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JETTED-IN MARINE ANCHORS

By

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ABSTRACT

Twenty-three lightweight anchors consisting of a 10-foot X 2½-inch pipe with a metal cone welded at the tip end were emplaced and tested in twenty-five feet of water at Lameshur Bay, St. John Island, U. S. Virgin Islands. The anchors were jetted into the coral sand bottom by forcing water through the pipe and out an aperture in the cone. Two divers guided the anchors into the sediment using the jet of water to excavate the soil beneath the cone. Four of the anchors were emplaced with a cement slurry to increase the holding power.

It was found that the emplacement procedures were straightforward and posed no problems to the divers. However, the injection of the cement slurry was very time-consuming due to numerous problems.

The pullout results and theoretical analysis showed the jetted anchors to be capable of developing 2,000 to 10,000 lbs holding capacities in the soil at the test site. The holding capacity may be increased by increasing any of the following: anchor cone diameter (area), emplacement depth, compaction of the overburden sediment, or use of a cement slurry.

Tests indicate that the use of these light weight jetted-in anchors may be of practical use where bottom tie-downs and light anchorages are required. Further testing is recommended.

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INTRODUCTION

Recent increases in underwater activity and increasing military and industrial requirements for divers to emplace and construct structures on the seafloor have generated new requirements for diver tools and work systems. One such tool requirement is for an inexpensive, easily emplaced, diver-handled anchor. Such anchors should be capable of sustaining vertical loads from 2,000 to 10,000 pounds. Uses of such anchors include pipe and cable tie-downs, instrument package tie-downs, tie-downs and pulling points for underwater construction and moorings.

After completion of Project TEKTITE, one concept for emplacement of such light-capacity, diver-emplaced anchors was evaluated by the TEKTITE Seabee Construction Divers. The test anchor consisted of a 10-foot pipe with a sheet metal cone welded at one end, as shown in Figure 1. They were "jetted" into the seafloor by a stream of water through the pipe (anchor shaft) and out the nozzle apex of the cone. As the water jet excavated the soil beneath the cone, two divers guided the anchor into the bottom while holding the shaft in a vertical position.

The tests were conducted at Greater Lameshur Bay, St. John Island in the U. S. Virgin Islands in a coral sand bottom at a depth of 25 feet. Twenty-three anchors were jetted in and tested during the program. In an effort to increase the holding power, four of these were emplaced with a cement slurry injected at the cone. The jet-in anchors were constructed inexpensively by SEABEE steel workers.

The objectives of this experiment were as follows:

1. To develop, acquire data for, and validate a standard procedure for diver emplacement of an anchor on the ocean floor by use of a water jet.

2. To investigate the use of a cement slurry to secure the anchor to the ocean floor material in a more permanent emplacement.

3. To investigate the holding properties of the indigenous coral sand.

In addition, the parameters of cone diameter (anchor area), cone angle and jetting nozzle size were varied in order to investigate their effect on the ease of emplacement and the holding power.

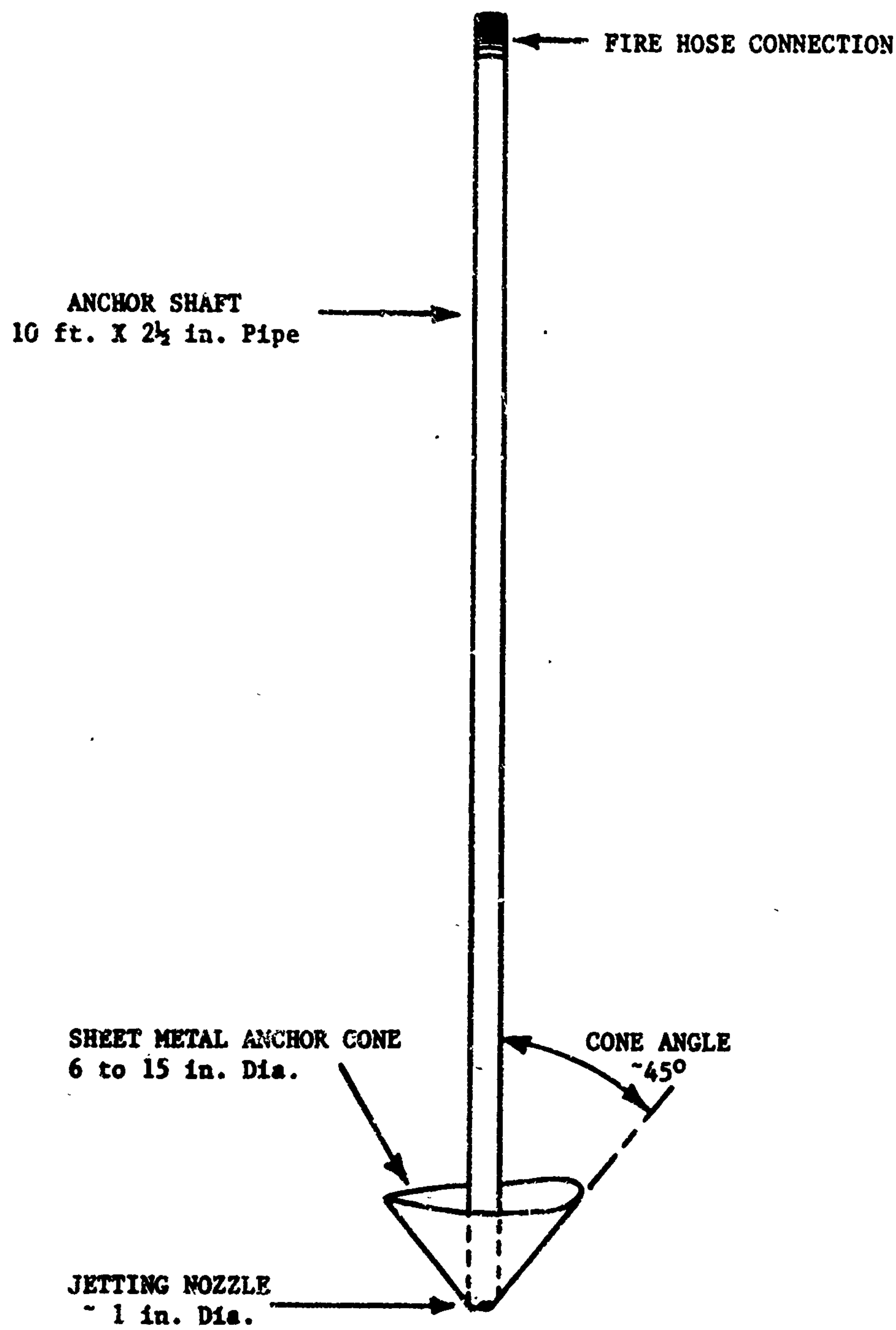


FIGURE 1. ILLUSTRATION OF JETTED ANCHOR

EMPLACEMENT AND TESTING PROCEDURES

Emplacement

The twenty-three anchors emplaced during the test program were in a plot 40-ft X 40-ft on a flat bottom in 25 feet of water. The plot was divided into 10-ft X 10-ft sections and stakes were driven at the corners. Thus, 25 anchor sites were laid out within the test plot.

The equipment used to emplace the anchors consisted of the fully equipped (SCUBA and shallow water gear) diving barge moored at the site, a standard P-250 Gas Driven Fire Pump, 2½-in. fire hose, hose adaptor, safety strap, signal line, and the anchors themselves.

The anchors were jetted by two divers to a depth of 6 to 9 feet into the sand bottom. The divers were equipped with SCUBA or lightweight surface-supplied life support equipment according to their own preference. The SCUBA divers were essentially neutrally buoyant and wore fins, whereas the surface-supplied divers used from 10 to 20 lb. weight belts and wore no fins. The jetting procedure is shown in Figure 2.

Cement Slurry Injection

The purpose of these tests was to determine the effect of a cement slurry injected either beneath the anchor cone or just above it in order to bind the anchor more permanently into the surrounding sediment. A cement slurry made of one part Portland cement, one part sand, and one part water was injected on four of the anchors emplaced at the test site. These were 9" in diameter. The equipment used, in addition to that used to jet the anchor, consisted of a slurry pot (see Figure 3), an L. P. air compressor, hose, assorted pipe fittings and valves.

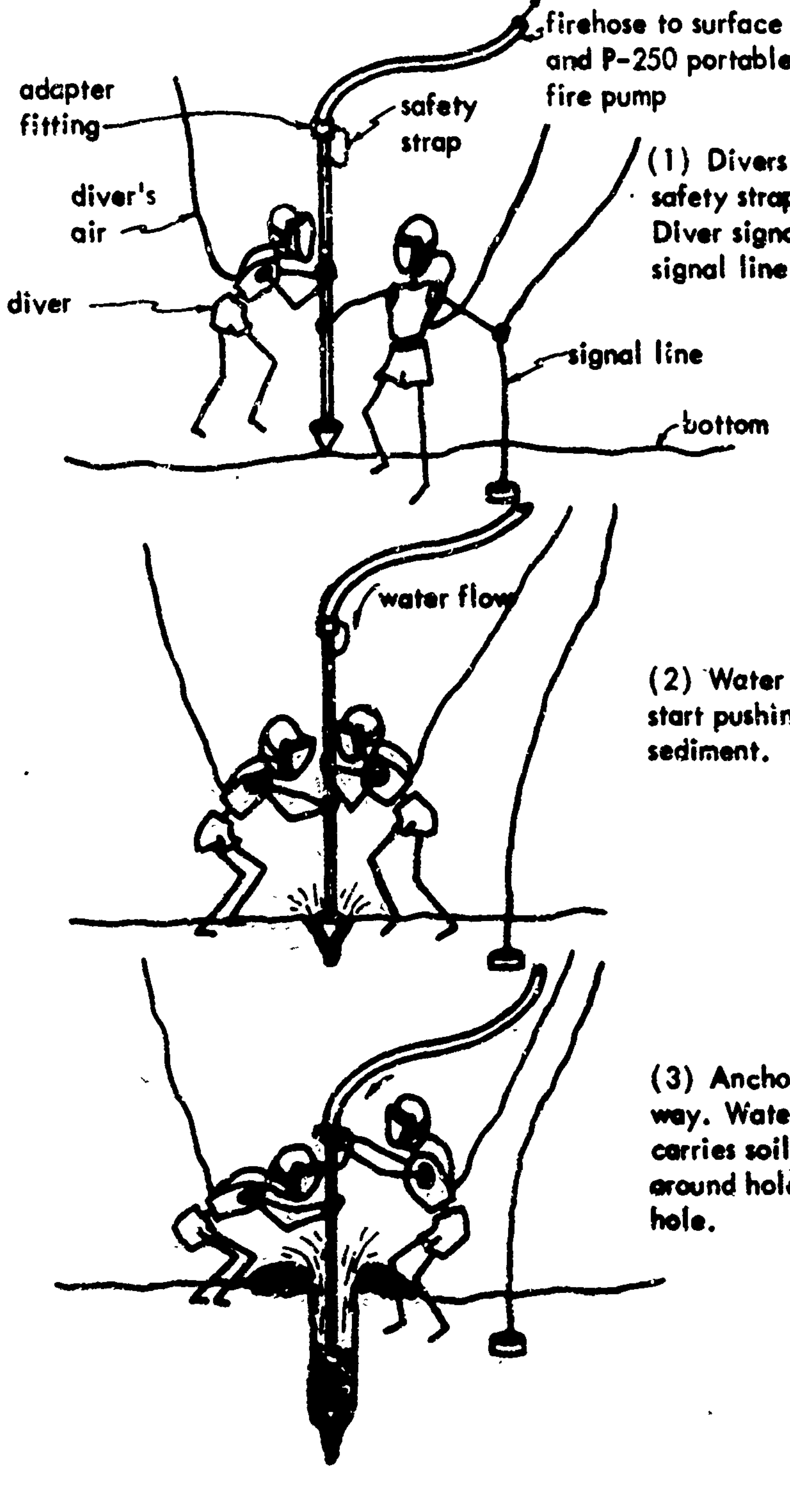
The procedure consisted of mixing the slurry, filling the pot with approximately 16 gallons of mix, sealing it, and injecting the slurry by pressurizing the pot to 35 psi (24 psi over bottom pressure) with air and opening the gate valve at the bottom of the pot. For two of the four anchors slurried, the slurry was injected through the anchor shaft and out the tip of the cone. The other two were slurried through a separate probe placing the cement just above the anchor cone.

Some problems were encountered during the slurrying operation. The first anchor that was slurried through the anchor shaft was done with the slurry pot attached to the top of the anchor, then jetted as shown in Figure 4. Although this setup was satisfactory for injecting the slurry, it was found to be unwieldy due to the bulkiness of the pot, and because its buoyancy changed drastically as the slurry was forced out. This procedure also proved to be very costly in terms of use of diver bottom time. Therefore, the other three anchors were slurried by keeping the pot on the diving barge and pumping the slurry to the anchor through a fire hose.

Figure 2.

JETTING OPERATION:

All anchors were jetted by two divers. A water jet was used to emplace the anchors six feet to nine feet into the sediment. Equipment used and jetting sequence is depicted below.

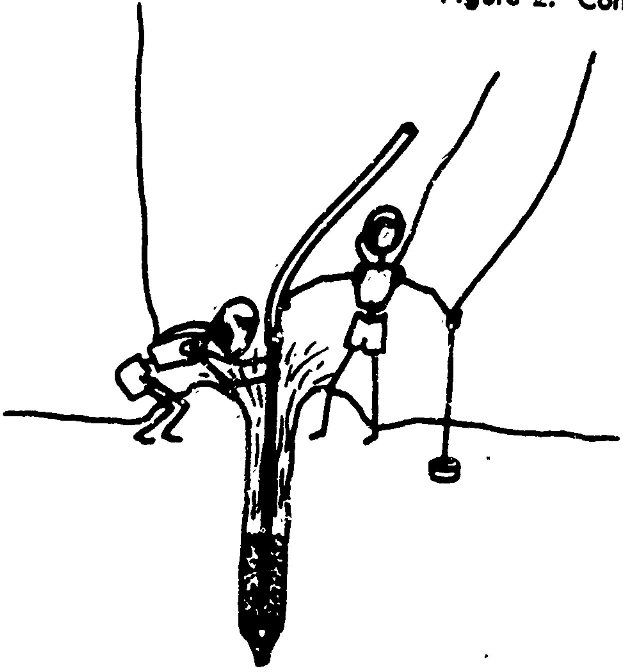


(1) Divers connect hose and safety strap, and erect anchor. Diver signals surface with signal line. "Start pump"

(2) Water jet starts. Divers start pushing anchor into sediment.

