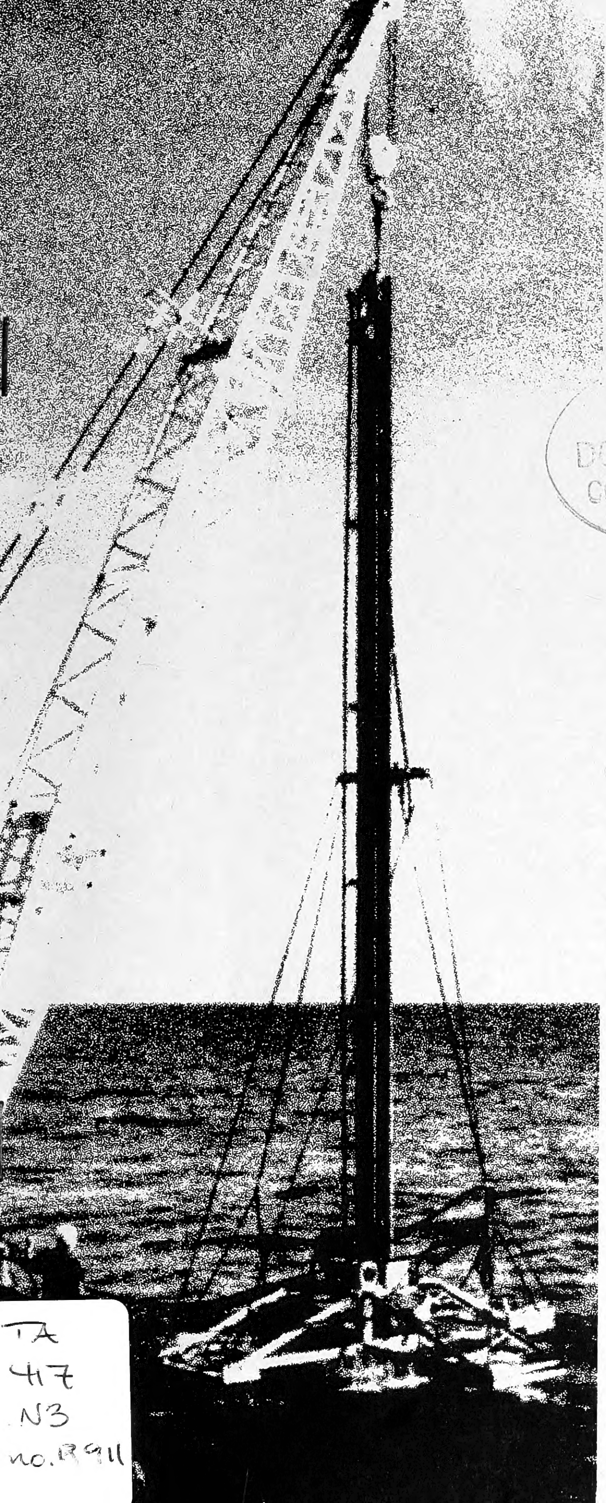


Technical Report R-911

XSP CONE PENETROMETER: A PERFORMANCE EVALUATION

By

R. M. Beard
B. A. Johnson

October 1984

Sponsored by

NAVAL FACILITIES ENGINEERING COMMAND
Alexandria, Virginia 22332

NAVAL CIVIL ENGINEERING LABORATORY
Port Hueneme, California 93043

METRIC CONVERSION FACTORS

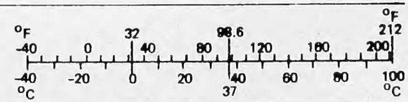
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2,000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1,000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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20. ABSTRACT *(Continue on reverse side if necessary and identify by block number)*

A lightweight seafloor soil test device has been developed that is capable of performing in-situ cone penetration tests (CPT) to 40-foot sediment depth in up to 200-foot water depths. This device, called the XSP (experimental static penetrometer), weighs 10,000 pounds and stands 50 feet tall. The water-jetting system, a unique feature, assists penetration to greater depths than is possible with the 10,000-pound reaction force available.

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20. Continued

The performance of the XSP has been evaluated in approximately 60 at-sea tests, and the penetrometer has proven reliable and produces repeatable data, which compare favorably with cores taken at the test sites. The water-jetting system aids penetration but does not affect the data. These XSP-supplied CPT data can be used to determine soil classification, relative density and friction angle in sand, and undrained shear strength in clay. They can also be used in the design of foundations (especially pile design) and in bearing capacity and settlement calculations.

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A lightweight seafloor soil test device has been developed that is capable of performing in-situ cone penetration tests (CPT) to 40-foot sediment depth in up to 200-foot water depths. This device, called the XSP (experimental static penetrometer), weighs 10,000 pounds and stands 50 feet tall. The water-jetting system, a unique feature, assists penetration to greater depths than is possible with the 10,000-pound reaction force available. The performance of the XSP has been evaluated in approximately 60 at-sea tests, and the penetrometer has proven reliable and produces repeatable data, which compare favorably with cores taken at the test sites. The water-jetting system aids penetration but does not affect the data. These XSP-supplied CPT data can be used to determine soil classification, relative density and friction angle in sand, and undrained shear strength in clay. They can also be used in the design of foundations (especially pile design) and in bearing capacity and settlement calculations.

CONTENTS

INTRODUCTION

This report describes the development of a seafloor soil test device capable of performing cone penetration tests to 40-foot sediment depths. The general objective of the project was to provide the Navy with a means of gathering geotechnical data on cohesionless soils to soil depths suitable for shallow piles and propellant anchors. The equipment developed was to be operable in water to 200 feet deep and suitable for operation from small anchored barges and other support vessels typical in Navy construction. The data provided from the electrical friction cone on the device was to be suitable for measuring soil strength and for soil classification.

Approximately 60 penetration tests were performed in various areas: in Norton Sound, Alaska; off Port Hueneme, Calif.; in San Francisco Bay, Calif.; and off Coronado, Calif. The data from some of these are presented in this report and data of this type can be directly applied to pile design using established methods. They are also suitable for selecting appropriate anchors and evaluating anchor performance. This development was funded by the Naval Facilities Engineering Command.

BACKGROUND

The Navy is responsible for constructing a variety of facilities in the nearshore and continental shelf regions (i.e., in water depths to 1,000 feet). These include small to moderate-sized pile-supported platforms and pile-supported piers and elevated causeways, as well as various types of moorings incorporating pile, direct-embedded, or conventional drag-embedded anchors. Knowledge of geotechnical properties of cohesionless materials, correlatable to end-bearing-capacity factors, skin-friction-capacity factors, horizontal subgrade reaction moduli, and other data is required for the design of these facilities. For the majority of Navy situations, it is adequate to limit the soil depth of geotechnical property determination to 40 feet. This choice of depth was based on the depth of shallow piles used in Navy operations such as for the elevated causeway system (ELCAS) and the depth of embedment of propellant-embedment anchors.

At the initiation of this work, systems capable of measuring the required parameters and of reaching subbottom penetrations to 40 feet in sands required use of either (1) a borehole, (2) diver-operated or remotely operated heavy equipment, or (3) coring and property estimation. These systems are undesirable for Navy use for several reasons. For instance, the major disadvantages of making a borehole are that the process is time-consuming and expensive and requires special drill ships or at-sea platforms to complete the work.

For the second method, equipment that can perform cone penetration tests to 40 feet from bottom-resting platforms, are available but are very heavy -- 40,000 pounds; Seacalf and Stingray are two examples. Seacalf can operate independently of a drilling platform but requires a special handling system and heave compensators. Stingray is used to control a drill string and, therefore, is much like testing out of a borehole. Smaller bottom-resting equipment are available such as MITS (Multipurpose In-situ Testing System) but are not capable of 40-foot penetrations.

Coring of cohesionless soils to 40-foot soil depths is possible with vibracorers weighing only a few tons. However, the highly disturbed samples are not suitable for measuring strength properties. Consequently, the properties must be guessed or estimated by rough rules of thumb which are less than desirable procedures and will not lead to confidence in a design.

To achieve a suitable way of evaluating cohesionless soils, the Naval Civil Engineering Laboratory (NCEL) first considered use of an instrumented vibracorer barrel to measure cohesionless sediment properties. A special instrumented barrel was fabricated that measured driving force, soil resistivity, and skin friction at several points along the barrel while taking a core. The idea was to use these in-situ measurements to control laboratory reconstitution of samples that would be extensively tested. Unfortunately, when the barrel was tested, the sensors were found to be easily damaged. Also, the data acquired were difficult to analyze (Lee, 1979). At this point, after a re-evaluation of candidate exploration techniques, it was decided that the use of established in-situ sensors was preferable even if a special bottom-resting platform needed to be developed to conduct the test (Lee, 1979).

Two possibilities existed: the standard penetration test (SPT) and the cone penetration test (CPT). The SPT is performed by dropping a 140-pound weight 30 inches (in air) onto a split-spoon sampler and counting the blows required to drive the spoon 1 foot into the bottom of a drilled hole after 6 inches of initial penetration. Because of the empirical nature of the SPT any change from this procedure negates the wealth of data that has been developed correlating blow count to soil properties. The SPT is designed to be performed in boreholes; therefore the objections that apply to drilling operations apply to the SPT.

The CPT, on the other hand, does not require a borehole. In this test, a cylindrical probe with a conical tip is pushed into the soil at a uniform rate of 0.02 m/sec or less. The probe is instrumented to measure the force on the tip of the cone and the friction on the side wall of the probe. The CPT provides detailed, continuous, and repeatable information on a site and is well-suited to solving many geotechnical design problems. Two disadvantages are that a large force is required to push the probe to desired depths, and no sample is obtained for inspection. However, the advantages outweighed the disadvantages, and the CPT was chosen as the testing device to be developed to satisfy the Navy's need for reliable data on cohesionless soils.

