100	-	900	2500	14.68	13600	Anchor bent over and pulled out.
C-19	J-9	H-6	B	3/4	I	40	90	4'	SUR	M	100	150	-	700	3800	18.92	-	1. Displacement was measured horizontally. 2. Anchor bent over and pulled out.
C-20	H-9	H-6	B	3/4	I	40	90	4'	SUR	M	-	150	-	700	3500	22.19	19270	-
C-21	G-8	H-6	B	3/4	A	45	45	4'	SUR	M	90	80	2350	2600	2700	6.02	-	Anchor pulled out 14"-16" before mounding took place
C-22	H-8	H-6	B	3/4	A	45	45	4'	SUR	M	-	55	2300	2450	2450	3.96	8500	Anchor pulled out 15"-16" before mounding took place.
C-23	G-7	H-6	B	3/4	A	45	45	4'	SUR	M	100	65	2400	2420	2450	5.37	9270	Anchor pulled out 15" before mounding took place.

Table B.6. Cyclic Test Results on Site C (Clayey Soils)

Test No.	Test Location	Anchor Type	Make	Shaft Size (in)	Pull	α_1 (degrees)	α_2 (degrees)	Depth	Q_u (lbs)	P_c/Q_u	n	P_p/Q_u	Cumulative Displacement (in) Δ_1	Δ_{10}	Δ_{100}	Δ_u	R_{10} (lb/in)	Q'_u (lbs)	Notes
C-24	E-7	H-6	B	3/4	A	90	90	4'	3575	0.75	83	0	1.67	3.00	-	16.55	12100	-	Reached the limit of loading ram at n = 83.
C-25	E-6	H-6	B	3/4	A	90	90	4'	3575	0.75	190	0	0.48	0.60	2.47	7.20	>50000	-	Reached the limit of loading ram at n = 190.
C-26	E-5	H-6	B	3/4	A	90	90	4'	3575	0.67	200	0.84	0.20	0.35	0.85	1.20	>50000	4000	-
C-27	D-6	H-6	B	3/4	A	90	90	4'	3575	0.67	161	0.84	0.30	0.90	5.00	16.00	9000	-	Test stopped due to excessive amount of displacement.
C-29	C-7	D-4	D	11/16	A	90	90	2'9"	1470	0.75	200	0	0.25	0.40	1.25	1.52	>100000	1900	-
C-32	C-6	D-4	D	11/16	A	90	90	2'9"	1470	0.75	200	0	0.10	0.25	1.00	1.07	>100000	2100	-
C-33	D-7	D-4	D	11/16	A	90	90	2'9"	1470	0.67	450	0.84	0.10	0.26	1.02	10.00	20000	-	-
C-34	D-8	D-4	D	11/16	A	90	90	2'9"	1470	0.67	200	0.84	0.02	0.06	0.15	0.18	>100000	2200	-
C-35	A-6	H-6	B	3/4	I	40	90	4'	2740	0.75	200	0	10.30	10.90	12.05	13.55	9000	3400	-
C-36	B-5	D-4	D	11/16	I	40	90	2'9"	1665	0.75	69	0	8.90	10.45	-	15.90	3500	-	Reached the limit of loading ram at n = 69.
C-37	A-4	D-4	D	11/16	I	40	90	2'9"	1665	0.75	41	0	11.60	13.60	-	17.50	5700	-	Reached the limit of loading ram at n = 41.
C-38	B-3	H-6	B	3/4	I	40	90	4'	3650	0.75	124	0	13.20	14.85	20.00	21.75	40000	-	-
C-39	A-2	D-4	D	11/16	I	40	90	2'9"	1470	0.67	200	0.84	1.60	2.15	3.70	4.30	3100	2100	-
C-40	E-2	H-6	B	3/4	I	40	90	4'	3650	0.67	200	0.84	1.50	2.10	4.20	4.80	12000	-	-
C-41	D-1	H-6	B	3/4	I	40	90	4'	3650	0.67	200	0.84	0.91	1.52	3.65	4.21	12000	5000	-
C-42	E-1	D-4	D	11/16	I	40	90	2'9"	1470	0.67	200	0.84	1.36	1.80	2.95	3.45	2100	2100	-

NBS-114A (REV. 2-8C)

U.S. DEPT. OF COMM. BIBLIOGRAPHIC DATA SHEET (See instructions)	1. PUBLICATION OR REPORT NO. NBS BSS 142	2. Performing Organ. Report No.	3. Publication Date May 1982

4. TITLE AND SUBTITLE

Load-Displacement Characteristics of Shallow Soil Anchors

5. AUTHOR(S)

Felix Y. Yokel, Riley M. Chung, Frank A. Rankin and Charles W. C. Yancey

6. PERFORMING ORGANIZATION *(If joint or other than NBS, see instructions)*

NATIONAL BUREAU OF STANDARDS
DEPARTMENT OF COMMERCE
WASHINGTON, D.C. 20234

7. Contract/Grant No.

8. Type of Report & Period Covered

Final

9. SPONSORING ORGANIZATION NAME AND COMPLETE ADDRESS *(Street, City, State, ZIP)*

Office of Policy Development and Research
U.S. Department of Housing and Urban Development
Washington, DC 20410

10. SUPPLEMENTARY NOTES

Library of Congress Catalog Card Number: 82-600509

☐ Document describes a computer program; SF-185, FIPS Software Summary, is attached.

11. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here)*

Tests on shallow soil anchors, commonly used by the mobile home industry, including 6-in single helix and 4-in double helix anchors as well as three types of swivel anchors, were conducted on three sites: a silty site, a sandy site, and a clay site. Test variables included direction of anchor installation; direction of loading; anchor depth; size of anchor plate; and cyclic load effects. The effect of these test variables on load-displacement characteristics, measured at the anchor head, is investigated. It is concluded that on most sites the anchor types tested, when installed in accordance with present industry practice for mobile home tiedown systems, did not deliver the anchor performance required in present standards. It is recommended that minimum load capacity requirements for anchors be waived; that all anchors be preloaded to 1.25 times the design load; and that one anchor per mobile home, or three anchors per site if soil conditions are uniform, be preloaded to 1.5 times the design load.

12. KEY WORDS *(Six to twelve entries; alphabetical order; capitalize only proper names; and separate key words by semicolons)*

anchors; cyclic loading; field testing; flood forces; foundations; load capacity; mobile homes; soil anchors; soil mechanics; stiffness; wind forces

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