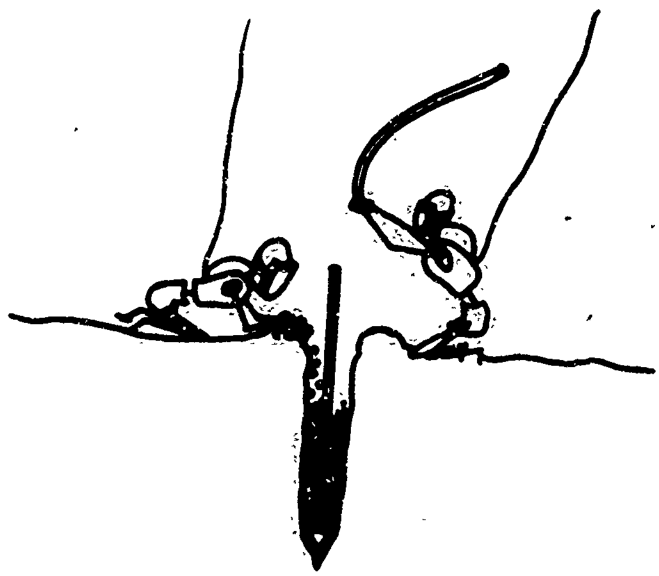
(3) Anchor jetted about half-way. Water flowing out of hole carries soil and builds mound around hole. Some soil stays in hole.

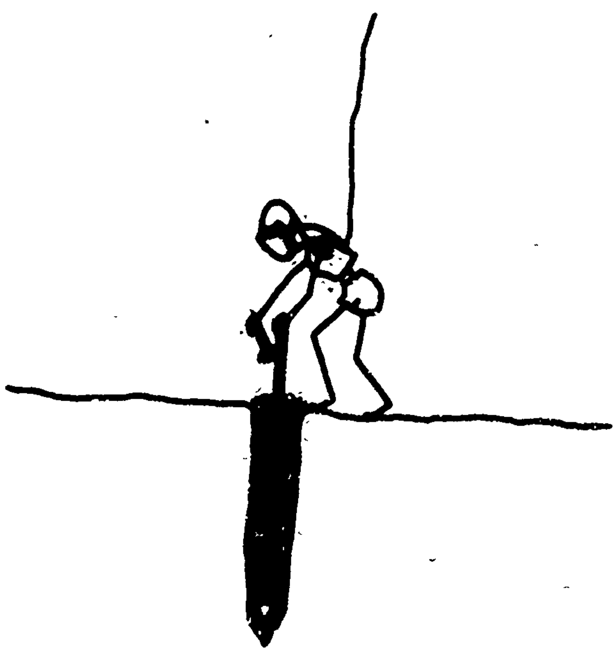
JETTING OPERATION Figure 2. Continued.



(4) Anchor 6' to 9' down. Diver uses signal line. "Stop pump"

(5) Divers disconnect hose and refill hole with soil.





(6) Diver vibrates loose soil with 10 raps from an 8 lb. hammer.

Figure 3. Slurry pot.

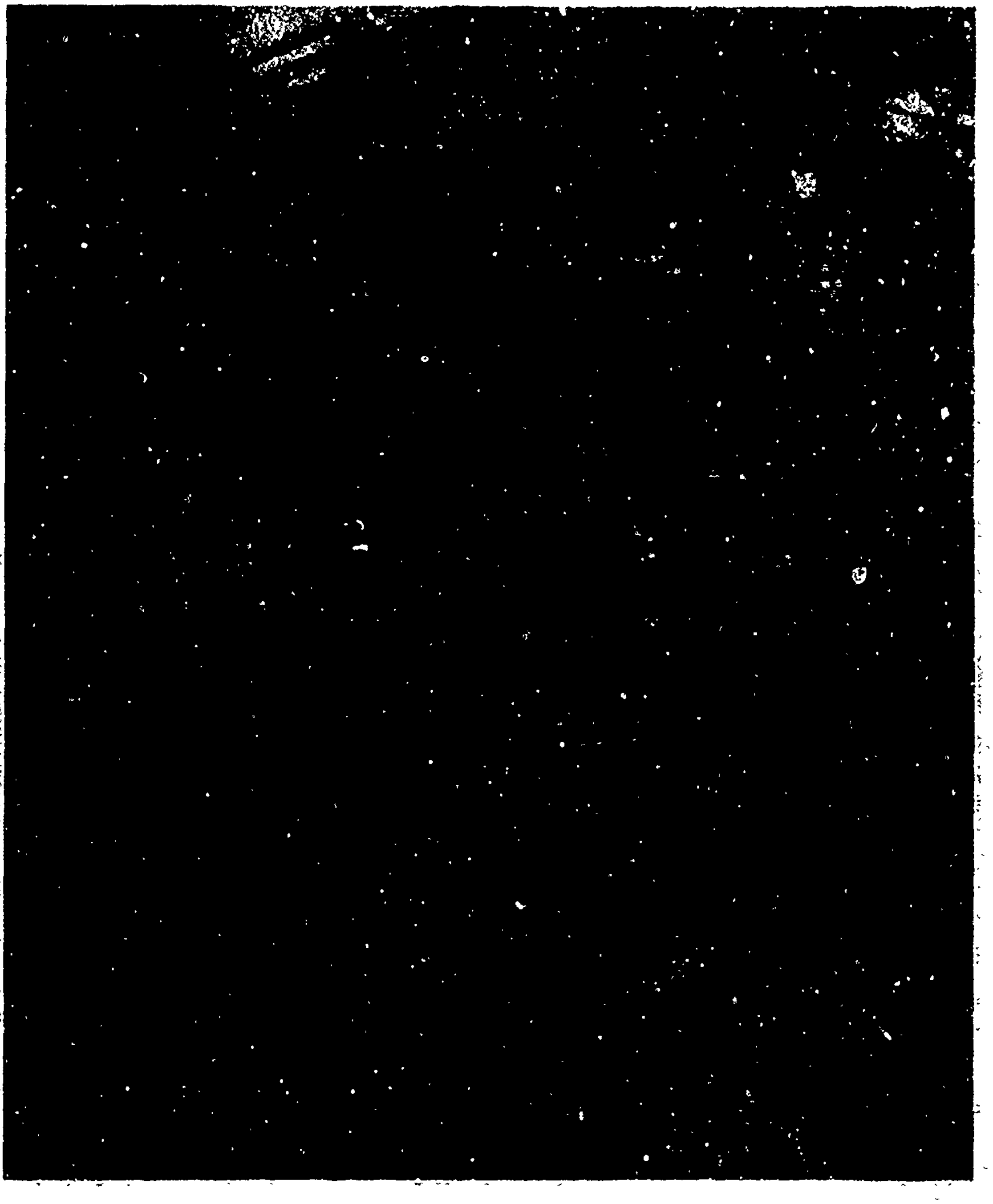


Figure 4. Diver jetting anchor with slurry pot attached.

Another problem encountered while injecting the slurry was that of the mix "setting up" in the pot prior to injection. This occurred twice, once necessitating that a hole be cut in the side of the pot, the concrete chunks removed, and the pot welded together again. This problem was finally traced to the fact that the pot had not been completely cleaned after the previous operation.

The slurrying effort placed approximately two cubic feet of mix beneath the anchor cone on two of the anchors, approximately two cubic feet above the cone on one anchor, and about one cubic foot of mix above the cone on the fourth anchor.

Extraction

One week after all anchors had been emplaced, the anchors were tested as described in Figure 5. During the pulling operations, it was sometimes necessary to connect two or three chain fall come-alongs in parallel since each come-along was rated at 3,000 lbs and transient loads up to 10,000 lbs were encountered. In addition, in-situ soil samples were obtained and returned to the Naval Civil Engineering Laboratory where the soil properties were determined. Results of all tests are presented and discussed later in the text.

Observations

The data obtained from the experiment were used in determining the ease of emplacement of the anchors, the holding power of the anchors and the soil characteristics at the site. The basic data recorded at the site included:

1. The set-up time, jetting time, breakdown time* for each anchor, and slurry injection time for the four anchors.

2. The overall time and man-hours required to emplace 15 of the anchors.

3. Emplacement problems.

4. Load vs. displacement for each anchor as it was extracted.

OPERATIONAL ANALYSIS

Emplacement

During the test operations, attention was paid to the ease of emplacement, the jetting procedures and the equipment used in order to develop

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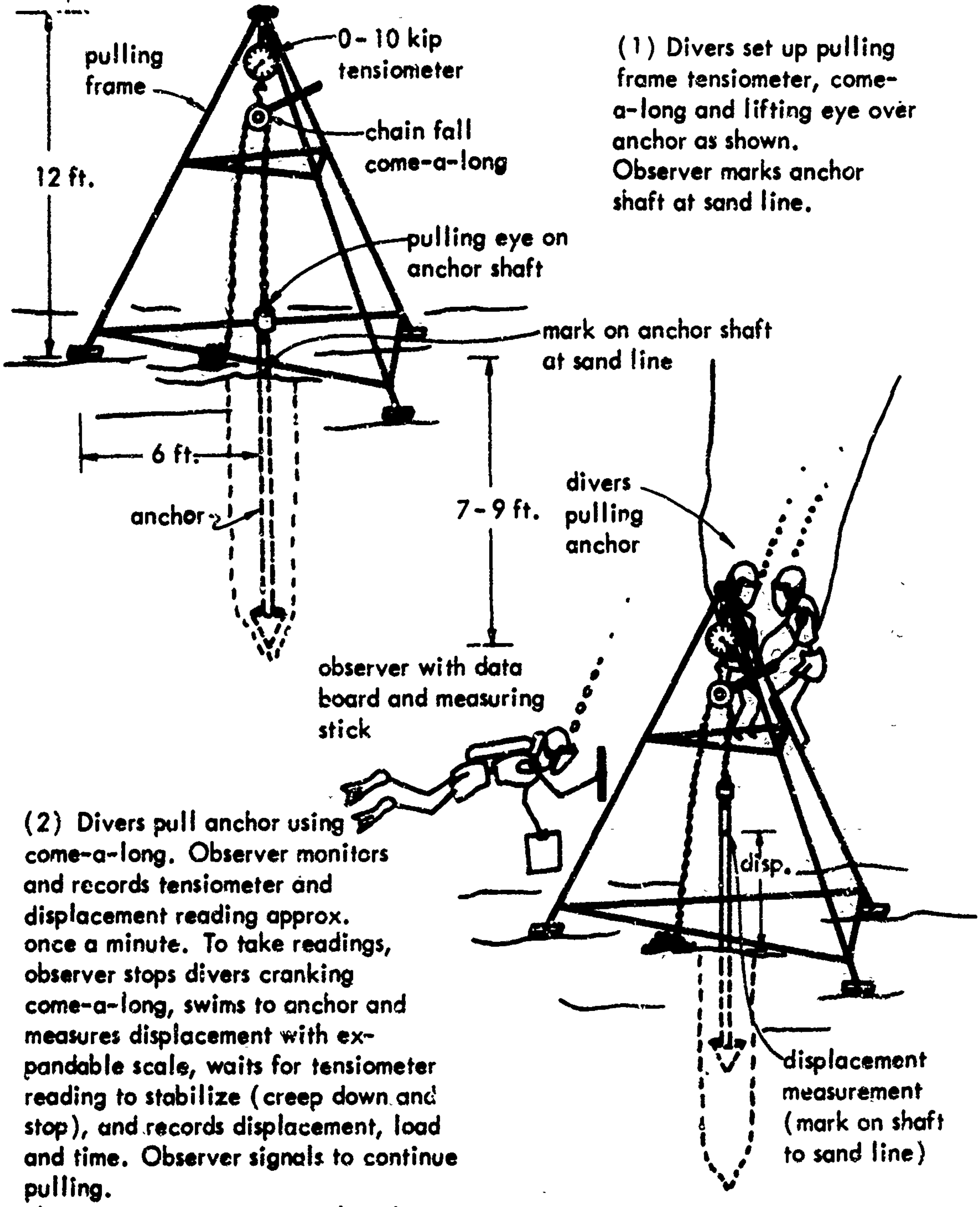
*Time to disconnect emplacement equipment and be ready for next emplacement.

EXTRACTION OPERATION : **Figure 5.**

The extraction and data gathering procedure is described and depicted below.



(1) Divers set up pulling frame tensiometer, come-a-long and lifting eye over anchor as shown. Observer marks anchor shaft at sand line.

(2) Divers pull anchor using come-a-long. Observer monitors and records tensiometer and displacement reading approx. once a minute. To take readings, observer stops divers cranking come-a-long, swims to anchor and measures displacement with expandable scale, waits for tensiometer reading to stabilize (creep down and stop), and records displacement, load and time. Observer signals to continue pulling.
This process is repeated until anchor cone breaks out of sediment.

and validate a standard procedure for the use of jetted anchors. An accounting was made of the time required for each operation of the emplacement sequence. Mean time for each operation and the overall emplacement time for each of the 23 anchors emplaced are given below:

Operation	Mean Bottom Time (Min)
Set-up	3.0 Min
Actual Jetting	2.0
Breakdown	1.8 Min
Fill-in and compact time	4.3 Min
Mean Overall Emplacement Time	11.1 Min

For one series of tests, the diving barge was on station a total of 2 hours and 58 minutes, during which time 15 anchors were emplaced. Thus, the time per anchor was approximately 11.9 minutes. This figure compares reasonably well with a mean overall emplacement time of 11.1 minutes, and indicates that a minimal amount of on-site topside preparation was necessary. No serious difficulties were encountered during the emplacement operations.

The actual operation was carried out by six men. Two divers and one man operating the pump and tending the fire hose were the primary operators; one diving supervisor and a tender for each surface-supplied diver were required backup personnel. Combining this information with the mean time on site per anchor (11.9 minutes) yields 1.2 man hour per anchor using the six-man crew. The economics of emplacing the anchors in the present case (using an experienced crew) included:

1.	Fabrication	Less than $10.00
2.	Transportation	Non-fragile, lightweight
3.	Emplacement	1.2 m/hr/anchor for an operating crew of six men.

To reduce the on-site time requirement per anchor and, in particular, the diver bottom time requirement, the fill-in and compaction time appears to be the easiest to reduce. Given the proper earth moving hand tools and vibration equipment, this job could probably be done in less than 3 minutes. Further time reduction could be obtained by using quick release fittings at the top of the anchor and on the hose supplying the water pressure. This could reduce the set-up and breakdown times to about one minute each. Thus, with minimal procedural changes, the diver bottom time per anchor could be reduced to about 7 minutes. This savings would be particularly noticeable when working at depths where the diver can only stay a short time without making decompression stops during ascent. An example would be an operation in 120 feet of water where the divers have only 15 minutes to[1] complete their work and still be within the no-decompression limits.

Cement Slurry Operations

As discussed earlier, many problems were encountered during the slurrying operations. As a result of these problems, and due to the fact that during this phase of the experiment many outside interruptions of the work were necessary, actual slurry time data were not obtained. However, the effort involved indicates that additional design and testing will be required to develop acceptable, inexpensive slurrying equipment and procedures. Future testing should include a trade-off comparison of the effort involved in placing the slurry to the increase in holding power attributed to the concrete.