CPT PRACTICE

The cone penetration test was first introduced in Europe about 50 years ago, but only in the last decade has its use in the United States become popular. Initial application was in design of piles. Increased sophistication of the CPT, particularly the development of the electrical-friction cone, has led to greater use of CPT-obtained data.

A recent advancement has been the piezocone, which measures pore water pressure response in addition to the mechanical response of the soil. Pore pressures are generated during penetration in the soil pore water that are recorded as part of the cone pressure. Being able to record this pore pressure and use it to make corrections during the analysis of CPT data are especially important in soft soils and offshore sands.

Interpretation of piezocone data is an active area of research. The presentation of CPT practice in this report is limited to the interpretation of electrical-friction cone data and the application of the data to geotechnical designs. Areas of interest are briefly discussed in terms of data interpretation and geotechnical design. As will be noted, interpretation and use of CPT data are different for sandy and clayey soils. Details of data interpretation and use of the data for design are presented in the Appendix.

Data Interpretation

The data gathered during a CPT are the cone pressure and side friction. Values for these items are influenced by many variables, including soil type, density, fabric, and stress states, among others. No single unique theoretical relationship relates all the variables to the cone data, but theories have led to better understanding and interpretation of CPT data. However, empirical relationships are still the primary means of interpreting test results.

Soil Classification. Efforts to classify soils from CPT data were first reported by Begemann (1965). He found that the ratio of sleeve friction to cone pressure correlated well to median grain diameter. His findings have been confirmed and improved by other researchers (see Reference list). Today, soil classification charts are used widely to identify soil types, and these charts are being expanded to describe and classify problem soils, such as carbonates. These charts are dependent on cone type; a chart recommended by Martin and Douglas (1981) for electrical-friction cone is given in the Appendix.

Relative Density. Relative density of sands is estimated from cone pressure data. However, caution is warranted because the cone pressure data are affected by other factors such as overburden pressure. Recent research has shown that a much better interpretation of relative density can be made if at least one triaxial shear test is performed to define the relationship of relative density to friction angle for a particular soil. A graph of relative density as a function of cone pressure and overburden pressure (Schmertmann, 1978) is presented in the Appendix.

Friction Angle. Friction angle is proportional to relative density for a given sand. The Appendix presents two charts (Figures 27 and 28) for estimating friction angle. The one by Schmertmann (1978) uses relative density as an intermediate parameter to estimate friction angle as a function of soil type and relative density. The other (Mitchell et al., 1978) represents the practice in the Union of Soviet Socialist Republics and presents friction angle as a function of cone pressure and overburden pressure.

Undrained Shear Strength of Clay. The undrained strength of clay is back-calculated from the cone pressure by applying the bearing capacity equation. The difficulty lies in choosing an appropriate bearing capacity factor. The Appendix discusses this problem and provides guidance for selecting a bearing capacity factor.

Other Properties. The remolded strength, sensitivity, and overconsolidation of clays can also be estimated, but with less reliable results than undrained shear strength discussed in the preceding paragraph. Compression moduli for both clays and sand can be estimated. The determination of these properties is discussed in the Appendix, but the full development of the procedures is beyond the scope of this report.

Geotechnical Design

Many different procedures have been developed for making geotechnical designs from CPT data. Each has its advantages, disadvantages, and a particular application where it is most suitable. Schmertmann (1978) prepared an extensive set of design guidelines for CPT data, and his work is often referenced. The procedures given in the Appendix have been extracted for the most part from Schmertmann's work.

Pile Design. Schmertmann recommends the "Dutch" procedure for estimating pile end bearing and the procedure of Nottingham (1975) for estimating side friction. These procedures are explained in the Appendix along with recommendations on how factors of safety should be applied to the results. The reader is also referred to Schmertmann (1978a) for methods used to analyze tapered piles, different shaped piles, and the effects of insertion methods.

Bearing Capacity. Bearing capacity in sand requires estimating bearing capacity factors from the CPT data and applying them to the Terzaghi bearing capacity equation. In clays, the cone pressure is used directly to estimate bearing capacity. The procedures and recommended factors of safety are given in the Appendix.

Settlement. Settlement calculations for footings on sand, with CPT data as the basis, are quite adequate. For footings over clays, the results are more uncertain. The procedure for making settlement estimates are not given in the Appendix because of their complexity and thus, exceed the scope of this report.

Pile Drivability. Drivability of piles has been correlated to cone pressure. One correlation was developed by DeRuiter and Beringen (1979) and is given in Appendix.

APPROACH

After the decision to develop a platform for performing penetration tests was set, the operational requirements were set at 40 feet of penetration into uncemented sands, silts, and soft-to-medium clays at a maximum water depth of 200 feet. Forty feet of penetration in these soils was chosen because it was sufficient for designing the common shallow piles used in Navy operations (e.g., ELCAS) and would satisfy most of the requirements in propellant-embedded anchor work. The 200-foot water depth represented the design depth limit of many NCEL-developed shallow water systems, such as the Offshore Bulk Fuel System (OBFS), and therefore seemed to be a logical depth limit for this experimental device. Also, within this depth limit, difficulties in transmitting data and providing power to the device were minimized. Another requirement in the project was that the tool be operable from the type of Navy-owned vessel typically available. This vessel is usually a small barge with a deck-mounted crane. Also, this constraint in effect limited the weight of the tool to about 10,000 pounds.

The first step in developing this tool was to generate a conceptual design conforming to these operational limits. Woodward-Clyde Consultants (1980) was contracted to do the design. Their first thought was simply to extend the capabilities of an already developed cone penetrometer. However, it was apparent that the reaction required to push a cone 40 feet into sand would be about 30,000 pounds. This posed two problems. First, to provide the reaction by self-weight would violate a design provision; weight was limited to 10,000 pounds. Second, the rod used to push the cone would be susceptible to buckling under such loads. As alternatives to pure mechanical insertion of the cone, vibration and water-jet-assisted mechanical insertion were studied. Vibration was eliminated because of concern that the 40-foot penetration would not be obtained and that the cone's sensors would be damaged. The water-jet-assisted penetration appeared promising because there was experience in water jet penetrations to the necessary depth.

This concept was further developed by analyzing the hydraulics of jetting and the cone rod design. The configuration believed to be most viable was a vibracorer-type frame with a remotely controlled chain-driven, water-jet-assisted cone penetrometer (Figure 1).

The conceptual design of the device was moved to final design and fabrication by Fugro-Gulf, Inc. (1981). This device, called the XSP (for experimental static penetrometer), is the subject of this report. It is described in detail, operational procedures are presented, and the results of its evaluation are given. Procedures for interpreting CPT data and using it with geotechnical designs are given in the Appendix.

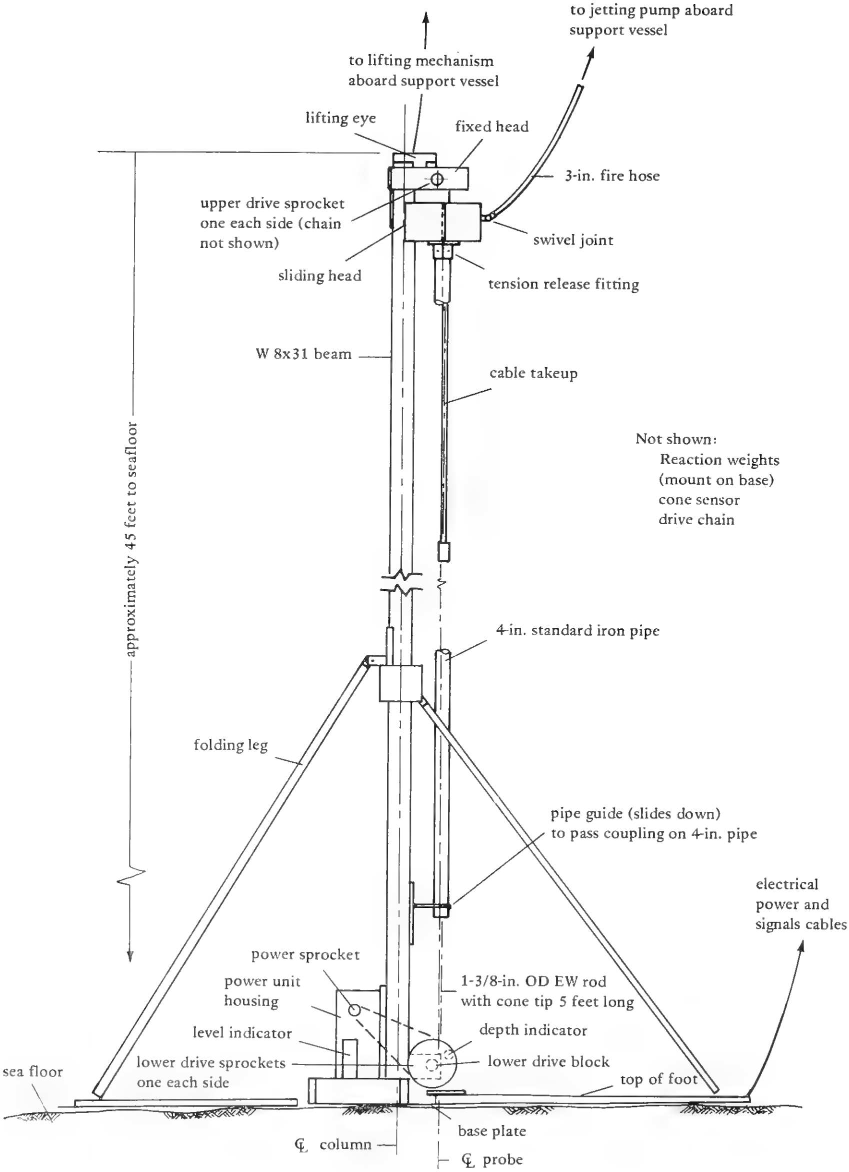


Figure 1. Conceptual design of a cone penetrometer with a 40-foot penetration capability.

EQUIPMENT DESCRIPTION

The XSP (Figure 2) is a static cone penetrometer consisting of two major components: a 50-foot tall, 10,000-pound structure containing the cone penetrometer and an instrumentation console. In operation, the structure is set on the seafloor to perform a cone penetrometer sounding, while the instrumentation console located on the ship's deck is used to control and monitor the sounding and record the data. The data are later analyzed to determine soil characteristics and design parameters.

The structure is composed of a structural frame, a cone penetrometer, a drive mechanism for the cone penetrometer, and a water-jetting system for assisting penetration. The water jet is a feature not found on any other cone penetrometer. The structure's components and the instrumentation console are described in the following sections.

Structural Frame

The steel structural frame (Figure 2) supports the cone penetrometer and provides a place to mount the motor and driving mechanism. This frame has three main structural parts: a square, table-like base; four support legs attached to the base with steel pins; and a tall, central H-beam bolted to the base. The base is approximately 4 ft^2 and 3 feet tall. Mounted on the top of the base are the electrical junction box to which all the electrical cables connect, the motor, and the gear box. On the bottom of the base is a space to add 1 ton of steel plate ballast. The legs are sturdy frames 4 feet wide and 8 feet long, pinned in place on the base. One of these legs is easily collapsed so it will fold up while pinned in place, allowing the structure to be either laid down on a ship's deck or hung close against the side of a ship. Since all four legs are similarly pinned to the base, this collapsible leg can be placed on any side of the base, depending on how the XSP will be placed on and deployed from the support vessel. All of the legs can also be unpinned at the top of the base and stretched out along the beam to make the structure more compact for shipping. The 20-foot span across the legs provide a stable base for the upright structure. The total bearing area provided by the bottom of the base and the pads on each leg is sufficient to support the structure on a very soft clay (about 0.5-psi shear strength). A central H-beam supports the push rods and parts of the rod's driving mechanism. The height of the H-beam can be changed to allow the XSP to operate in either a 40-foot or 20-foot soil penetration mode, in which the XSP stands 50 and 30 feet tall, respectively.

Cone Penetrometer

The cone penetrometer (Figure 2) consists of two components: the push rods and the cone unit. The latter contains the electrical cone penetrometer tip. The push rods are 10-foot sections of Acker 2-1/4-inch OD AW flush-joint drill casing. Four push rods are used in the 40-foot mode and two in the 20-foot mode. The waterproof electrical cord to the cone unit runs inside the rods.

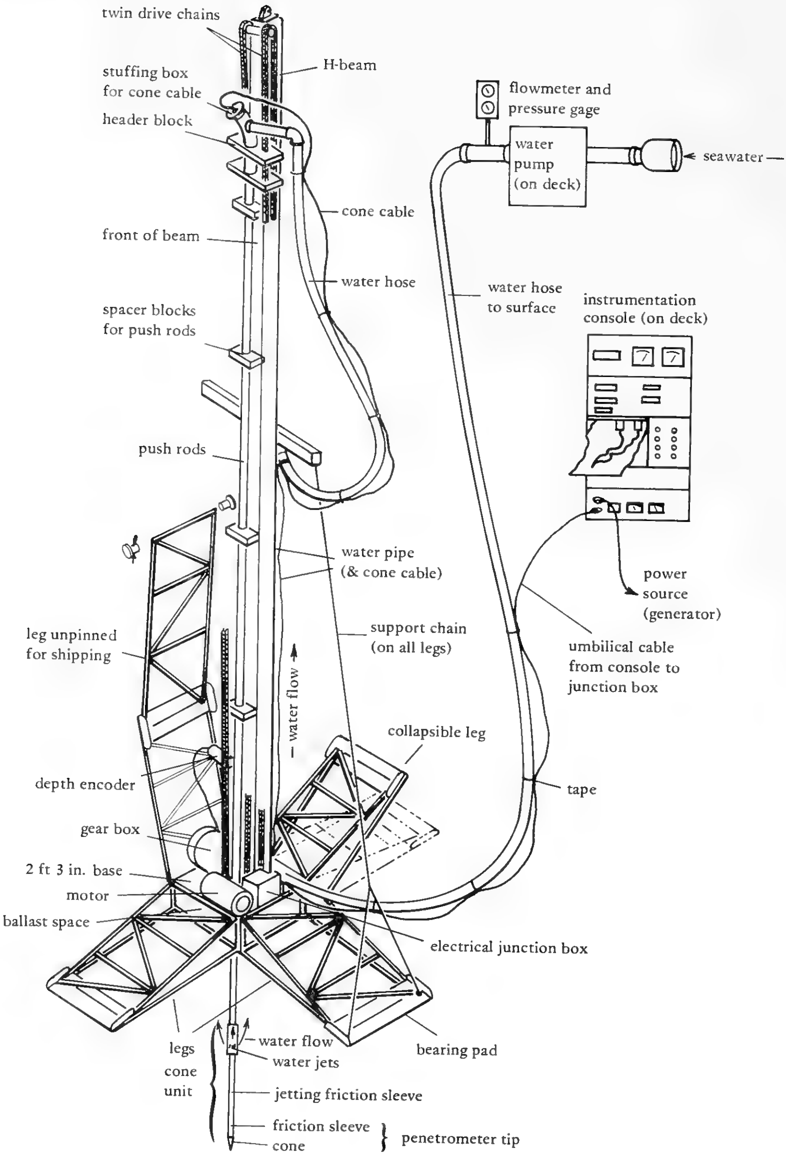


Figure 2. The XSP system.

Attached to the lower end of the push rods is the cone unit (Figure 3). The cone unit contains the jetting nozzle, upper sounding rod, jetting friction sleeve, lower sounding rod, and the electric penetrometer tip. This tip consists of a cone and a friction sleeve. The XSP penetrometer tip is a 5-ton Fugro Wison Cone.

The jetting nozzle will be discussed in more detail in the section entitled Water Jetting System. Basically, water is pumped through the push rods and out the jetting nozzle. The water coming out the jetting nozzle is aimed back up the outside of the push rods to fluidize the soil and ease penetration of the push rods.

The upper sounding rod is a watertight structural connection between the jetting nozzle and jetting friction sleeve and it creates a separation between the waterjets and jetting friction sleeve. This sleeve is a strain-gaged section used to measure differences in the friction caused by the jetting water near the jets and the friction at the friction sleeve on the electric penetrometer tip.

The lower sounding rod is both a structural connection and separator between the jetting friction sleeve and the electric penetrometer tip. In the penetrometer tip, strain gages are used to measure pressure on the cone and friction on the sleeve 3 inches above the cone. The signals from the cone and the friction sleeve are amplified and transmitted up the cone cable and ultimately to the surface instrumentation console.

Instrumentation Console

The instrumentation console (Figure 4) contains the controls for the XSP, monitors the sounding process, and records the data. This console, about 4 feet tall and 2 feet square, is housed in a fiberglass case. A power cable connects the console to a 208-volt, 3-phase AC, 30-ampere power source. The console is, in turn, connected to the junction box on the XSP's base by an underwater umbilical cable. From the junction box, cables lead to the motor, depth encoder, and the cone unit. The console also has readouts for two electronic pendulums located inside the junction box. The pendulums detect structure tilt in two vertical planes rotated 90 degrees from each other. Signals from these devices on the XSP are amplified and scaled for output to the panel meters and recorder. The major control on the console is an UP-STOP-DOWN switch for the driving mechanism and the cone penetrometer. There are digital readouts for the cone pressure, cone pressure and friction sleeve, friction sleeve, jetting friction sleeve, and depth of soil penetration. Electronically, the cone and friction are measured together and then the cone is subtracted to display the friction. Other gages provide readouts for the manually controlled 24-hour clock, tilt gages, a voltmeter, and an ammeter. The cone pressure, cone friction sleeve, and jetting friction sleeve are recorded on a Watanabe strip chart recorder as a function of depth.