SOIL PROPERTIES

Several small grab-samples of bottom material were taken from the test site for laboratory testing. The test site soil consisted of partially cemented coarse sand and fine gravel, both derived from coral material. The sand particles were sub-angular to angular in form. Numerous shell fragments were also present. Grain size, density and direct shear analyses were performed on the samples obtained.

Mechanical Analysis

Following mechanical analysis, the representative sample of the bottom material was classified as a poorly graded sand with some large pieces of broken shell and marine rubble. In addition, the mechanical analysis showed that there was little or no crushing during the direct shear tests. This was verified by a standard grain size analysis[2] run before and after each shear test. The results of a typical analysis are in Appendix A, Figures A-1 and A-2.

Density Analysis

To evaluate the range of densities possible in the field, tests were performed to determine maximum and minimum void ratios. The following qualitative definitions are made:

Very loose - the minimum density obtainable in the lab using a technique of submerged sedimentation. (See Appendix A.)

Loose - the minimum density obtainable in the lab using oven-dried samples.

Dense - the sample underwent three minutes of vibratory packing using a load of about four psi.

Direct shear tests were run on only the dense and loose samples; the relatively large amount of difficulty involved in preparing samples by submerged sedimentation made it impractical to run direct shear tests on very loose specimens. In each case, three densities were measured:

Initial - Density before normal load was applied.

Loaded - Density after normal load had been applied.

Relaxed - Density at end of direct shear test after the shear load (but not the normal load) had been removed.

Quantitative measurements of these procedures and values used are shown in Appendix A, pages A-4 and A-5.

Direct Shear Tests

The direct shear test procedure was standard[2] with the exception that the test samples were oven-dried before the test was performed. A strain-controlled loading unit was chosen so that ultimate resistance and a better measure of the peak resistance could be obtained. Relatively large normal loads were used in order to offset the effects of inherent friction within the test apparatus. A shear displacement rate of approximately .015 in/min was used. Average soil properties for the various test samples are shown below:

AVERAGE SOIL PROPERTIES

	DENSE	LOOSE	VERY LOOSE
Dry Density (lb/Ft^3)	93.	82.	76.
Porosity	0.46	0.52	0.56
Void Ratio	0.86	1.10	1.28
Buoyant Unit Weight (lb/Ft^3)	59.	52.	48.
Angle of Internal Friction	43^{o}	37^{o}	35^{o*}

*This value was assumed since no direct shear tests were run on the very loose samples.

Shear load and volume change versus shear displacement for each test were measured and the results are shown in Figures A-6 through A-8, Appendix A. For all of the shear tests performed, the ultimate and peak strengths were noted and a plot of shear stress versus normal stress was obtained, Figure 6. Peak angles of internal friction were determined by using the best straight line fit through the data (Figure 6), and assuming that the apparent cohesion depicted in the graphs was caused by

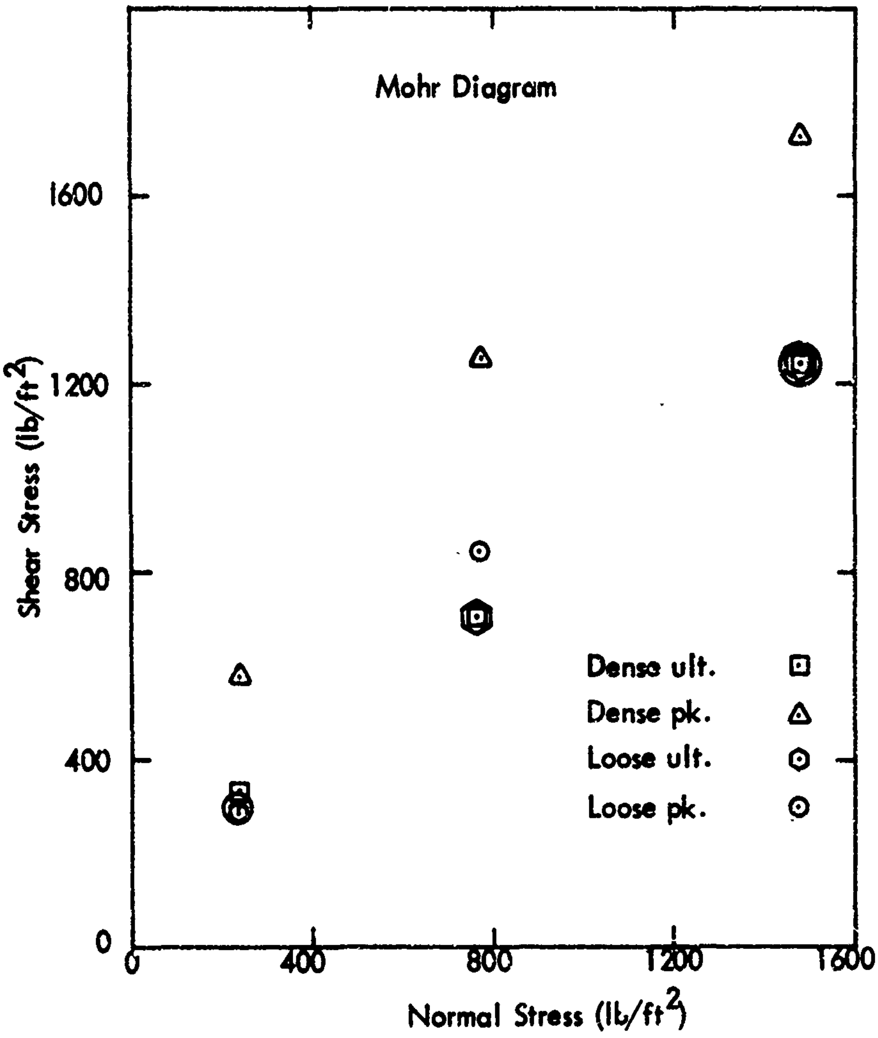

Mohr Diagram
1600
1200
800
400
0
0
400
800
1200
1600
Shear Stress (lb/ft2)
Normal Stress (lb/ft2)
Dense ult.
Dense pk.
Loose ult.
Loose pk.

Figure 6.

frictional resistance in the direct shear device. Peak angles at internal friction were 43° and 37° for the dense and loose sand, respectively.

THEORETICAL ANCHORAGE STUDY

The resisting force or holding capacity of an anchor is due to the anchor weight and to the resistance to movement offered by the confining medium. Vesic[5] suggests that the failure patterns in the overburden soil, which greatly affect the holding capacity, depend on the relative depth of the anchor (depth of burial divided by projected area of anchor), the type of soil and its sensitivity. Several model studies of sand anchors have been conducted but few large-scale field tests have been performed. Kalajian[4] did conduct a large-scale study on holding capacity of marine anchors in sand but no attempt is made to predict the holding capacity of similar anchors based on his experiments.

When predicting anchor holding capacity, three failure mechanisms are normally considered. They are: dead weight theory, cylindrical failure surface theory, and the torical theory.

Dead Weight Theory

This theory is designed to predict the minimum holding capacity developed by a jetted anchor. It applies only when there is no friction developed between the backfilled cylinder above the cone and the undisturbed soil. The theory assumes that the maximum anchor pullout capacity (Q_{max}) is numerically equal to the effective weight of the projected cylinder of sand above the anchor, that is;

$$\boxed{Q_{max} = \gamma_b \cdot A \cdot d} \qquad \text{Equation (1)}$$

where γ_b = buoyant unit weight of soil, A = projected anchor area, d = original depth of embedment.

Cylindrical Failure Surface Theory

This theory is particularly applicable for predicting the maximum resistance to breakout of an embedment anchor which is jetted into partially cemented or very dense granular materials, then backfilled with little or no densification of the backfill material.

In-situ observations indicated that the cylinder of sand jetted out by the anchor upon emplacement, when backfilled, did not reach a density comparable to that of the surrounding sediment, Figure 7. Therefore, it is reasonable to assume that the resistance to pullout developed by the anchor was equal to the effective weight of the cylinder of sand above

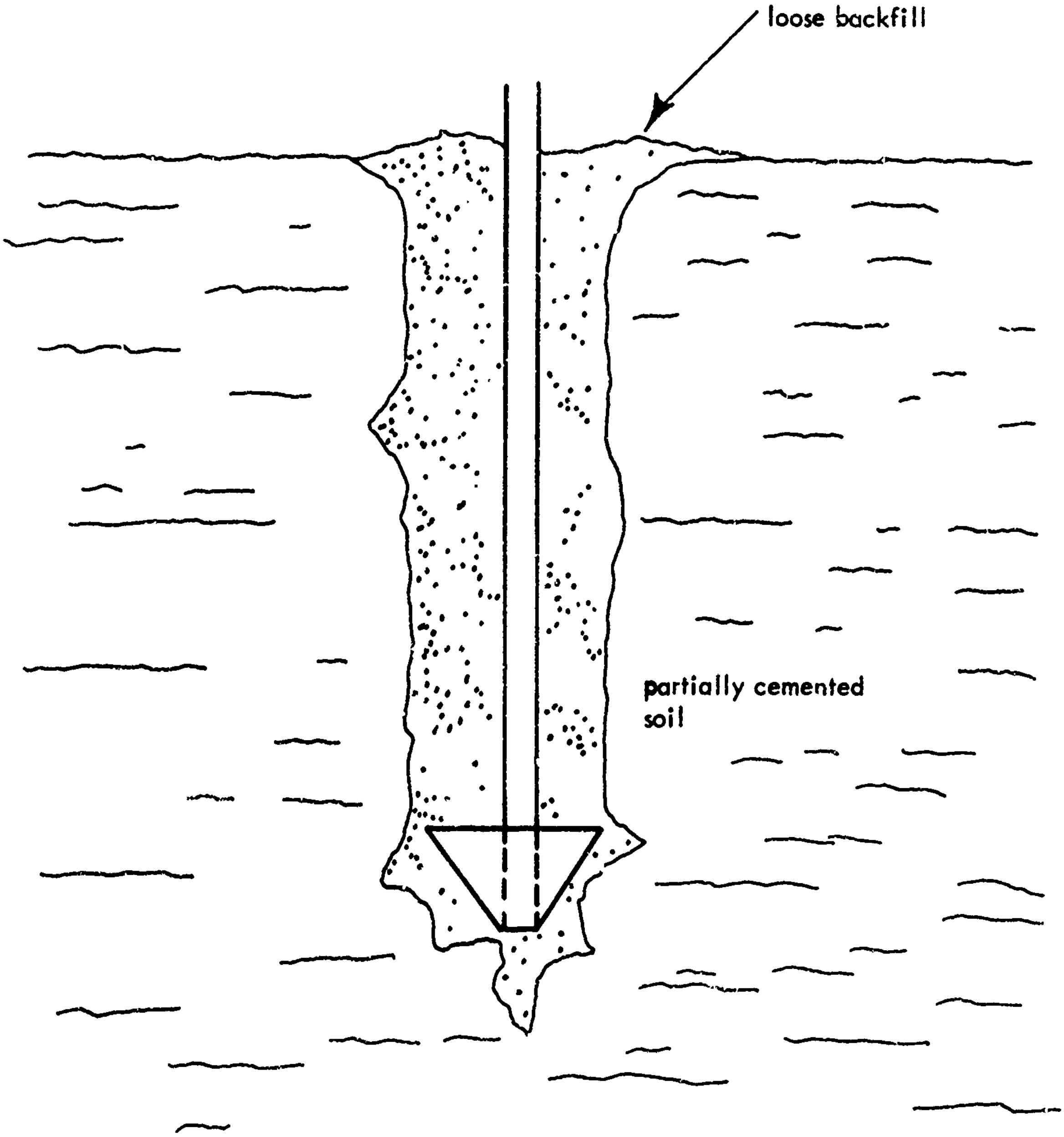


Figure 7. Anchor after emplacement.

the anchor plus whatever friction was developed at the cylinder walls upon pullout. Figure 8 shows a simpler failure surface than described by Vesic[5] (see Figure 9) for well-compacted soil but it seems reasonable in the present case.

Following is the development of the cylindrical failure surface theory.

Definition of Terms:

Q_{max} - maximum anchor pullout capacity

A - projected area of anchor $[Ft^2]$

K_o - coefficient of lateral earth pressure at rest

d - depth of embedment of anchor $[Ft]$

σ_v - vertical stress $[lb/Ft^2]$

σ_n - horizontal stress $[lb/Ft^2]$

B - cone diameter $[Ft]$

ϕ - angle of internal friction

p - perimeter of cone at base $[Ft]$

dz - differential depth measurement $[Ft]$

Derivation:

Assume Q_{max} = weight term and friction term where;

friction term = $\int_o^d \sigma_n (z) \cdot \tan \phi (z) \cdot p \cdot dz$

and the weight term = $\gamma_b \cdot A \cdot d$

Then

$$Q_{max} = \gamma_b \cdot A \cdot d + \int_o^d \sigma_n (z) \cdot \tan \phi (z) \cdot p \cdot dz$$

as first approximation assume:

(a) $\phi \neq$ function of depth

(b) $\sigma_n = K_o \cdot \gamma_b \cdot Z$

(c) No arching of the soil

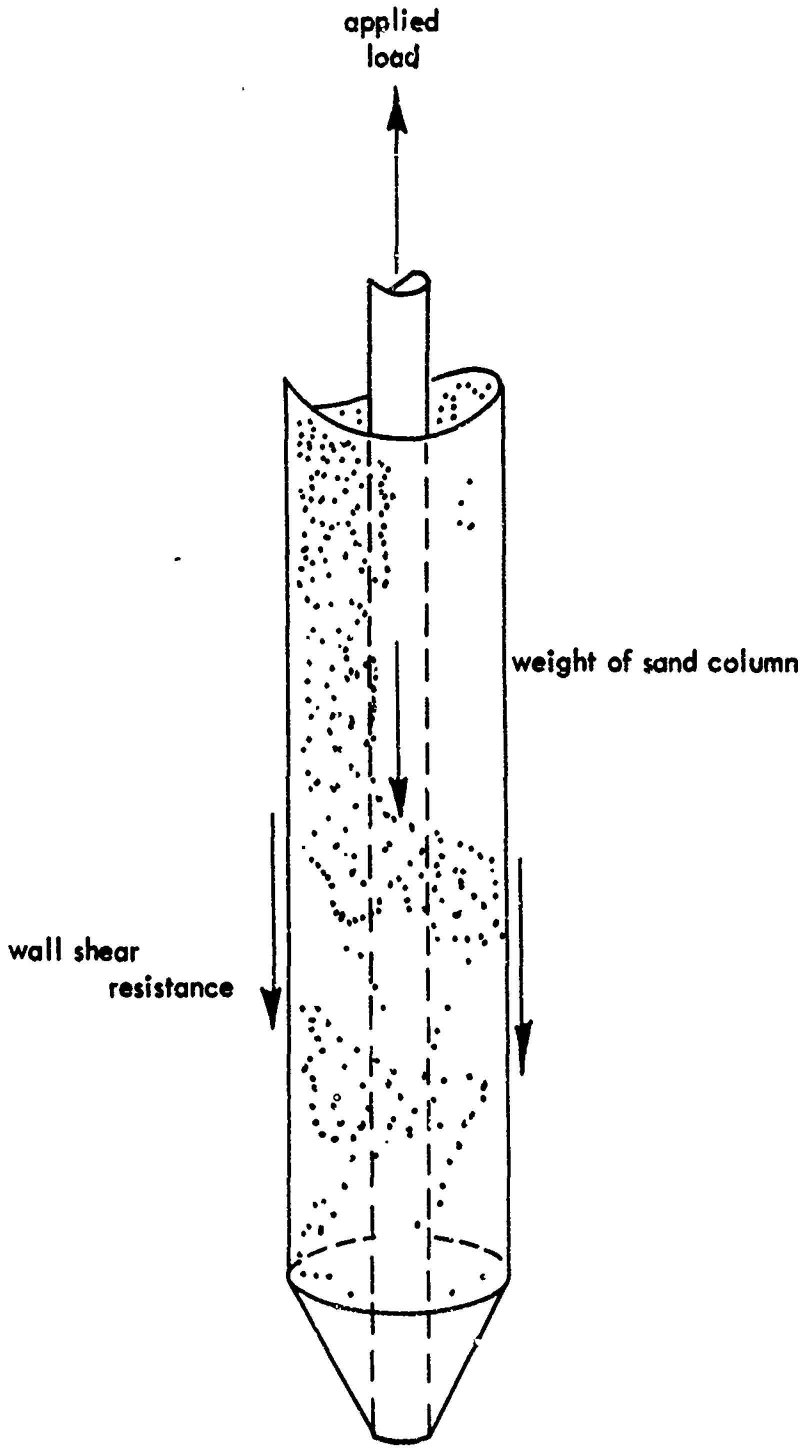


Figure 8. Cylindrical failure surface.
(Free body diagram)