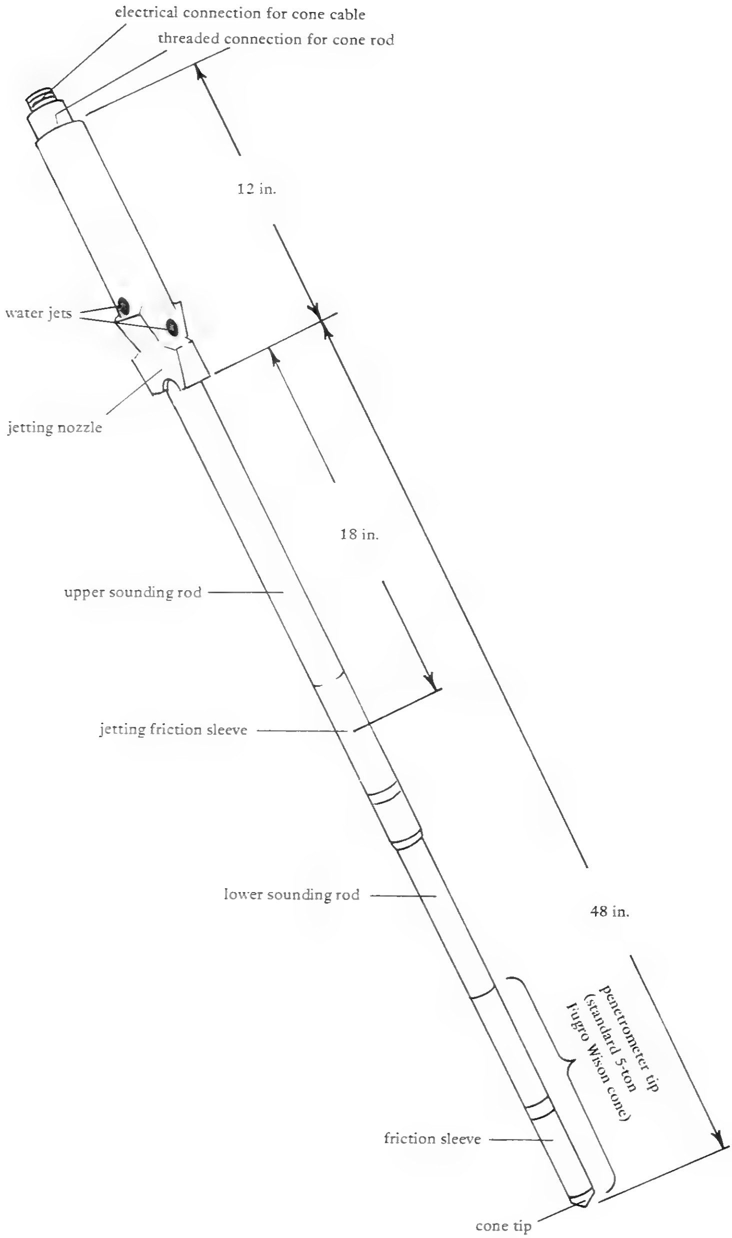


Figure 3. Cone unit of XSP.

Figure 4. Instrumentation console for XSP system.

Drive Mechanism

The driving mechanism pushes the cone penetrometer into the seafloor. This mechanism is composed of a 1-hp motor and gearbox on the base, twin drive chains along the sides of the beam, and a guide track and bracing system for the push rods along the front of the beam (Figure 2). The drive chains are connected to a header block at the top of the push rods. The block slides along a track mounted on the front of the beam. The push rods are supported and guided along the track by spacer blocks. When the cone control switch is flipped to DOWN, the motor drives the chains which steadily push the header block and, consequently, the cone penetrometer downward. The depth of penetration is measured by a depth encoder on the side of the beam which is gear-driven by one of the drive chains.

Water-Jetting System

The water-jetting system on the XSP is a unique feature. The jetting system consists of the jetting nozzle on the upper end of the cone unit, water hoses, and an on-deck water pump. The components are shown in Figure 2. Seawater is pumped through hoses to the header block, where it is directed down through the push rods and out and upward from the jetting nozzle. The purpose of the jetting is to fluidize the soil adjacent to the push rods to reduce soil friction and ease penetration. If too much water and pressure are used, the jetting can adversely affect the penetration data; this condition will be detected by a difference in the two friction sleeve readings. Experience has shown that 50 gpm at 50 psi does not influence the cone readings in a variety of mixed soils and sands. Jetting is not needed to achieve maximum penetration in most clays.

EQUIPMENT OPERATION

To operate the XSP, the equipment must be assembled and the operating mechanisms checked out. After checkout the equipment is laid on the support vessel and transported to the location for the first sounding. The vessel is anchored or otherwise held stationary as the XSP structure is deployed and the cone penetrometer pushed into the soil while the console records the data. The process of inserting the cone penetrometer into the soil is often referred to as a sounding. The deployment, operating, and data acquired by the XSP are described in the following sections.

Deployment

The XSP can be deployed in either of two ways: it can be stood up and deployed from the ship's deck with a crane, which depends on deck space and the type of lifting equipment available, or it can be deployed from a hanging position along the side of the vessel. To lay the XSP in the 40-foot mode down on the deck for transit to the test site, a triangular space 50 feet long with a 20-foot base must be available, along with a crane capable of uprighting and lifting the 10,000-pound structure (Figure 5). The XSP in the 20-foot mode would need a 30-foot triangular space with a 20-foot base, and the structure weighs 8,000 pounds. Once the structure is uprighted, it can be lowered to the seafloor either by the crane (Figure 6) or switched to a lowering line off an A-frame.

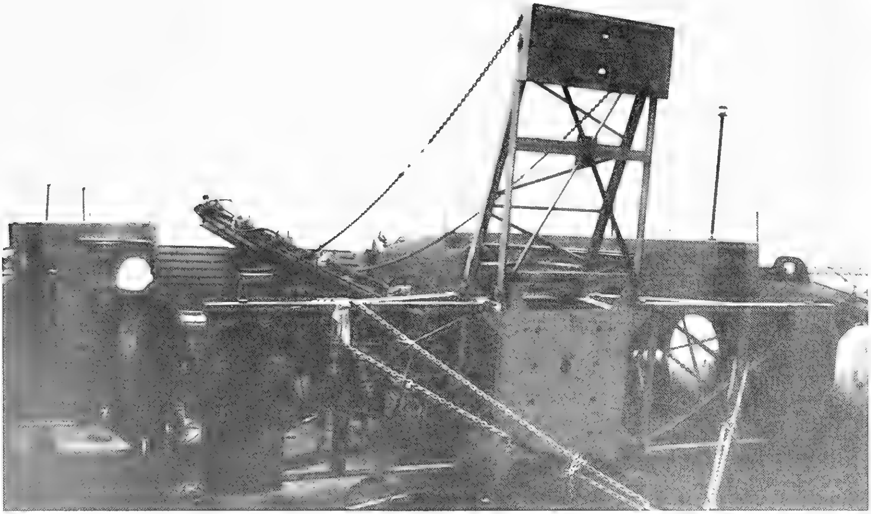


Figure 5. XSP in 40-foot mode lying on deck of an LCU.

If the XSP cannot be deployed in this manner, then the structure can be hung horizontally from a pair of davits along the side of the support vessel (Figure 7). For deployment, the base is lowered from its davit until the structure is upright; then it is transferred to the lowering line and placed on the seafloor.

Operation

At the sounding location, the support vessel must be anchored or otherwise held stationary during the sounding operation. Once the XSP structure is deployed and sitting on the seafloor, the water pump is started up to initiate the jetting system if it is to be used for the sounding, and the instrumentation is zeroed at the console.

If any slope to the seafloor exists, the initial position of the tilt-gage needles should be noted. The cone penetrometer is started down for a sounding, and its progress is monitored by observation of the strip chart recordings and the ammeter and tilt gages. A movement of either tilt gage indicates that the cone has met refusal. The cone penetrometer must be stopped immediately and retracted. Because the cone is no longer penetrating the soil, continued operation of the driving mechanism will cause the structure to crawl up the push rods. Eventually, the frame will tilt, and the cone unit and extended push rods will be broken off and lost.

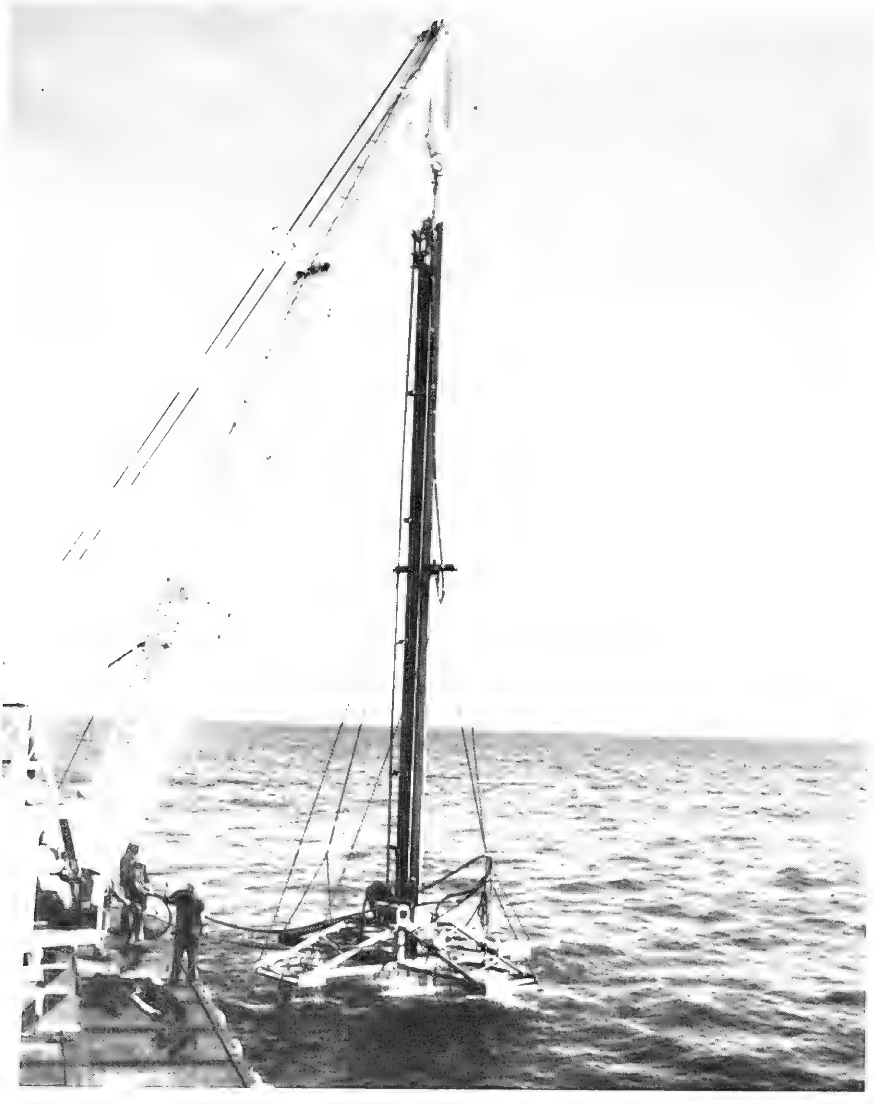


Figure 6. XSP in the 40-foot mode being deployed by crane from deck of NCEL Ocean Research Craft (ORC).

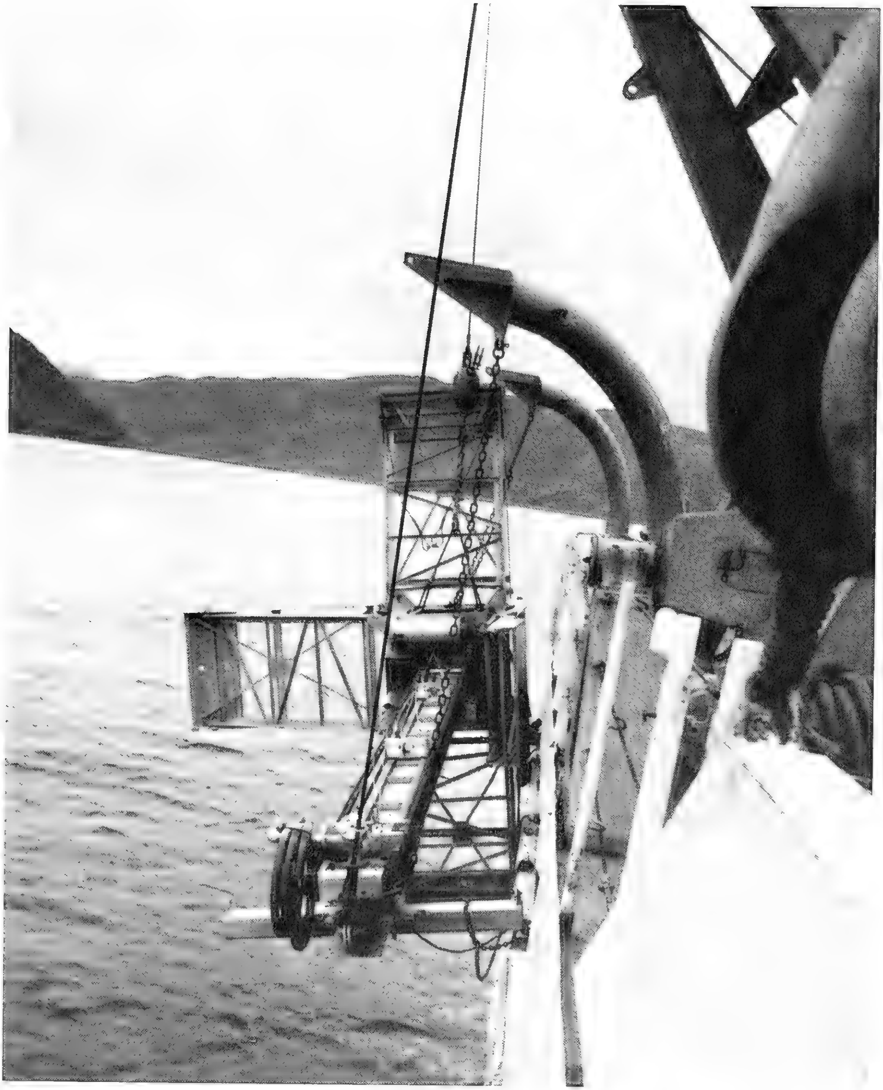


Figure 7. XSP in the 20-foot mode being deployed from davits on the side of the support vessel.

When the sounding has been completed or refusal met, the cone is retracted. The structure can then be either moved over slightly to repeat the sounding for comparison or brought back on board before moving to a new sounding location.

Data Acquired

The data from the cone pressure, friction sleeve, and jetting friction sleeve for one sounding as recorded by the strip chart recorder are shown in Figure 8. Measurements are all recorded in kilograms per square centimeter. The magnitude of the chart scales can be varied for different soils by changing the millivolt settings on the strip chart recorder. The depth scale is dependent upon the chart speed, which can be set by chart controls or controlled by the depth encoder.

TEST PROGRAM

The testing of the XSP had two objectives. The primary one was to evaluate the jetting system in terms of its effectiveness in allowing deeper seafloor penetration and its effect on the data. A secondary objective was to perform a general evaluation by testing the penetrometer in different soil types. To meet these objectives field tests have been conducted with the XSP in Norton Sound, Alaska; near Port Hueneme, Calif.; in San Francisco Bay, Calif.; and near Coronado, Calif. The XSP test sites are shown in Figure 9.

The Norton Sound tests were conducted in conjunction with the U.S. Geological Survey (USGS). USGS's purpose in conducting these tests was to provide quantitative geotechnical information on the behavior of a variety of marine sediments that may be involved in processes (such as gas charging, wave-induced liquefaction, and ice gouging), potentially hazardous to offshore development. For the Navy, these tests provided an opportunity to evaluate the jetting system at a site with dense sands. The Port Hueneme tests were conducted to evaluate the jetting system in mixed soils (over 20% each of sand, silt, and clay) and to provide data to a project evaluating propellant-embedded anchor holding capacity. The XSP data can be used to calculate the undrained shear strength of the soil which is a parameter in the equation for propellant-embedded anchor holding capacity. The San Francisco Bay tests were conducted to evaluate the XSP in a "mud" seafloor and to provide additional data on the anchor holding capacity project. The Coronado tests were conducted to evaluate the water jet. The data were used by another project to determine the depth at which a layer of hard material, possibly cobbles, exists at this site.

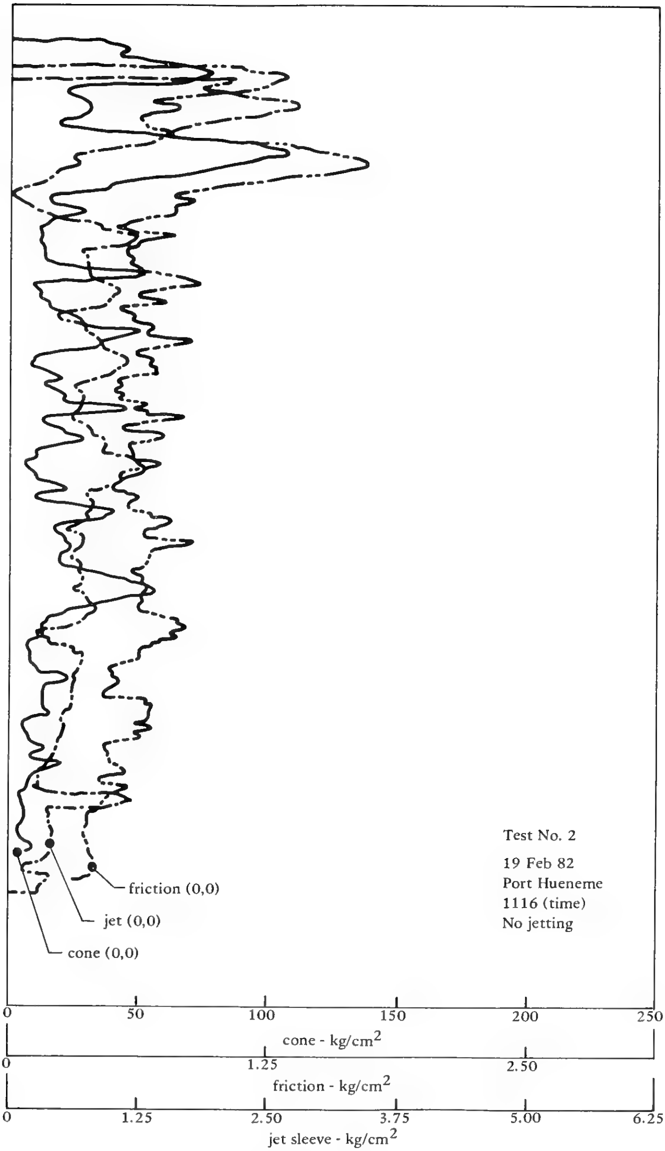


Figure 8. Strip chart recording of data from XSP instrumentation console.