Then

$$Q_{max} = \gamma_b \cdot A \cdot d + \int_o^d K_o \cdot \gamma_b \cdot p \cdot \tan \phi \cdot z \cdot dz$$

$$\boxed{Q_{max} = \gamma_b \cdot A \cdot d + K_o \cdot \gamma_b \cdot p \cdot \tan \phi \cdot \frac{d^2}{2}} \qquad \text{Equation (2)}$$

Torical Slip Surface Theory

It is suggested by Vesic[5] that the assumption of a torical slip surface, Figure 9, will yield the maximum possible effective resistance of the involved soil mass. This assumption is based on observations in small-scale model tests with anchor plates and anchor piles at Duke University. It is noted that this shape occurs only in the case of relatively shallow anchors in dense sand or stiff silty clay[4]. It is evident that the difference between the soil weight for an assumed cylindrical slip surface is small for small diameter objects at shallow depths, but may be very significant for circular objects at greater depth. The predictions for a cohesionless material, based on this theory, follow:

$$\boxed{Q_{max} = \gamma_b \cdot A \cdot d \cdot \overline{N}_q} \qquad \text{Equation (3)}$$

where A, d, and Q_{max} have previously been defined and $\overline{N}_q$ is a theoretical breakout factor that may be obtained from Figure C-1, Appendix C. The values obtained from Figure C-1 should be adjusted according to the following procedure:

For very loose sand

$d/B \le 2$: Use $\overline{N}_q$ at that d/B

$d/B \ge 2$: Use $\overline{N}_q$ at $d/B = 2$

*For loose sand

$d/B \le 4$: Use $\overline{N}_q$ at that d/B

$d/B \ge 4$: Use $\overline{N}_q$ at $d/B = 4$

For dense sand

$d/B \le 10$: Use $\overline{N}_q$ at that d/B

$d/B \ge 10$: Use $\overline{N}_q$ at $d/B = 10$

*This value was assumed.

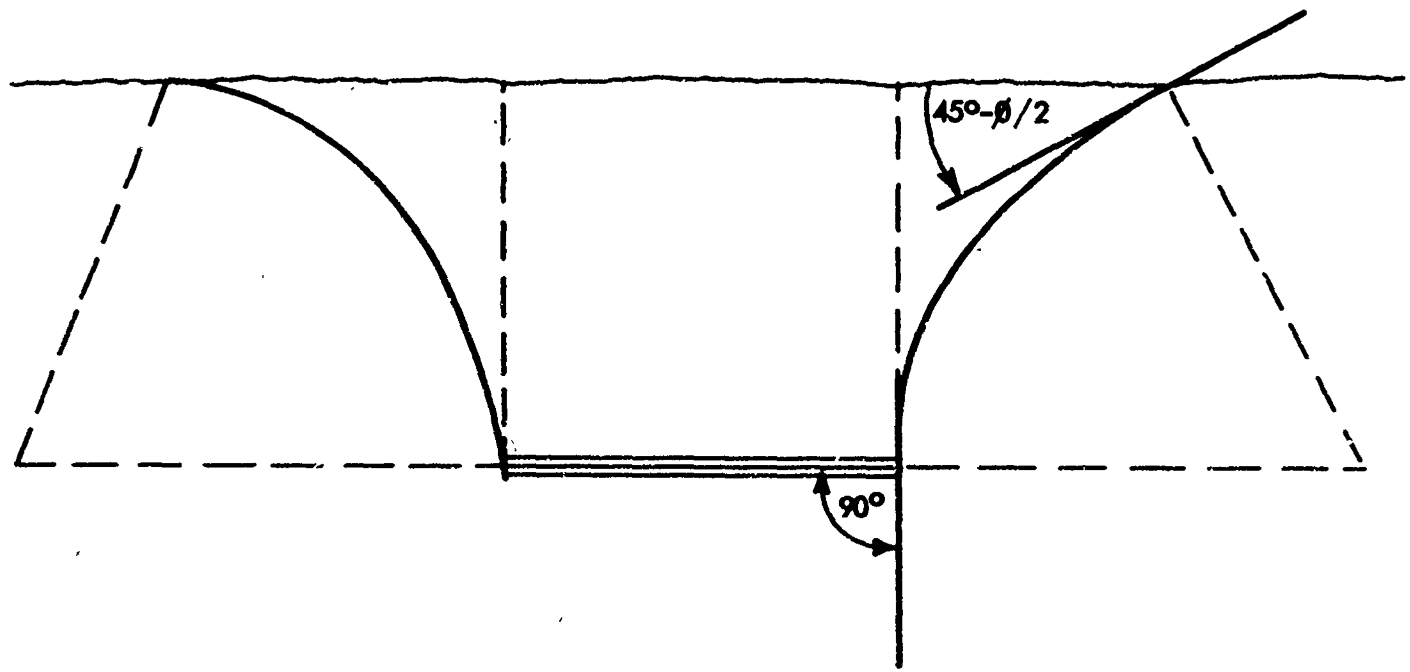


Figure 9. Shape of slip surface for circular buried objects.
(From Vesic[5])

This modification is necessary because available experimental evidence from experiments on 3" Ø plates suggests that the critical relative depth D/B above which embedded objects should behave as shallow anchors depends upon the relative density of the soil. This limiting depth increases from D/B = 2 for a very loose sand to D/B = 10 for a dense sand (Vesic[5]).

ANCHOR PULLOUT RESULTS

The field tests indicated that increasing the cone diameter generally increased the anchor holding capacity. Varying the cone angle and nozzle size had no apparent effect on the anchor holding capacity. In addition, increased depth of burial increased the holding power, as would be expected. Those anchors that were jetted with a cement slurry generally had a higher holding capacity than those without the cement slurry.

Unslurried Anchors

Figures B-1 to B-5, Appendix B, depict graphically the holding capacity versus the depth of embedment for the unslurried anchors. An examination of the figures shows that the anchors seemed to fail by two distinct mechanisms.

1. The anchor displaced at almost constant load until breakout occurred. In this case, the pullout force required was small.

2. The sustained load which the anchor held increased uniformly to a maximum and then dropped off uniformly with increasing displacement. In this case, the required pullout load was somewhat higher.

These two cases are illustrated in Figure 10. A comparison of this figure with Kalajian's data*, Figure 11, which shows results of similar pullout tests in loose (relative density, Dr, < 40) and dense sand (Dr < 80), indicates that there is a definite correspondence between in-situ density and force-displacement curve shape. Therefore, it should be possible to infer the in-situ relative densities from the shape of the field force-displacement curves. This inference has been made, as no measurements of in-situ density were possible.

Data from the anchor tests and corresponding theoretical predictions of anchor holding capacity are presented in Table I. The qualitative descriptions of relative density were inferred from the field force-displacement relationships. Equations (1), (2), (3) and the average soil properties determined during the direct shear tests were utilized in making the theoretical predictions indicated.

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*E. H. Kalajian and S. N. Bemben[4] conducted an investigation of the vertical pullout capacity of marine anchors embedded in sand by vibration. He also notes that there appears to be two mechanisms of failure within the soil mass.

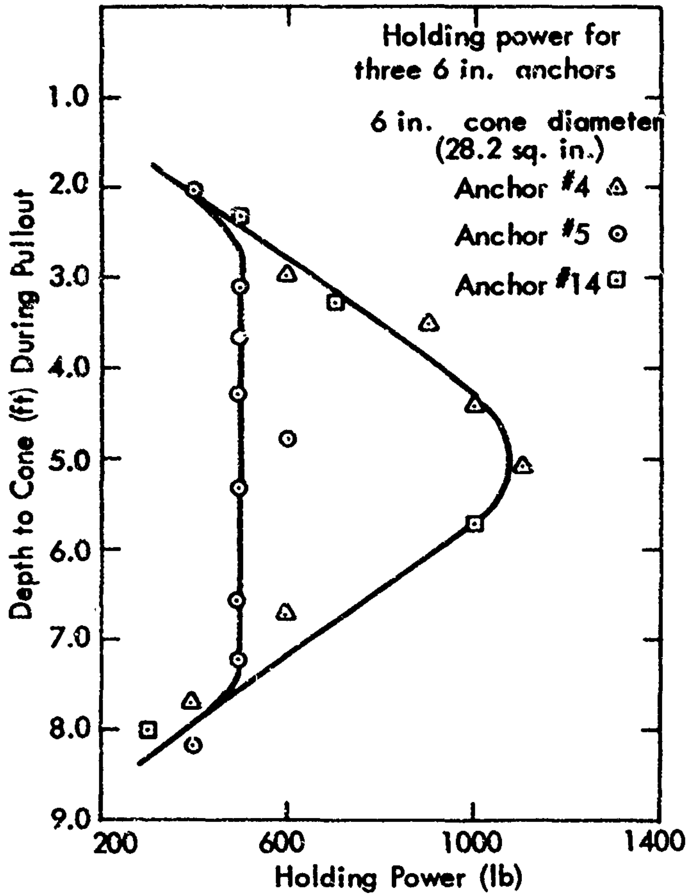

Figure 10.

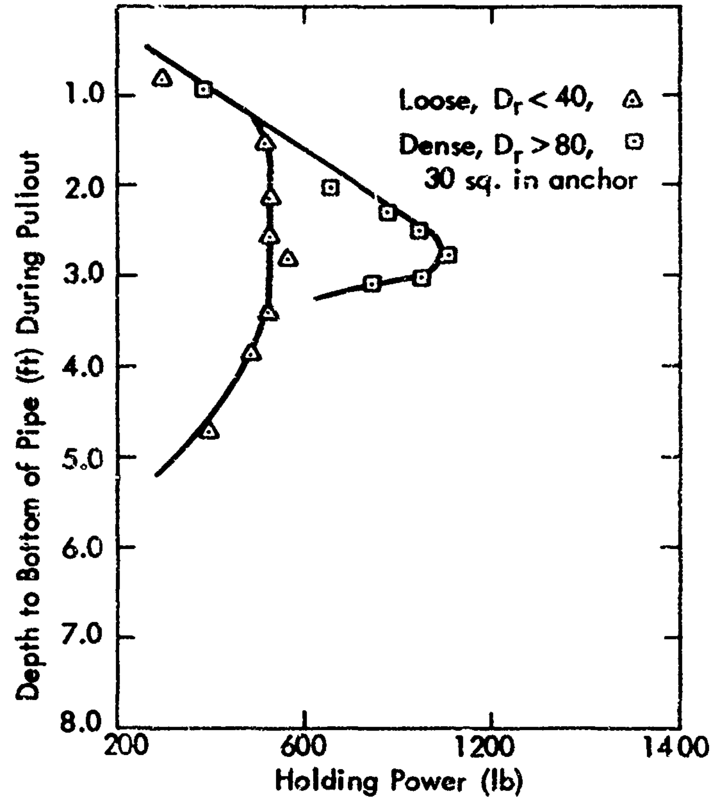

Figure 11. (Data from Kalajian[4])

TABLE I

ACTUAL AND THEORETICAL HOLDING CAPACITIES

SITE NUMBER	DEPTH OF BURIAL	CONE DIAMETER	STATIC Q_{max} MEASURED	Q_{max} CYL	Q_{max} TORICAL	Q_{max} DEAD WT.	RELATIVE* DENSITY
	FEET	INCHES	POUNDS	POUNDS	POUNDS	POUNDS	
4	7.71	6	1100	993	735	79	Loose
5	8.17	6	600	1046	308	77	V. Loose
14	8.02	6	1000	1079	772	83	Loose
18	8.04	15	2300	2979	4820	517	Loose
6	7.94	9	1900	1574	1710	184	Loose
7	7.90	9	1900	1553	1700	183	Loose
13	8.67	9	1100	1815	740	185	V. Loose
25	8.63	9	800	1744	736	184	V. Loose
9	7.68	9	2800	1760	8200	200	Dense
11	7.28	9	2000	1600	7800	190	Dense
12	8.71	9	3300	2248	9120	228	Dense
16	7.67	9	900	1413	650	163	V. Loose
19**	8.54	9	3700	-	-	-	-
20**	8.50	9	7400	-	-	-	-
23***	8.96	9	2800	-	-	-	-
24***	8.75	9	4000	-	-	-	-
2	6.29	12	1800	1489	2410	259	Loose
3	6.88	12	1200	1743	2630	283	Loose
15	8.15	12	900	2249	1236	309	V. Loose
17	7.75	12	3300	2570	10,000	360	Dense
8	8.54	12	1600	2621	3264	351	Loose
10	8.00	12	800	2073	1210	303	V. Loose

*Relative density inferred from general shape of holding power vs displacement curve for each anchor

**Cement slurry used under cone

***Cement slurry used over cone

Values of K_o used in the calculations were assumed as follows:

Very loose	K_o = .56
Loose	K_o = .52
Dense	K_o = .43

Explanation of Data

From Table I, it is readily apparent that the dead weight approach to predict holding capacity is very conservative. This occurs because the soil is assumed to exhibit no shear strength which is in contradiction to the results obtained from the direct shear tests previously presented.

Results of the Torical and Cylindrical analyses are also presented in Figure 12. Actual holding capacity is plotted against theoretical holding capacity. Ideally, the data should fall on a 45° line through the origin but it is clearly evident that this is not the case. The results of the Torical solution for dense sand were much greater than the actual results and could not be plotted realistically with the rest of the data. A possible explanation is that the backfilled soil was not actually in a dense state and therefore the coefficient $\bar{N}_q$ used in holding capacity calculations was much too large and resulted in much greater predicted than actual values of holding capacity.

There are fallacies involved in using either the Cylindrical or Torical failure criteria for all values of d/B for this problem. The cylindrical failure theory assumes a general shear type failure at all values of d/B which is simply not the case. A localized or punching type failure occurs in very loose sand at $d/B \geq 2$ and in a dense sand at a $d/B \geq 10$ (Vesic[5]).

The Torical failure theory for the particular boundary conditions imposed on the anchor is not applicable. Torical theory assumes that the soil through which the failure surface will form is uniform. Because the soil is backfilled with very little control over its final density, it is doubtful whether the jetted out soil could be emplaced at its in-situ density.

If the backfilled soil is denser than the surrounding soil, then its failure mode should be controlled by the surrounding soil. However, if the backfill is less dense than the surrounding soil, its failure mode should be controlled by the backfill. Since in-situ density was not determined, Torical theory should not be used.

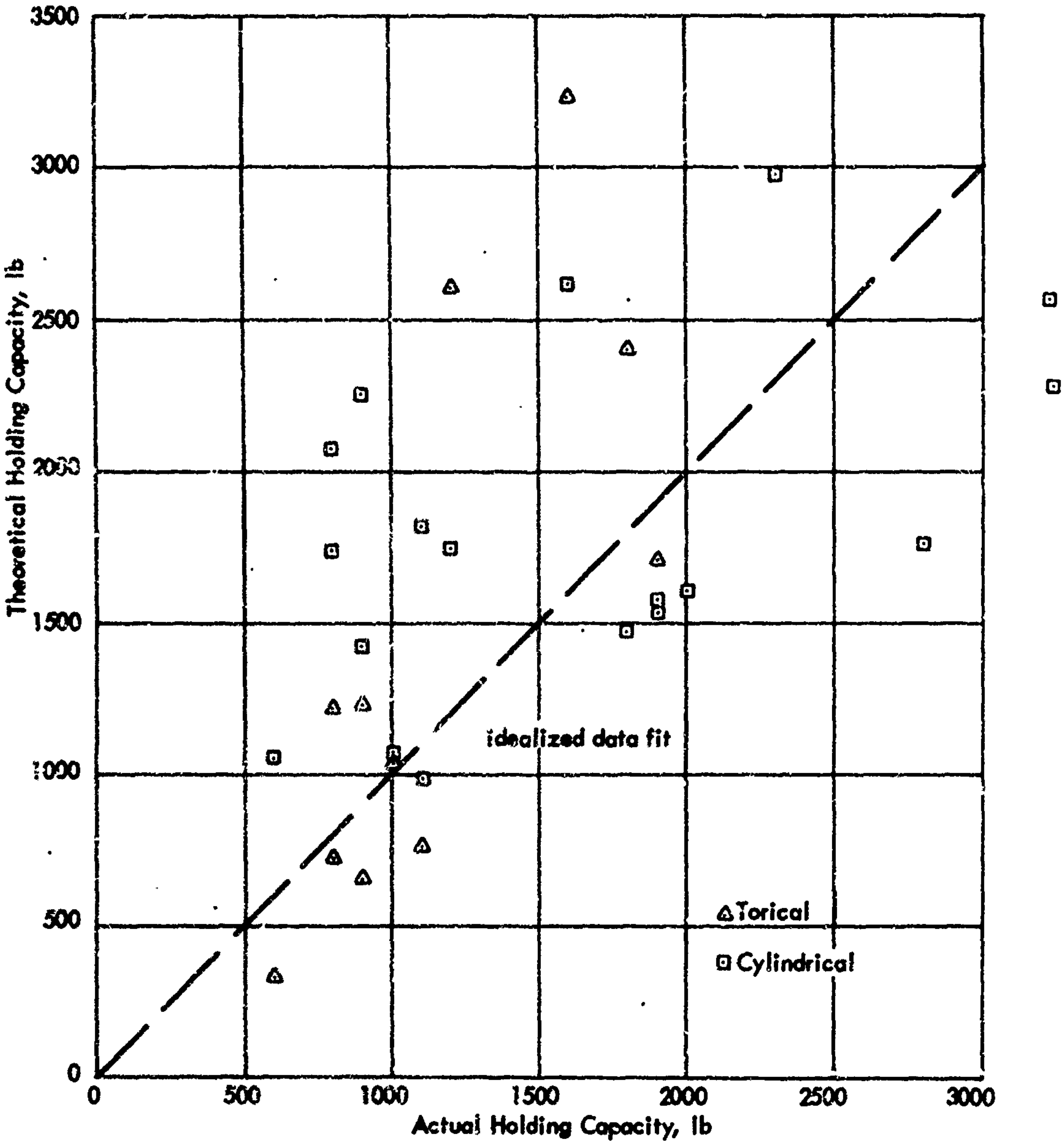

Figure 12. Theoretical verses actual holding capacity for embedment anchors.

Cement Slurried Anchors

Tabulated also in Table I is the approximate depth of embedment versus the holding capacity for each anchor upon which the cement slurry was injected.

The four anchors slurried showed markedly greater holding powers than the non-slurried anchors. The mean maximum sustained load of these 4 was 4,475 lbs whereas the corresponding mean for unslurried anchors of the same size was 1,838 lbs. Two of these anchors were slurried through the apex of the cone. After testing, traces of concrete were found adhering to the surface of the cone and concrete in the shaft was protruding from the tip of the cone which apparently had broken loose from the concrete under the anchor.

The other two slurried anchors were slurried by injecting the mixture alongside the anchor shaft above the cone as discussed above. The slurry was injected approximately 2 feet above the top of the anchor cone and formed upward displacing the loose sediment above. Evidence of some bonding with the surrounding sediment structure was noted but the primary increase in holding power for this case is attributed to the increased friction surface between the anchor and the side of the anchor hole as the anchor was extracted.

For the fourth anchor slurried, only about half the full slurry load was emplaced, as discussed earlier. This anchor exhibited increased holding power, although no cement was found adhering to the anchor after testing.

The use of a cement slurry to form additional holding power for the jetted anchor was thus a limited success. The experimental evidence indicates increased holding power due to slurry use; however, the small number of tests performed precludes the possibility of drawing qualitative conclusions. A more controlled experiment, using a large number of anchors and an adequately engineered emplacement procedure, is indicated in order to obtain conclusive evidence of the value of the cement slurry for increased holding power.

FINDINGS AND CONCLUSIONS

From cost and manhour requirement considerations, the jetted-in cone anchors tested here seem to be a feasible means of obtaining easily emplaced, light duty, bottom tie-downs and anchorages . The anchors required approximately 1.2 man hours each to emplace using a six man crew. Several means have been suggested which could reduce the emplacement time and, in particular, considerably reduce the diver bottom time required per anchor. In general, the anchors proved easy to handle and emplace. No special skills, other than diving, were required of the Seabee enlisted personnel which emplaced them.

Results of the holding capacity tests and the resulting analysis indicate that the jetted cone anchors can develop holding powers within the desired range of 2,000 to 10,000 lbs . The primary determining parameters of holding power are the anchor size, the depth to which the anchor is jetted and the compaction obtained during backfilling of the hole. The variation of cone angle or nozzle size showed no measurable effect on the holding capacity of the anchors.

Correlation of actual with predicted values of holding capacity was not very good. Until better control of the backfill density is realized, it will be very difficult to predict holding capacity with any of the available theories.

The attempt to increase the anchor holding power by injection of a cement slurry proved to be time consuming in terms of topside and diver manhours. Encouraging, although non-conclusive, results were obtained indicating the use of such slurry injection may be a most effective means of increasing the holding power of the anchors tested.

The addition of the cement slurry may have increased the resistance to pullout by: (1) increasing the dead weight of the anchor; (2) increasing the projected area of the anchor; and (3) penetrating into the undisturbed soil. Mechanical and procedural difficulties during the injection of the slurry accounted for the excessive manhour requirement and limited the usefulness of the results.

RECOMMENDATIONS

Further tests using the jetted anchor described here are recommended. These tests should be conducted in both sand and clay soils using a larger number of each size anchor tested. Emphasis should be placed on determination of the effects of anchor size, depth of emplacement and soil properties on the holding power. In addition, further design work is recommended to develop the equipment and procedures for injection of a cement slurry on the anchors. This should be followed by further testing of the concept of increasing the holding power with the slurry. Specific suggestions concerning the implementation of these recomendations follow:

1. All experiments should consider diver human factors and the implementation of procedures which will reduce the effort required to emplace the anchors.

2. The backfill soil, if required as in the present case, should be compacted adequately. Either simple rodding or some form of hand vibrator device should be used. This should cause the holding capacity to approach the values predicted by the Torical theory explained above. Some attempt at compaction of the surrounding soil should be made in any case.

3. Measurements of in-situ soil density should be made before anchor pullout in order to facilitate mathematical analysis of the anchor failure mechanism. It is suggested that a cone penetrometer or similar device be used for this purpose.

4. Larger cone diameters should be tested and more variation in depth of burial should be used. This would bring the ratio of embedment depth/cone diameter down into a range covered more adequately by previous tests.

5. The use of flukes on the anchors which would open after emplacement should be investigated.

6. When cement slurry is used, procedures for obtaining accurate placement of the slurry must be instituted. Excavation around a few slurried anchors may be useful in determining by what mechanism the cement is increasing the holding power.

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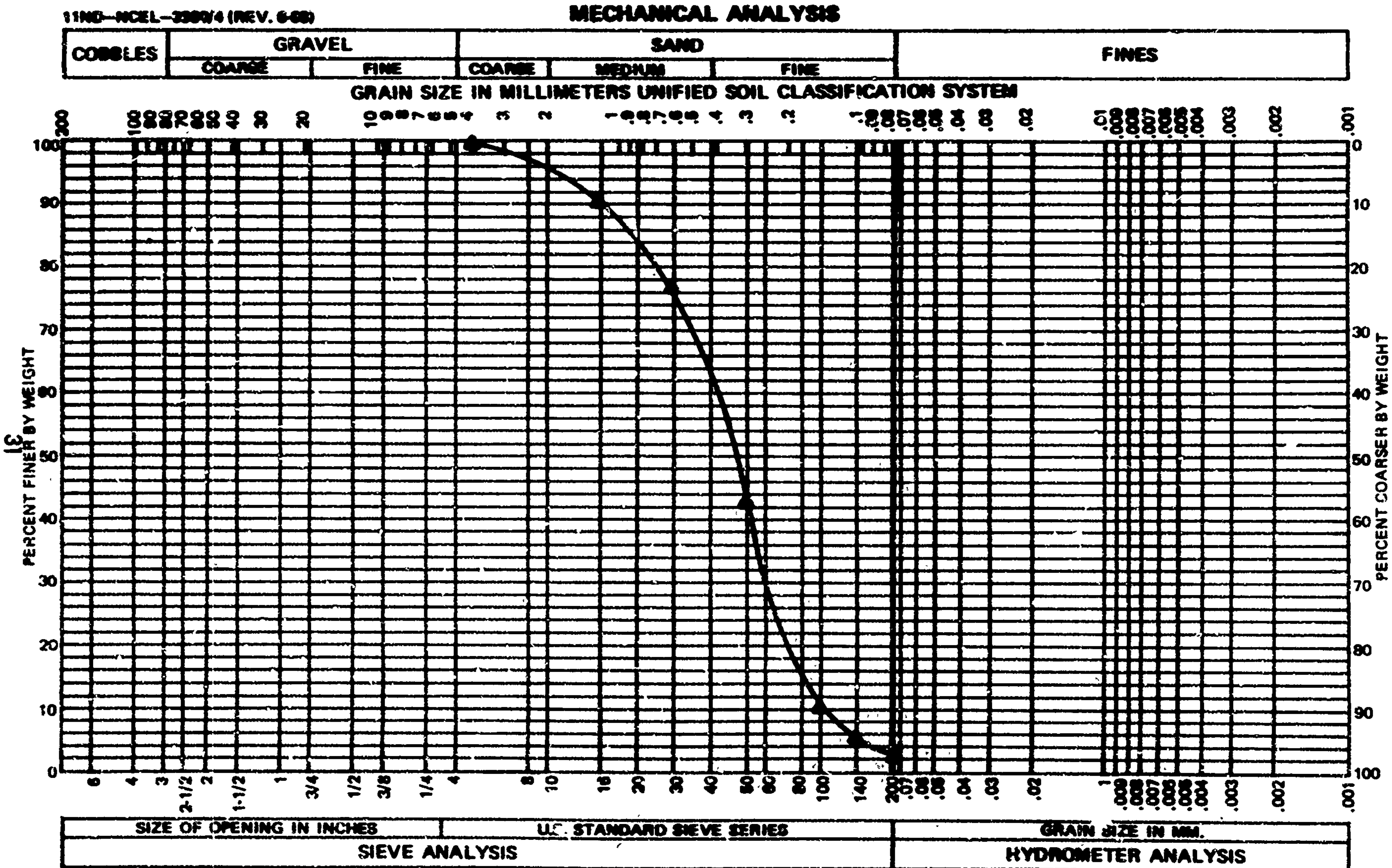
The authors gratefully acknowledge the work of Commander W.J. Eager of the Naval Facilities Engineering Command on this experiment. The jetted-in cone anchor concept, which originated at the Naval Electronics Laboratory, San Diego, California, was developed further by Commander Eager and he ran preliminary on-site tests of several anchors. Commander Eager originated the concept of using a cement slurry to increase holding power and developed the static test apparatus used. He also developed the basic procedure for the experiment and set forth the idea of running a parametric study of the factors affecting the anchor holding power. Without his efforts before and during the testing, this experiment would not have been possible.

Senior Chief Utilitiesman R. Miller provided much assistance in organizing and supervising the Seabee Divers who tested the anchors. His efforts and those of the Seabees who constructed and tested the anchors are greatly appreciated.

The assistance of Messrs. H. G. Herrmann and Homa J. Lee of the Naval Civil Engineering Laboratory is gratefully acknowledged. They provided much needed advice and assistance with the soils testing and the use of the holding power prediction theories.