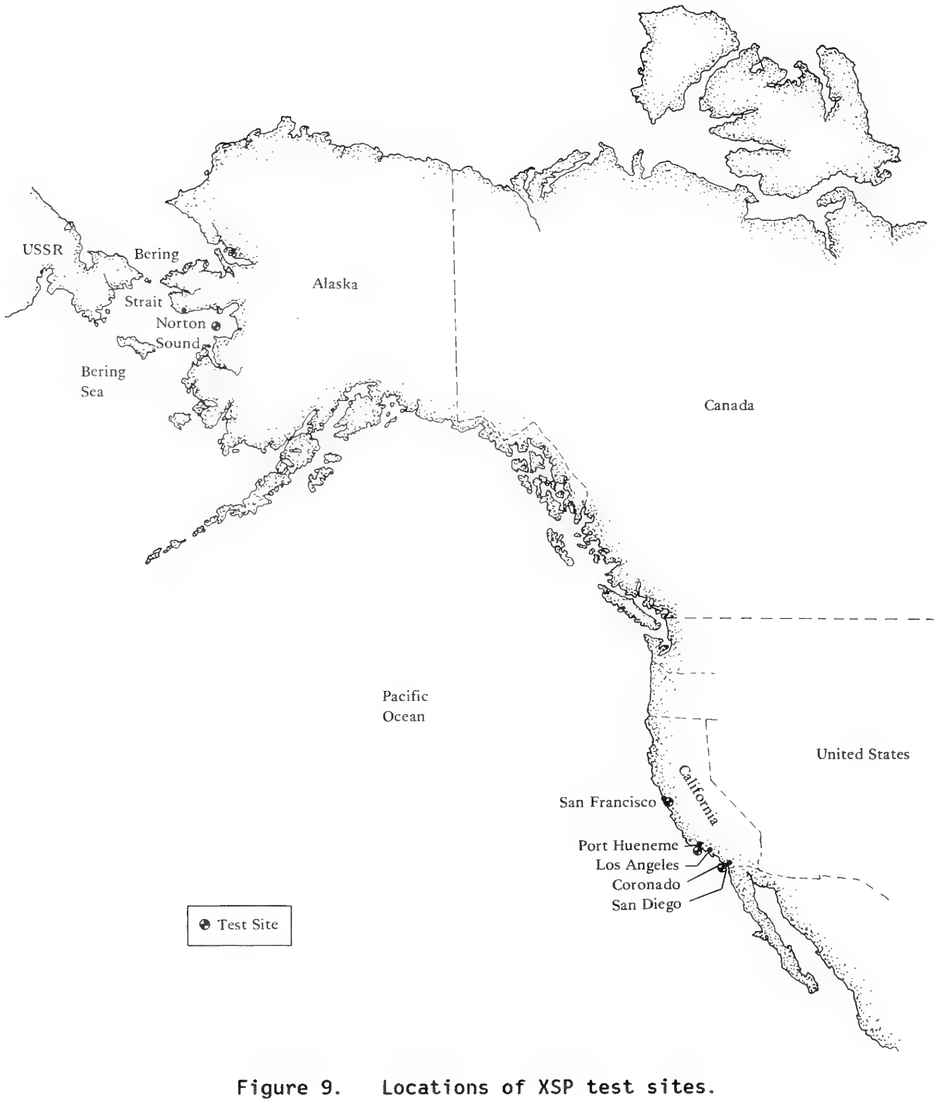


Figure 9. Locations of XSP test sites.

DATA REDUCTION

The data from the strip chart recordings (Figure 8) were digitized and used to calculate the friction ratio, which is the ratio between the sleeve friction and the cone pressure. This friction ratio is usually expressed as a percent. The cone pressure and friction ratio data were then plotted as a function of depth. Examples of these plots are given in Figures 10 and 11. Before forming the friction ratio, the sleeve friction and cone pressure readings were corrected to a common depth point by assuming that the measured cone pressure represents the behavior of sediment at the cone tip and that the friction stress represents behavior at the center of the friction sleeve which is 3 inches above the cone. Using these plots, soil profiles were developed over the depths of penetration using Figure 12 (a chart of soil type as a function of cone pressure and friction ratio). This chart was derived by Martin and Douglas (1981) for determining stratigraphy from data taken with electrical friction cover. Details on reducing the data are provided in the Appendix.

TEST RESULTS

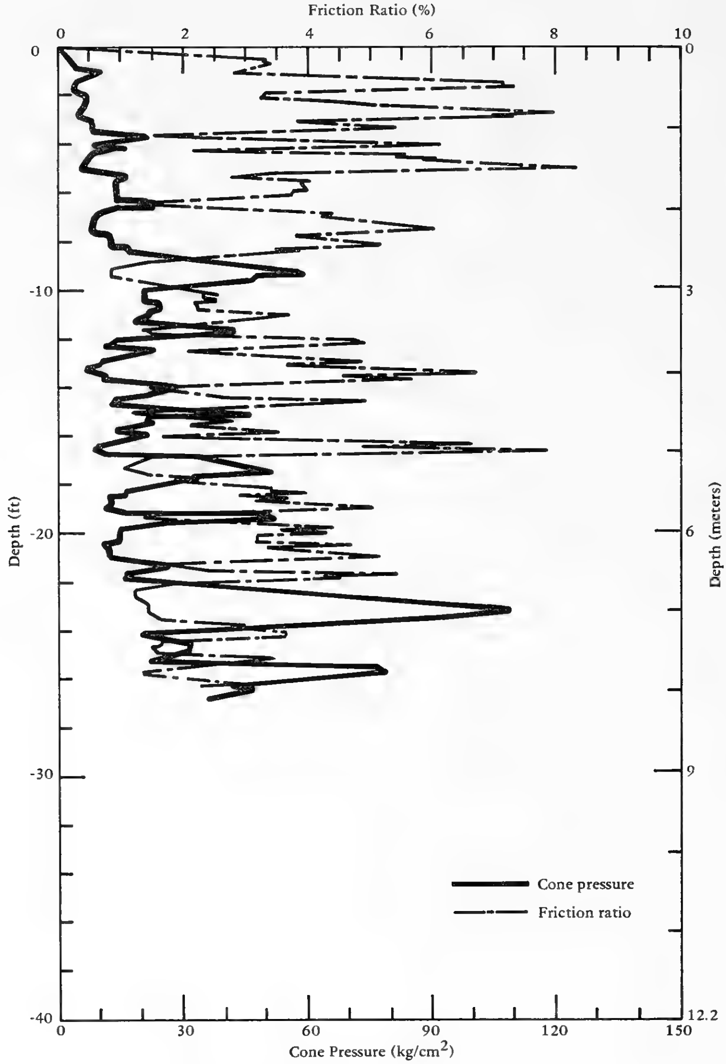
Norton Sound, Alaska

During the summer of 1981, a total of 40 soundings were made with the XSP in the 20-foot mode at the Norton Sound test site off the west coast of Alaska. National Oceanic and Atmospheric Administration's (NOAA) ship R/V DISCOVERER was used as the support vessel. In deployment from this ship, the XSP was hung horizontally over the side of the ship from a pair of davits (Figure 7).

All of the soundings were made without jetting because the cone unit with the jetting nozzle was broken off and lost on the first sounding. The backup cone (a penetrometer tip only) was fitted to the push rods and used to perform the remaining soundings. With this penetrometer tip, water jetting was not possible because the cone unit was not complete because it did not contain a jetting nozzle (see Figure 3). No problems were encountered while performing the remaining 39 soundings.

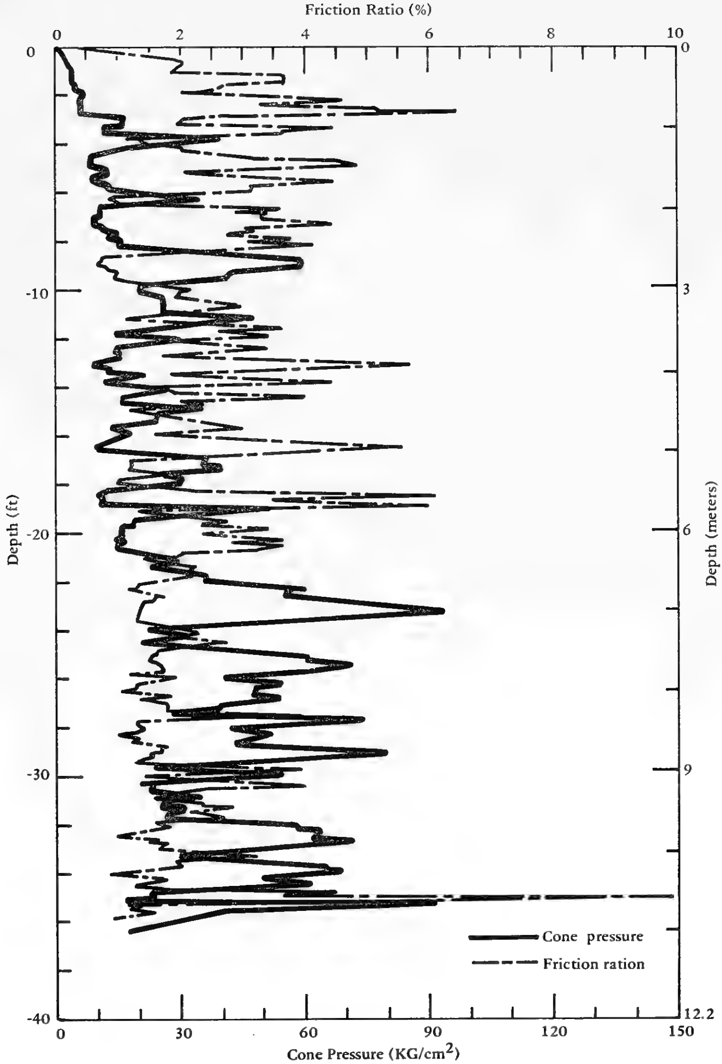
The soils encountered at the test site were very dense, limiting penetration to between 1.6 and 12 feet. Multiple soundings were made at most sounding locations, and the data were consistent between replicate soundings. Most of the sounding locations were also subsequently sampled with a vibratory corer and these cores compared well with the XSP data.

Within the test site, five areas were selected for the XSP soundings. These areas, shown in Figure 13a, were selected to provide coverage of the Norton Sound area and to cover in some detail areas that may be involved in processes potentially hazardous to offshore development. These potential geologic hazards -- gas charging, wave-induced liquifaction, and ice gouging -- were first detected by high resolution seismic profiling, side seam sonar, and geochemical and geological evaluation of soil cores. Reliable in-situ data were needed to quantitatively evaluate the hazard potential; thus the XSP was used.



Sounding 2: cone pressure and friction ratio (no jetting)

Figure 10. Plot of cone pressure and friction ratio versus depth for XSP sounding 2 done without using the jetting system offshore Port Hueneme.



Sounding 4: cone pressure and friction ratio (jetting)

Figure 11. Plot of cone pressure and friction ratio versus depth for XSP sounding done with the jetting system at the same location as the test shown in Figure 10.