APPENDIX A

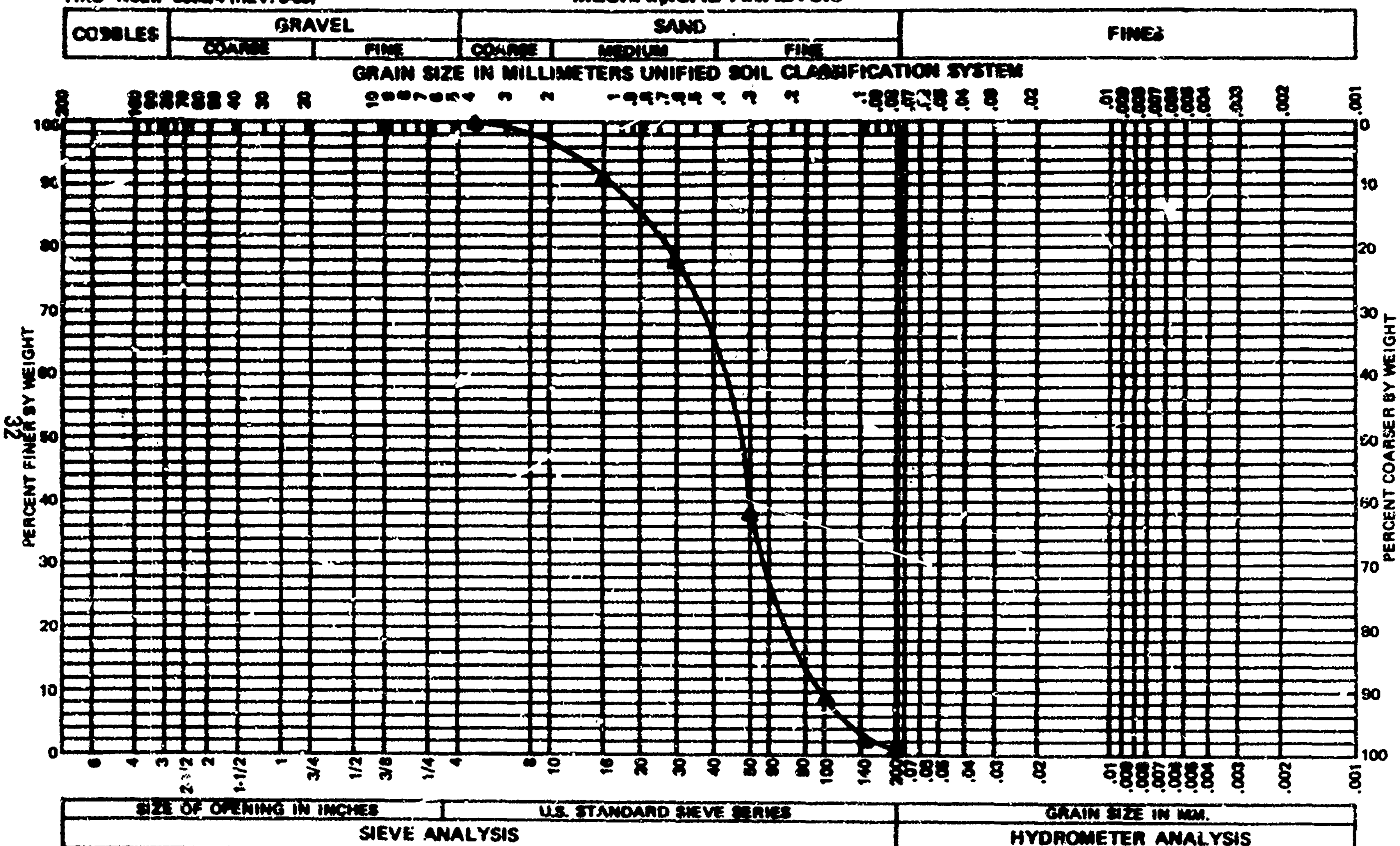
LABORATORY SOIL TESTS



JOB	LOCATION	PLOTTED BY	DATE
ANALYSIS DONE BEFORE DIRECT SHEAR TEST USING 771 lb/ft^2 NORMAL LOAD		W. A. Venezia	7/23/69

11NO-NCEL-3000/4 (REV. 6-68)

MECHANICAL ANALYSIS



JOB	LOCATION	PLOTTED BY	DATE
ANALYSIS DONE AFTER DIRECT SHEAR TEST USING 771 lb/ft^2 NORMAL LOAD		W. A. Venezia	7/23/69

LABORATORY DETERMINATION OF VERY LOOSE DENSITY

A. PROCEDURE

A cylindrical glass container of known dimensions was partially filled with fresh water. The weight of the water and container was noted. Dry sand was then carefully spooned into the cylinder, and the volume of the sand; and the combined weight of the sand, water, and container were noted.

B. DATA

Diameter of cylinder .734 inches

weight of cylinder & water	weight of cylinder, water & sand	Height of sand
66.11 gm	81.62 gm	1 53/64 in
71.04 gm	82.96 gm	1 34/64 in
70.45 gm	103.19 gm	3 57/64 in

C. CALCULATIONS

density = weight/volume = (wt of sand [gm]) $(.002205 \left[\frac{lb}{gm}\right])$ /

(height of sand [in.]) (dia. of cyl. [in.] $)^2 \pi/4 \ \frac{1}{1728} \left[\frac{Ft^3}{in^3}\right]$

D. RESULTS

Density $[lb/Ft^3]$ 75.96 75.54, 75.34

CALCULATION OF DRY DENSITY, POROSITY, VOID RATIO, AND BUOYANT UNIT WEIGHT

A. PROCEDURE

During each shear test, three volumes were noted: the initial volume, relaxed volume, and loaded volume. At the end of each test, the test sample was weighed using only the sand that remained in the test cylinder.

B. CALCULATIONS

DRY DENSITY $(\gamma_d) = \dfrac{\text{weight of sample}}{\text{volume of sample}} \left[lb/Ft^3 \right]$

POROSITY $(n) = 1 - \dfrac{\gamma_d}{Gs\gamma_w}$

VOID RATIO $(e) = \dfrac{n}{1-n}$

BUOYANT UNIT WEIGHT $(\gamma_b) = \dfrac{Gs - 1}{1 + e} \gamma_w \left[lb/Ft^3 \right]$

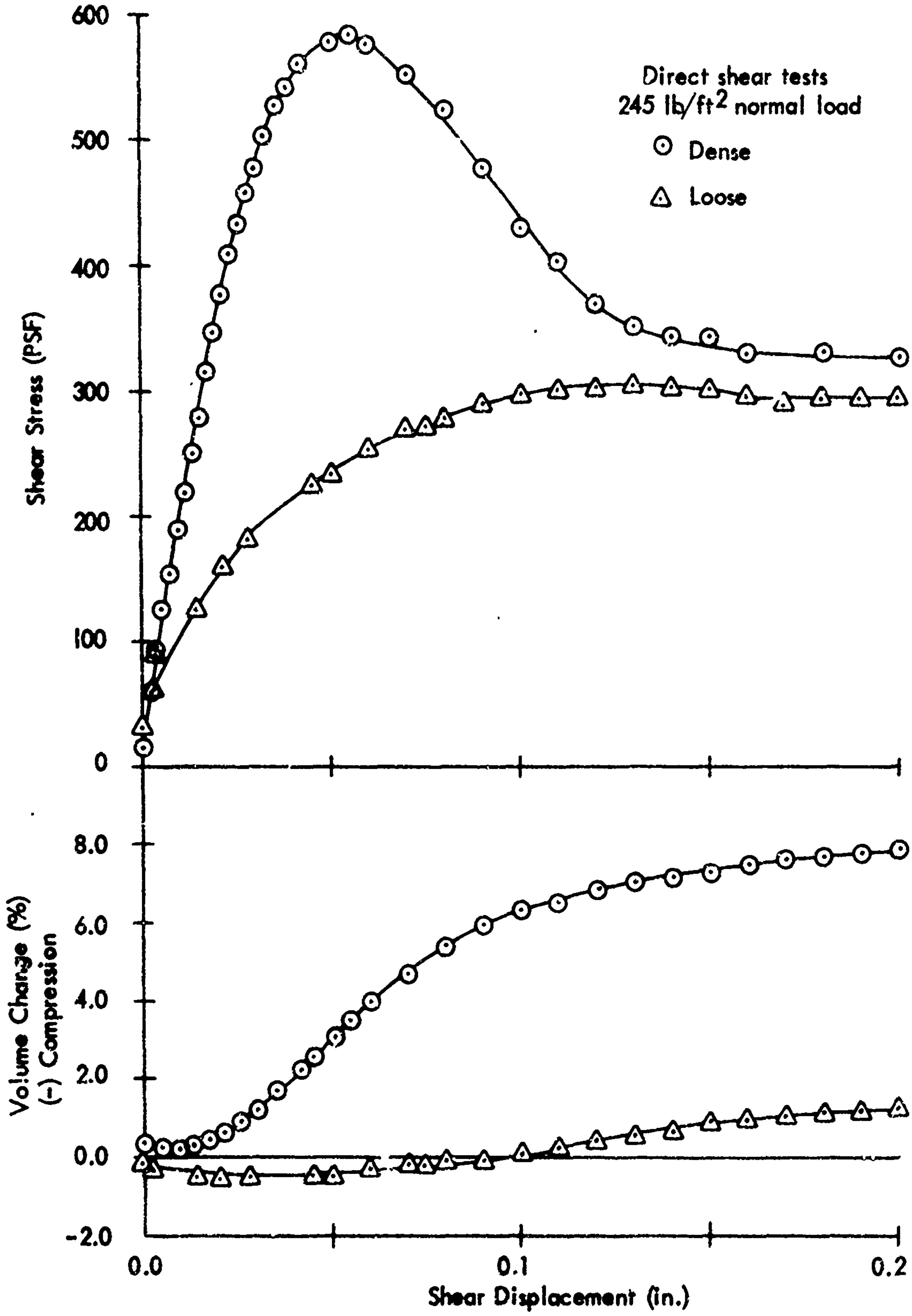
WHERE γ_w = density of water

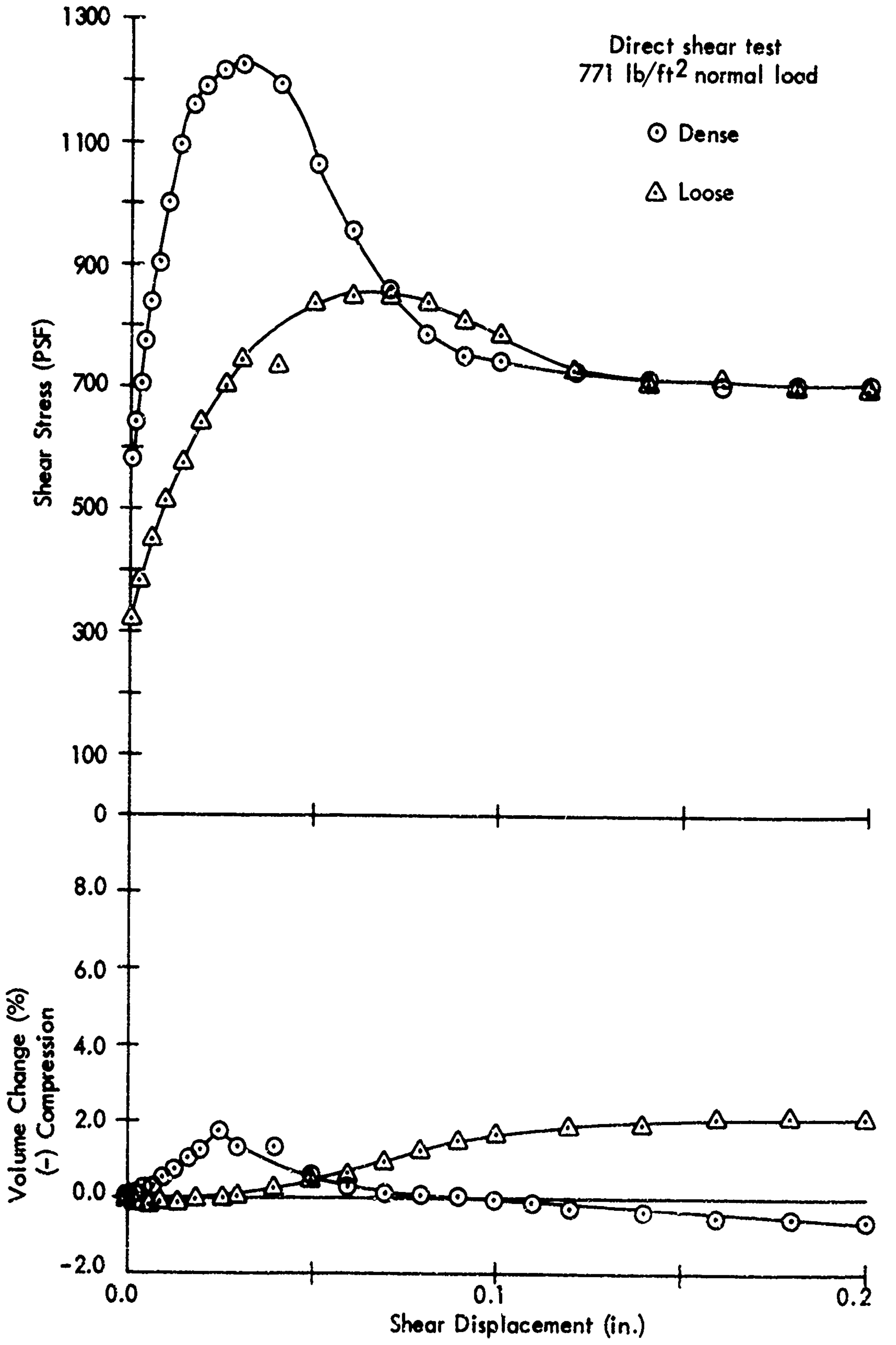
G_s = specificty gravity of sample (2.76) (2.76)

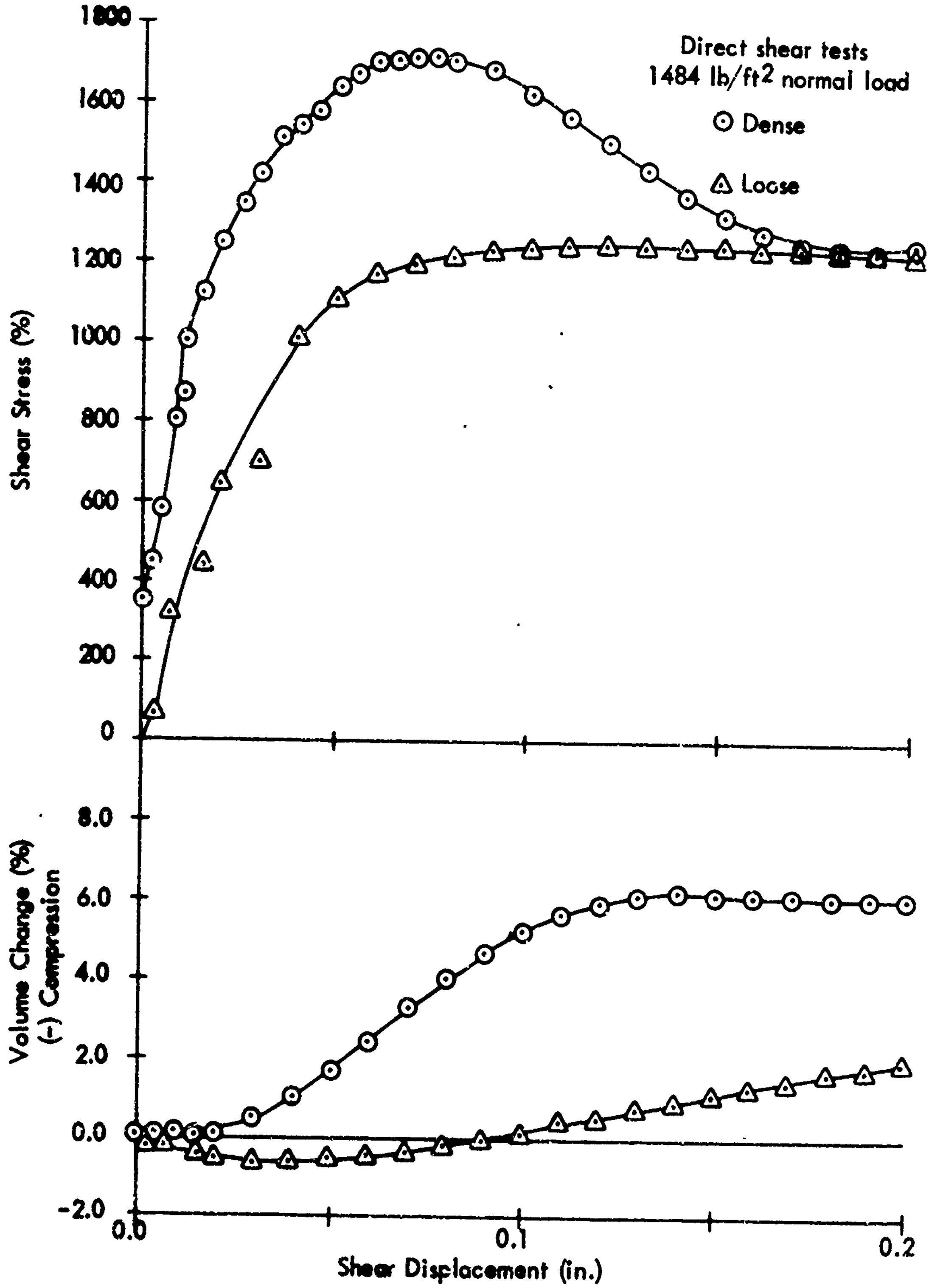
TABLE A-1

DIRECT SHEAR AND DENSITY ANALYSIS RESULTS

	$244^{\#}/Ft^2$	NORMAL LOAD	$771^{\#}/Ft^2$	NORMAL LOAD	$1484^{\#}/Ft^2$	NORMAL LOAD
	DENSE	LOOSE	DENSE	LOOSE	DENSE	LOOSE
DENSITY lb/Ft^3						
INITIAL	95.44	78.24	90.46	80.24	89.18	80.81
LOADED	95.45	78.39	92.53	81.21	90.14	87.41
RELAXED	89.57	78.49	91.02	80.5	86.57	81.28
POROSITY						
INITIAL	.4458	.5457	.4748	.5340	.4822	.5308
LOADED	.4458	.5448	.4628	.5285	.4766	.4925
RELAXED	.4799	.5443	.4715	.5326	.4973	.5281
VOID RATIO						
INITIAL	.8044	1.2012	.9040	1.1459	.9312	1.1312
LOADED	.8044	1.1968	.8615	1.1209	.9106	.97044
RELAXED	.9227	1.1944	.8921	1.1394	.9892	1.1191
PK. STRENGTH lb	20.6	10.8	43.2	30.0	60.5	44.1
PK. STRESS lb/Ft^2	585.4	306.9	1228.	852.6	1719.	1253.
UL. STRENGTH lb	11.6	10.4	25.0	25.0	44.0	43.6
UL. STRESS lb/Ft^2	330.	296.	710.	710.	1250.	1239.

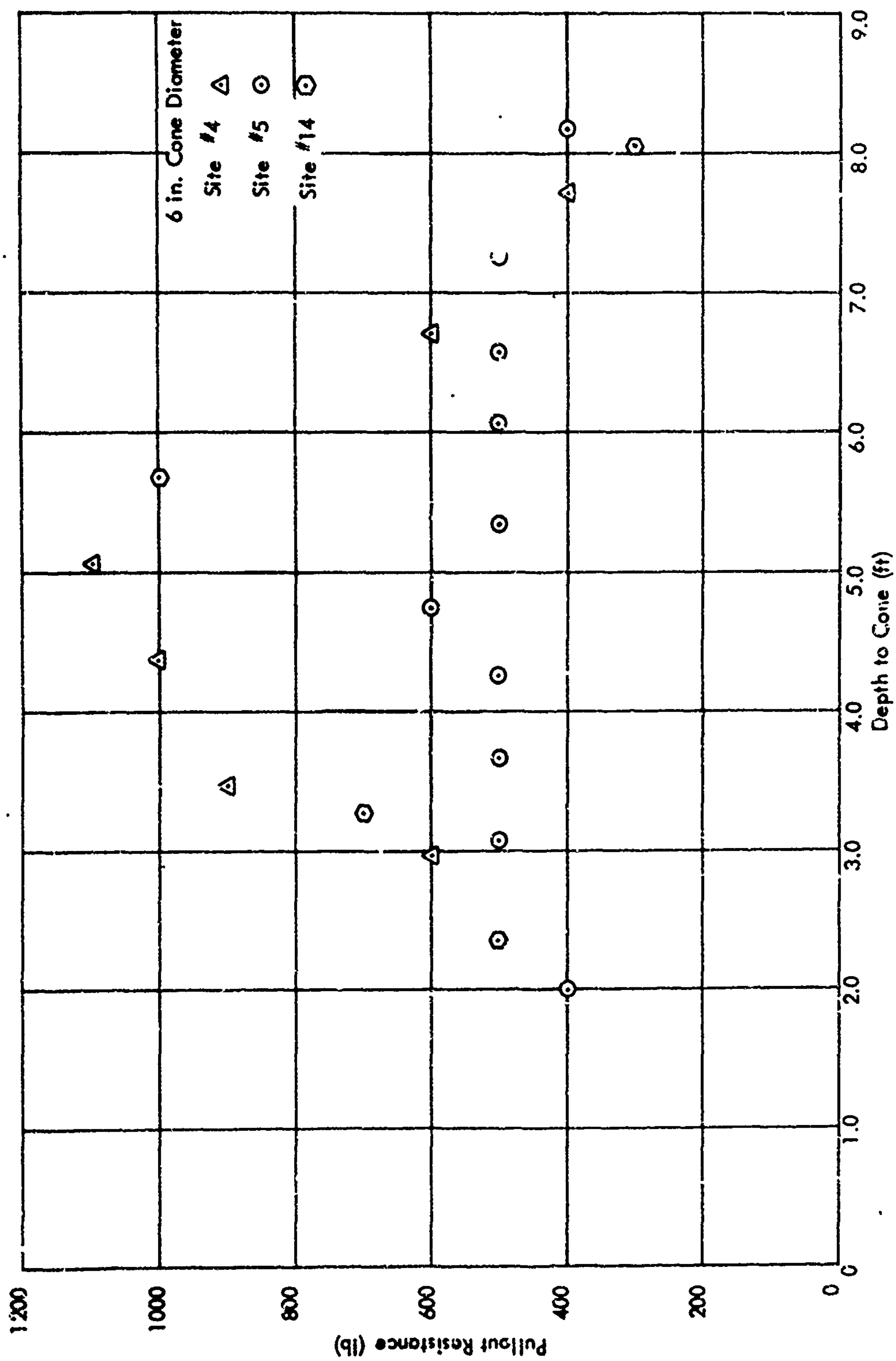

600
500
400
300
200
100
0
Shear Stress (PSF)
Direct shear tests
245 lb/ft2 normal load
Dense
Loose
8.0
6.0
4.0
2.0
0.0
-2.0
Volume Change (%)
(-) Compression
0.0
0.1
0.2
Shear Displacement (in.)


Direct shear test
771 lb/ft2 normal load
Dense
Loose
Shear Stress (PSF)
1300
1100
900
700
500
300
100
0
Volume Change (%)
(-) Compression
8.0
6.0
4.0
2.0
0.0
-2.0
0.0
0.1
0.2
Shear Displacement (in.)

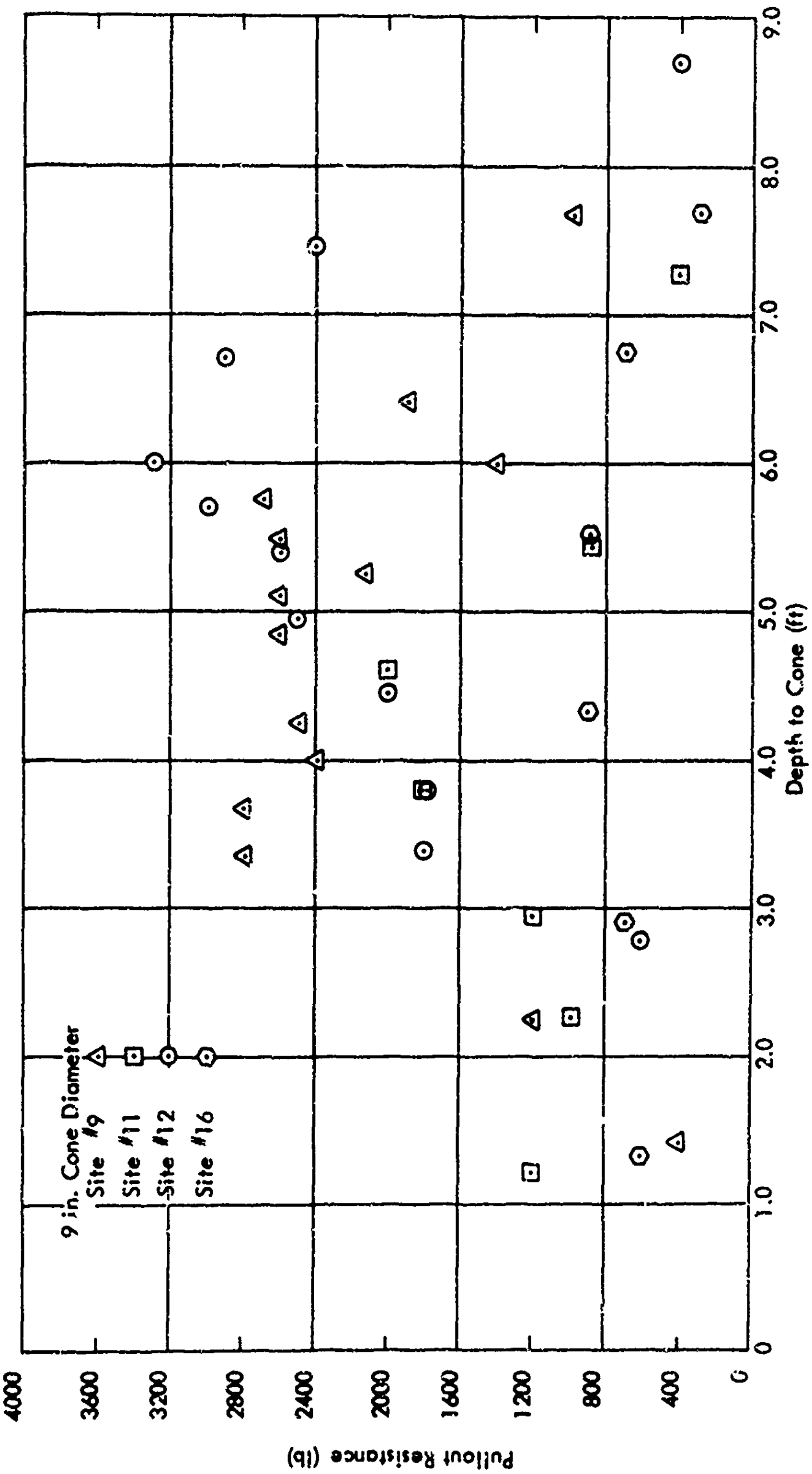

Direct shear tests
1484 lb/ft2 normal load
Dense
Loose
Shear Stress (%)
1800
1600
1400
1200
1000
800
600
400
200
0
Volume Change (%)
(-) Compression
8.0
6.0
4.0
2.0
0.0
-2.0
0.0
0.1
0.2
Shear Displacement (in.)