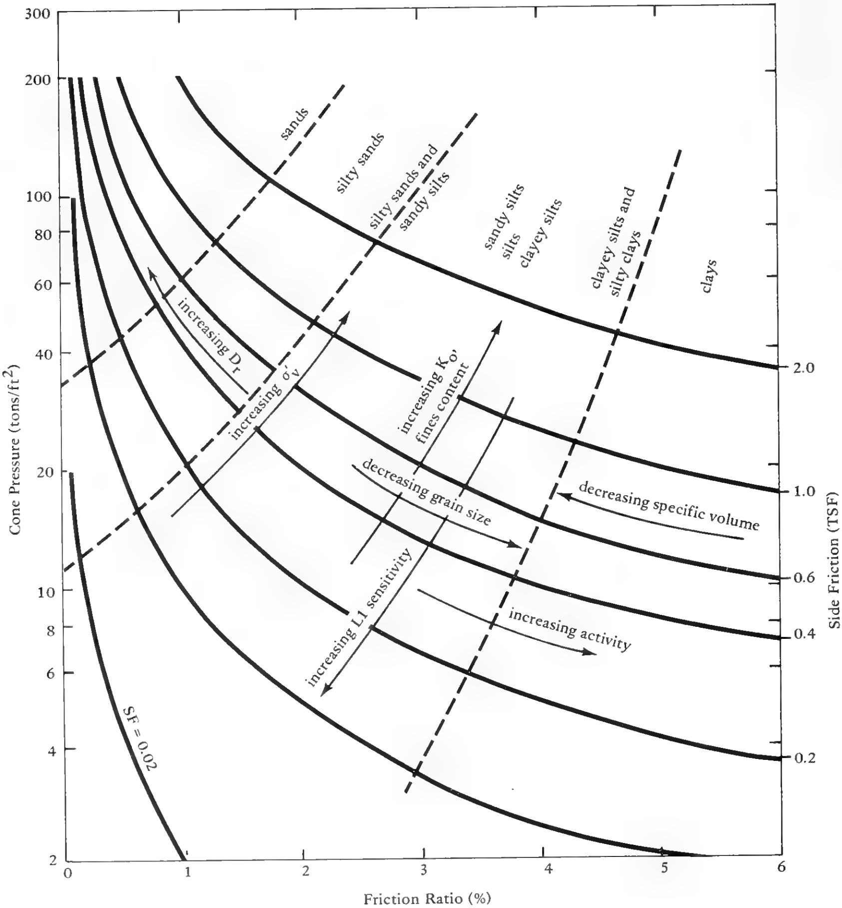
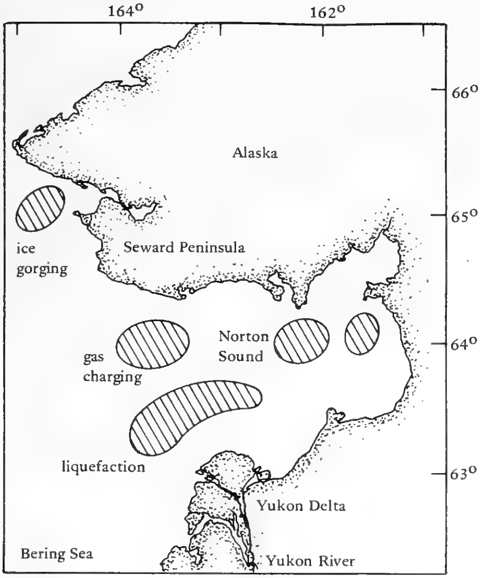
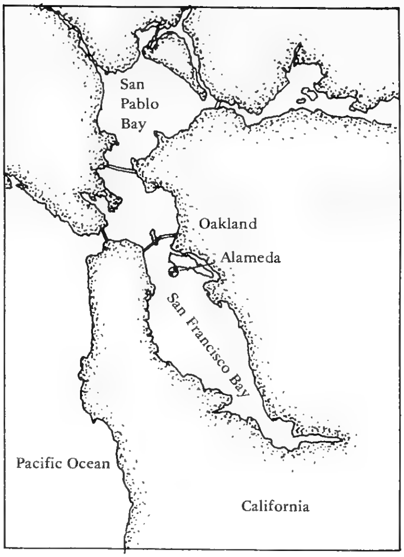


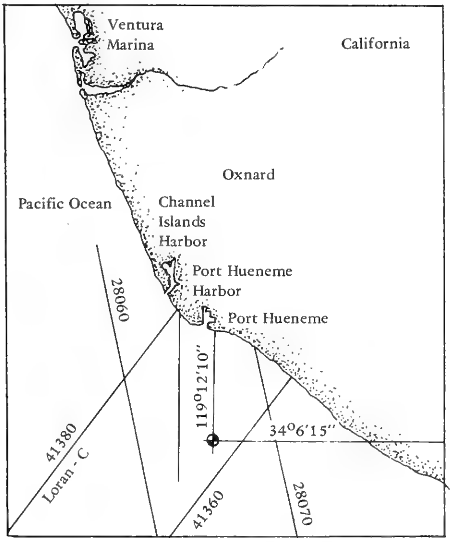
Figure 12. Classification chart for CPT data (Martin and Douglas, 1981).



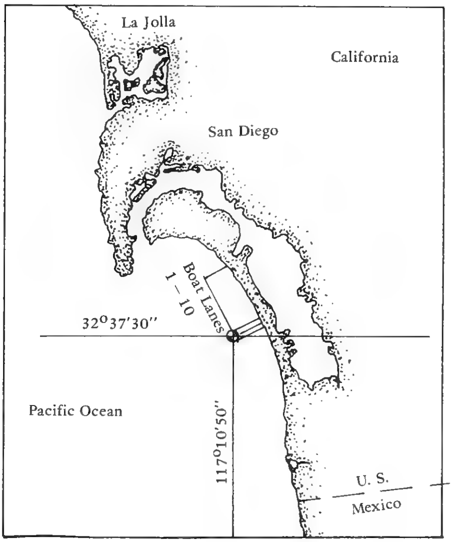
a. Norton Sound showing five sounding areas.



c. San Francisco Bay.



b. Port Hueneme.



d. Coronado.

Figure 13. XSP test sites.

Four of the 40 soundings (see Figure 14) will be discussed in more detail. The sounding plots are shown in Figure 15. The classification of the sediment from these four soundings is shown in Figure 16. These classifications agree reasonably well with classifications made on vibracore samples. The first sounding (Figure 15a) was within an acoustically identified gas-charged sediment. Low strength sediments due to gas charging may be more vulnerable to scour and storm-wave-induced shearing stresses. The second (Figure 15b) was 0.6 miles west of the first and just out of the gas-charged sediment area. The drops in cone pressure seen in Figure 15a corresponded with gas-charged zones in the sediment identified from vibracores. The slight difference between the peak envelopes of cone pressure in Figures 15a and 15b may be a result of somewhat lower effective stresses in the lighter gas-charged material. The soundings shown in Figures 15c and 15d are from the Yukon prodelta area. The Yukon prodelta contains sediment that is in the fine sand to silt range which is often associated with liquefaction due to cyclic loading on shore. Storm waves propagating northward from the Bering Sea generate large cyclic bottom shearing stresses in Norton Sound. This could result in liquefaction and movement of large sheets of sediment within this area. The first sounding (Figure 15c) is from a more protected area to the northwest and the second (Figure 15d) from an area on the west side that is more exposed to intense storm activity. It is apparent that the sediment in the protected area is not as dense as the sediment in the exposed area (Figure 15d).

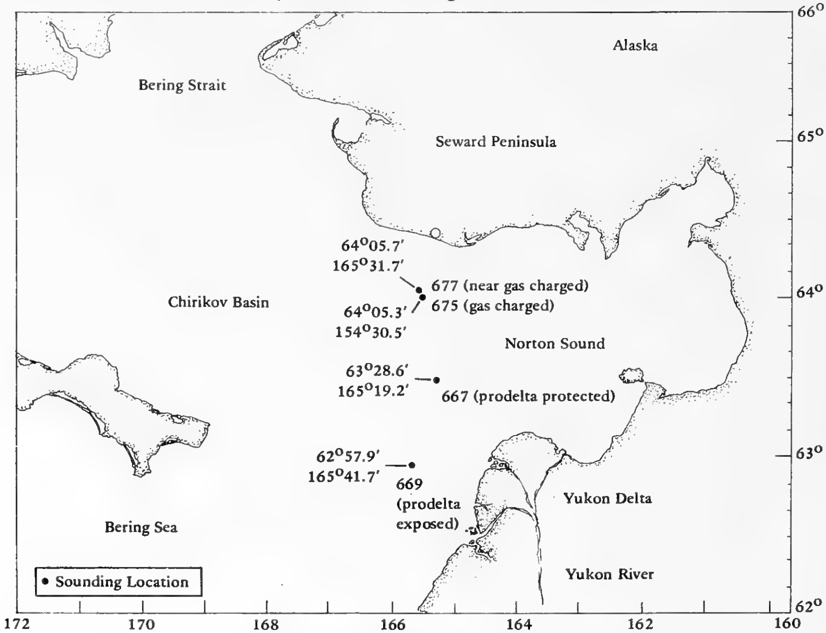
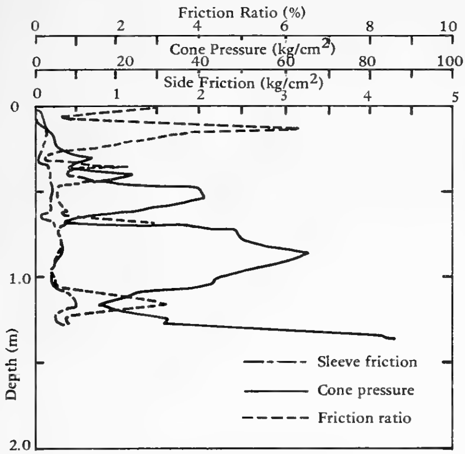
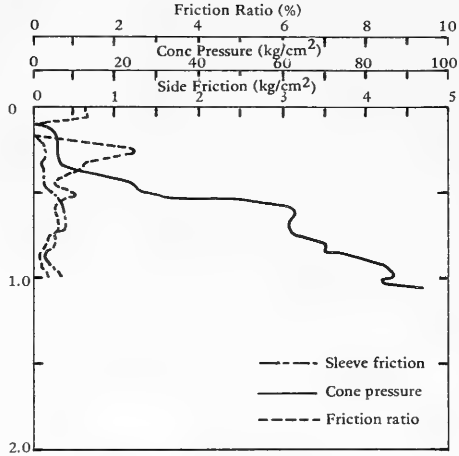


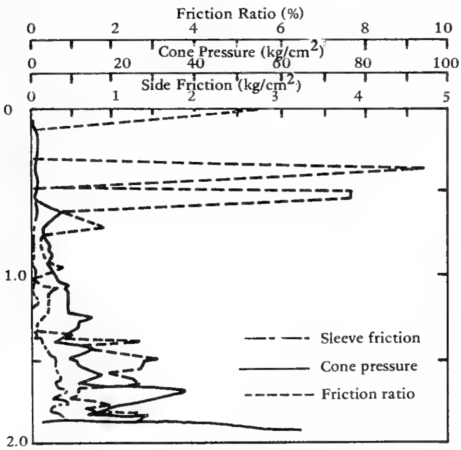
Figure 14. Four of Norton Sound's sounding locations.