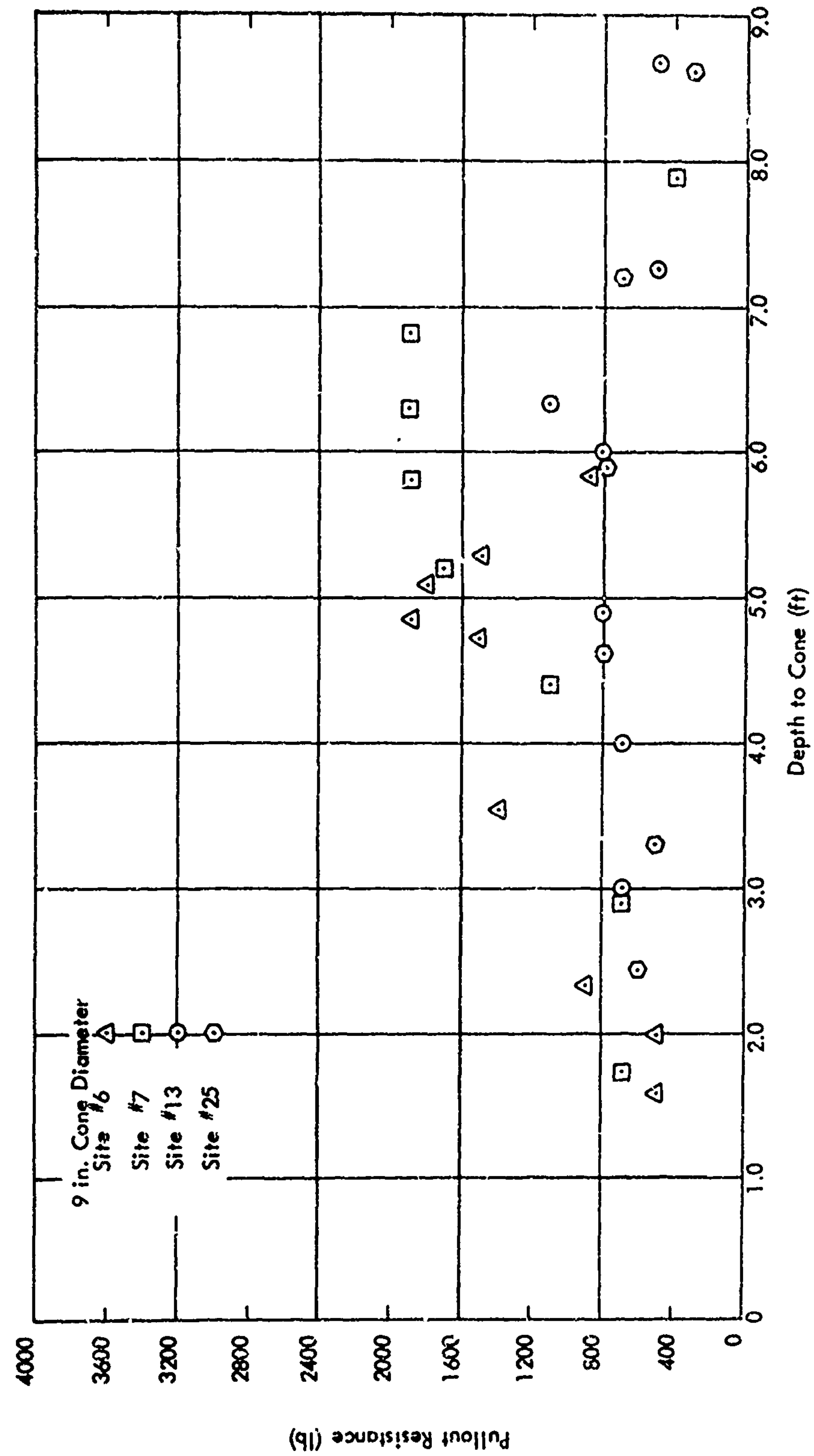
APPENDIX B

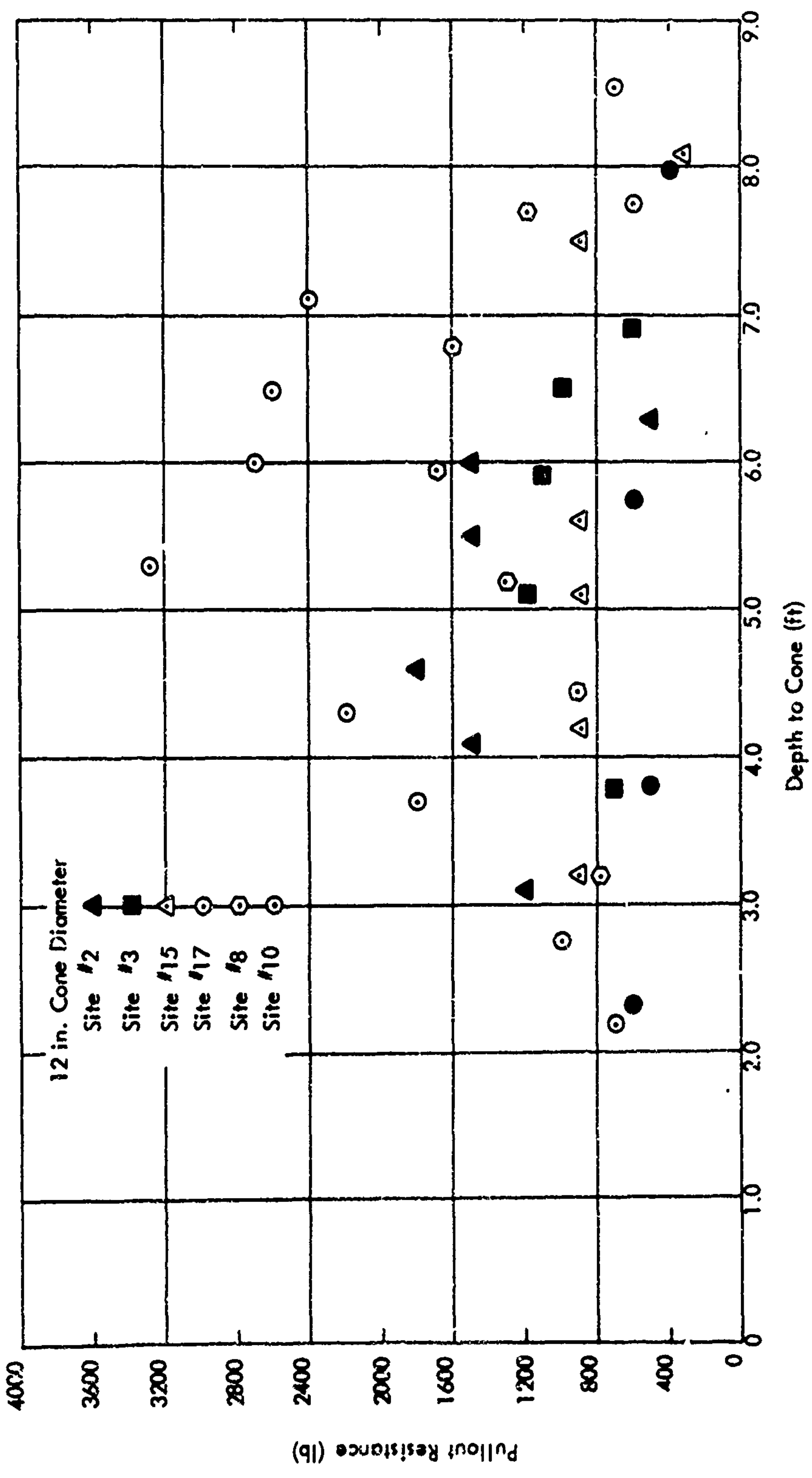
PULL-OUT RESULTS

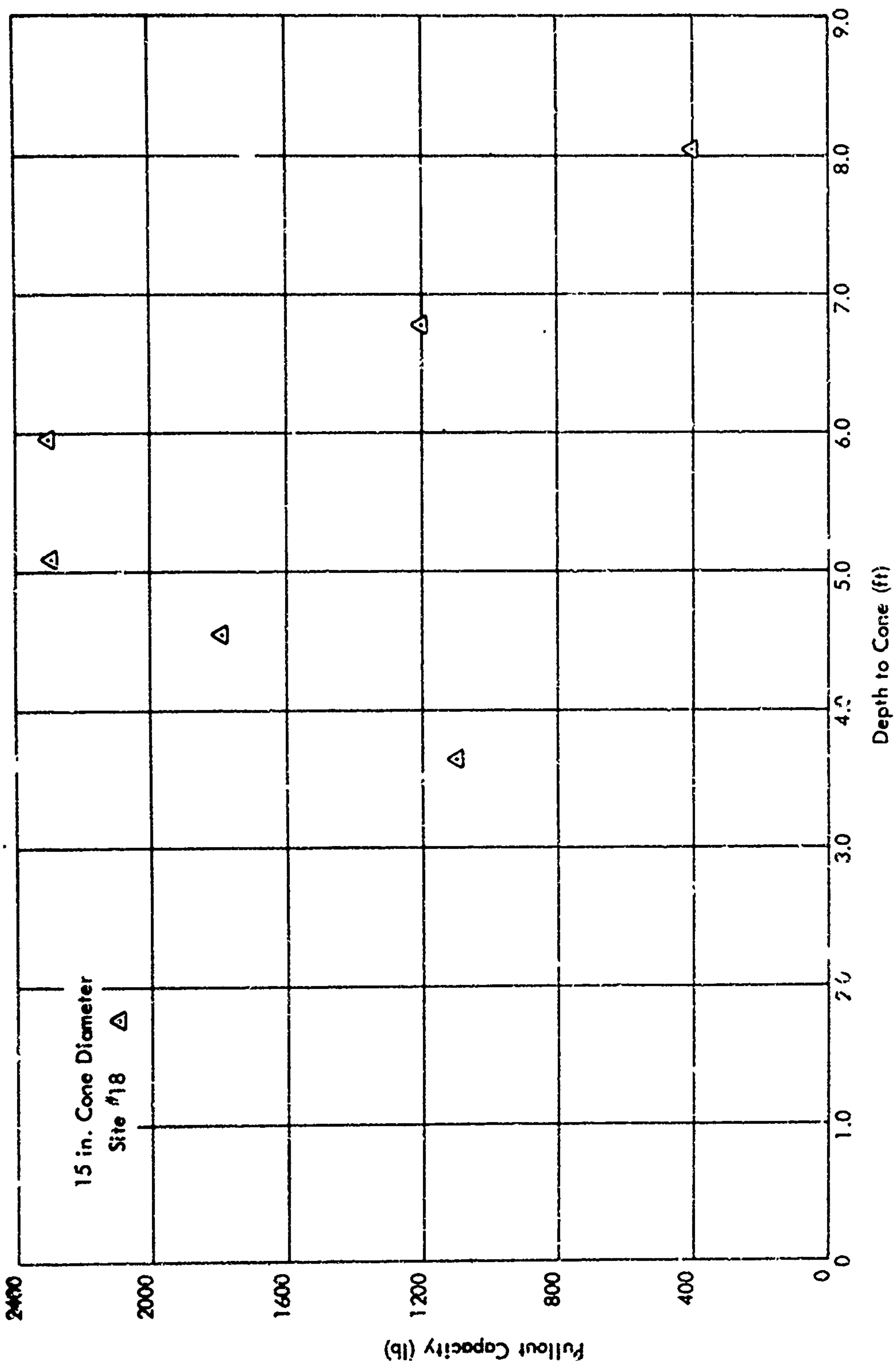


6 in. Cone Diameter
Site #4
Site #5
Site #14
Pullout Resistance (lb)
Depth to Cone (ft)
0
200
400
600
800
1000
1200
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0


9 in. Cone Diameter
Site #9
Site #11
Site #12
Site #16
Depth to Cone (ft)
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
Pullout Resistance (lb)
4000
3600
3200
2800
2400
2000
1600
1200
800
400
0

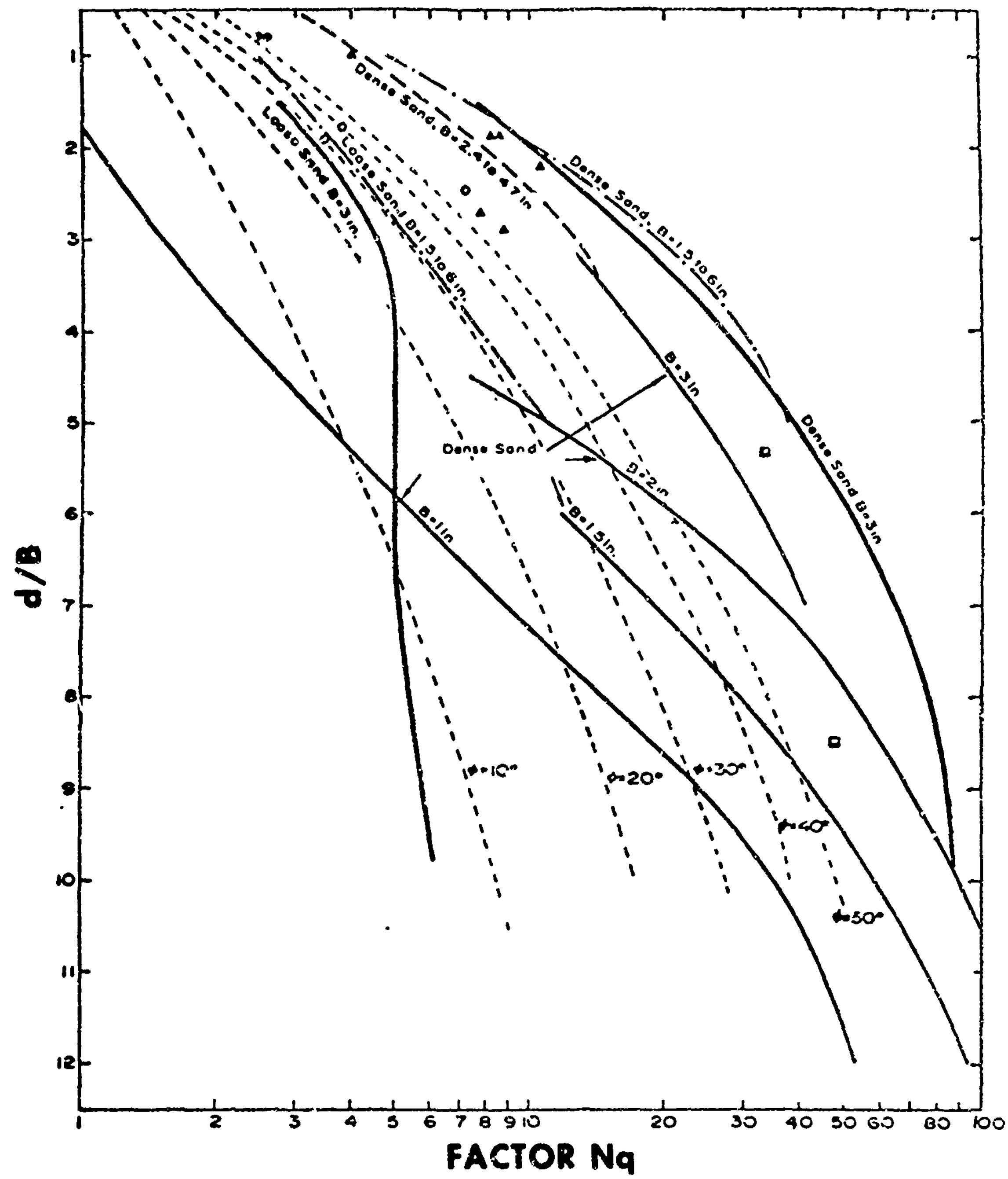

9 in. Cone Diameter
Site #6
Site #7
Site #13
Site #25
Depth to Cone (ft)
Pullout Resistance (lb)
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000


12 in. Cone Diameter
Site #2
Site #3
Site #15
Site #17
Site #8
Site #10
Pullout Resistance (lb)
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
Depth to Cone (ft)
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0


15 in. Cone Diameter
Site #18
Pullout Capacity (lb)
0
400
800
1200
1600
2000
2400
Depth to Cone (ft)
0
1.0
3.0
5.0
6.0
7.0
8.0
9.0

APPENDIX C

PULL-OUT FACTORS CHART



Anchor Breakthrough Factors Nq

Unclassified
Security Classification

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13. ABSTRACT

Twenty-three lightweight anchors consisting of a 10-foot X 2½-inch pipe with a metal cone welded at the tip end were emplaced and tested in twenty-five feet of water at Lameshur Bay, St. John, U. S. Virgin Islands. The anchors were jetted into the coral sand bottom by forcing water through the pipe and out an aperture in the cone. Two divers guided the anchors into the sediment using the jet of water to excavate the soil beneath the cone. Four of the anchors were emplaced with a cement slurry to increase the holding power.

It was found that the emplacement procedures were straightforward and posed no problems to the divers. However, the injection of the cement slurry was very time-consuming due to numerous problems.

The pullout results and theoretical analysis showed the jetted anchors to be capable of developing 2,000 to 10,000 lbs holding capacities in the soil at the test site. The holding capacity may be increased by increasing any of the following: anchor cone diameter (area), emplacement depth, compaction of the overburden sediment, or use of a cement slurry.

Tests indicate that the use of these light weight jetted-in anchors may be of practical use where bottom tie-downs and light anchorages are required. Further testing is recommended.

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14 KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Anchors (structures)						
Foundations						
Jetting						
Ocean bottom						
Placing						
Marine sediments						
Slurries						
Loads (forces)						
Performance tests						

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