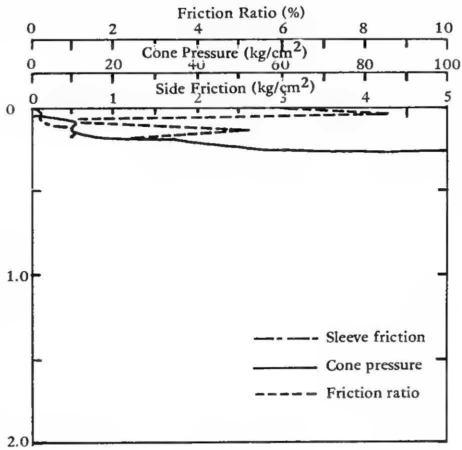
a. Sounding records for site 675 (gas charged sediment).



b. Sounding records for site 677 (near gas charged area).



c. Sounding records for site 667 (Yukon prodelta, protected).



d. Sounding records for site 669 (Yukon prodelta, exposed).

Figure 15. Example XSP sounding records from Norton Sound for the four soundings.

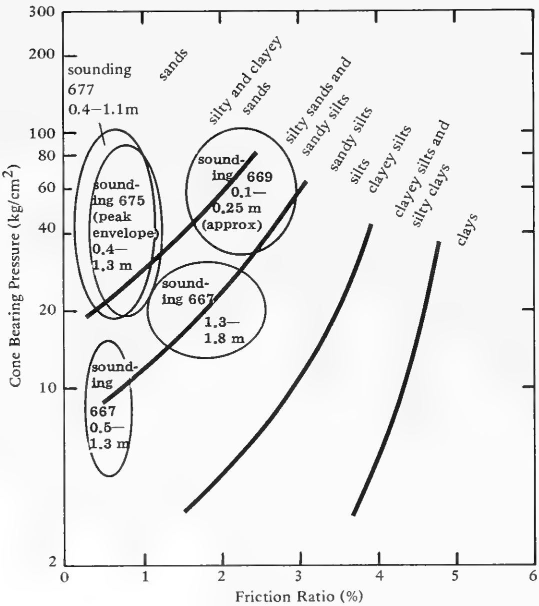


Figure 16. Correlation of friction ratio, cone pressure, and sediment type for the four soundings at Norton Sound.

Port Hueneme, California

The XSP was tested at a site offshore Port Hueneme in February 1982. The purpose of the testing was to evaluate the XSP and its water-jetting system in a mixed seafloor and to provide data for an anchor holding capacity project.

Six soundings were conducted with the XSP in the 40-foot mode at the site offshore from Port Hueneme (Figure 13b). A detailed map of the sounding locations given in LORAN-C coordinates is shown in Figure 17. The NCEL Ocean Research Craft (ORC) (warping tug) was used as the support vessel. The XSP was deployed using a crane and lowered to the seafloor from the A-frame with a line.

One minor problem with the XSP occurred in conducting this test. The jetting sleeve malfunctioned at the beginning of the testing. Side-by-side soundings (with and without jetting) were done for comparison of the effect of jetting on the cone data. Soundings 1, 3, and 4 were conducted with jetting at a water flow rate of about 50 gpm and a pressure of 50 psi; soundings 2, 5, and 6 were done with no jetting. Examples of reduced sounding data from this location are shown in Figures 10 and 11. Figure 10 is sounding 2 where the water jet was not used and Figure 11 is sounding 4 where the water jet was used. These two soundings were separated by about 100 feet. It should be noted that 10 more feet of penetration was achieved when the water jet was used. The data were consistent between soundings, proving the XSP is capable of providing reproducible data. Stratigraphy developed from each of the six soundings is shown in Figure 18.

Two cores were taken with an Alpine Vibracore at the LORAN-C sounding locations of both soundings 1 and 2. The stratigraphy developed from the core data is shown in Figure 19 as is the stratigraphy developed from soundings 1 and 2. The stratigraphy from sounding 1 compares favorably with the stratigraphy from cores 1 and 2 taken nearby. The same good comparison is found between cores 3 and 4 and nearby sounding 2. This again shows that the XSP provides reliable data.

San Francisco Bay, California

The XSP was tested at a site in San Francisco Bay (Figure 13a) on 16 June 1982. This testing was to evaluate the XSP in a silty-clayey (usually called a "mud") seafloor and to provide supporting soil data for an anchor holding capacity project.

Three soundings were conducted with the XSP in the 40-foot mode using an LCU as the support vessel. The LCU-1466 was provided by the 481st Transportation Company (Heavy Boat) of the U.S. Army Reserves. The XSP was handled with a crane loaded on the LCU and tied down to the deck. The XSP was laid down on deck with the top end hanging out over the bow (Figure 5). The locations of the soundings are shown in Figure 20. Exact coordinates for sounding locations of soundings 1 through 3 are not known because navigation equipment was not available.

The first sounding reached a subbottom depth of 29 feet, and the second sounding reached 38 feet. The third sounding was terminated at 20 feet because the increasing current was causing the ship to drift away from the structure which made it increasingly difficult to retrieve the structure. The data from these soundings showed the soil to be a very soft silty clay to clayey silt. The overburden pressure was subtracted* from the cone pressure in the analysis of these data. The data from the second sounding is shown in Figure 21.

No problems were encountered in using the XSP, except for the ship's inability to maintain station.

*If it is not subtracted, it changes the soil classification to an incorrect classification.

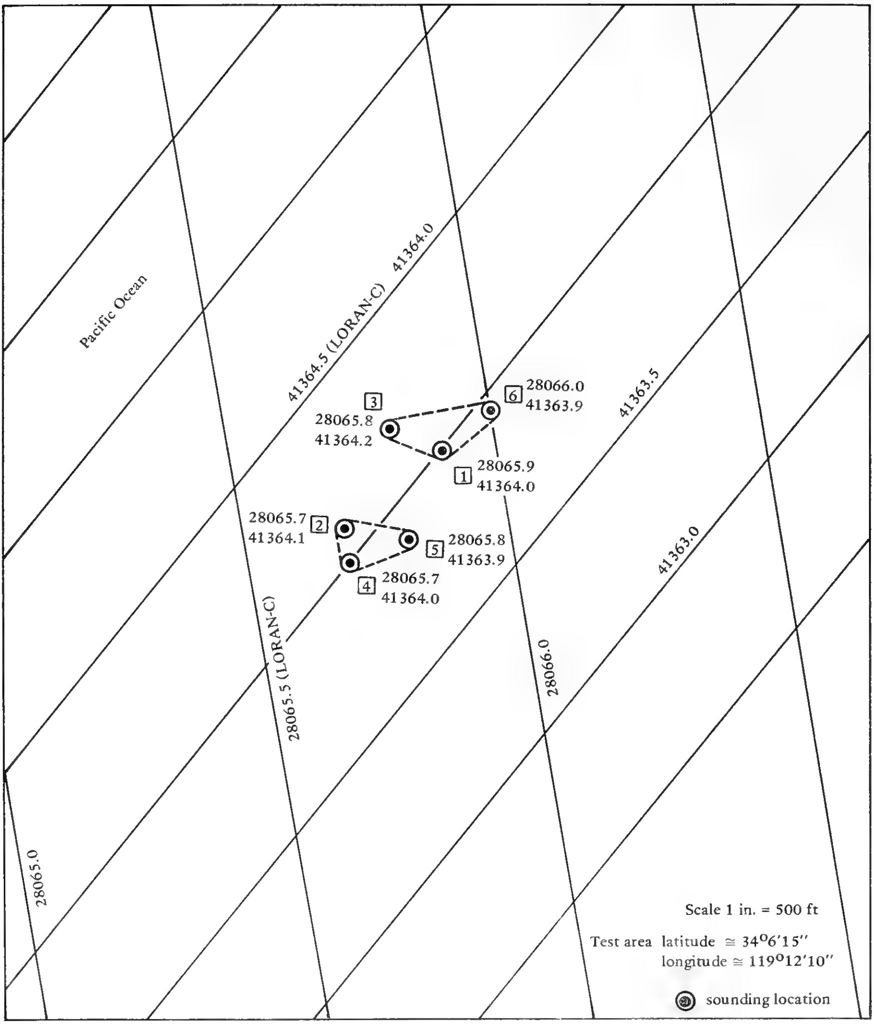


Figure 17. Port Hueneme test site for the XSP in the 40-foot mode on 19 February 1982, showing six sounding locations given in LORAN-C coordinates.

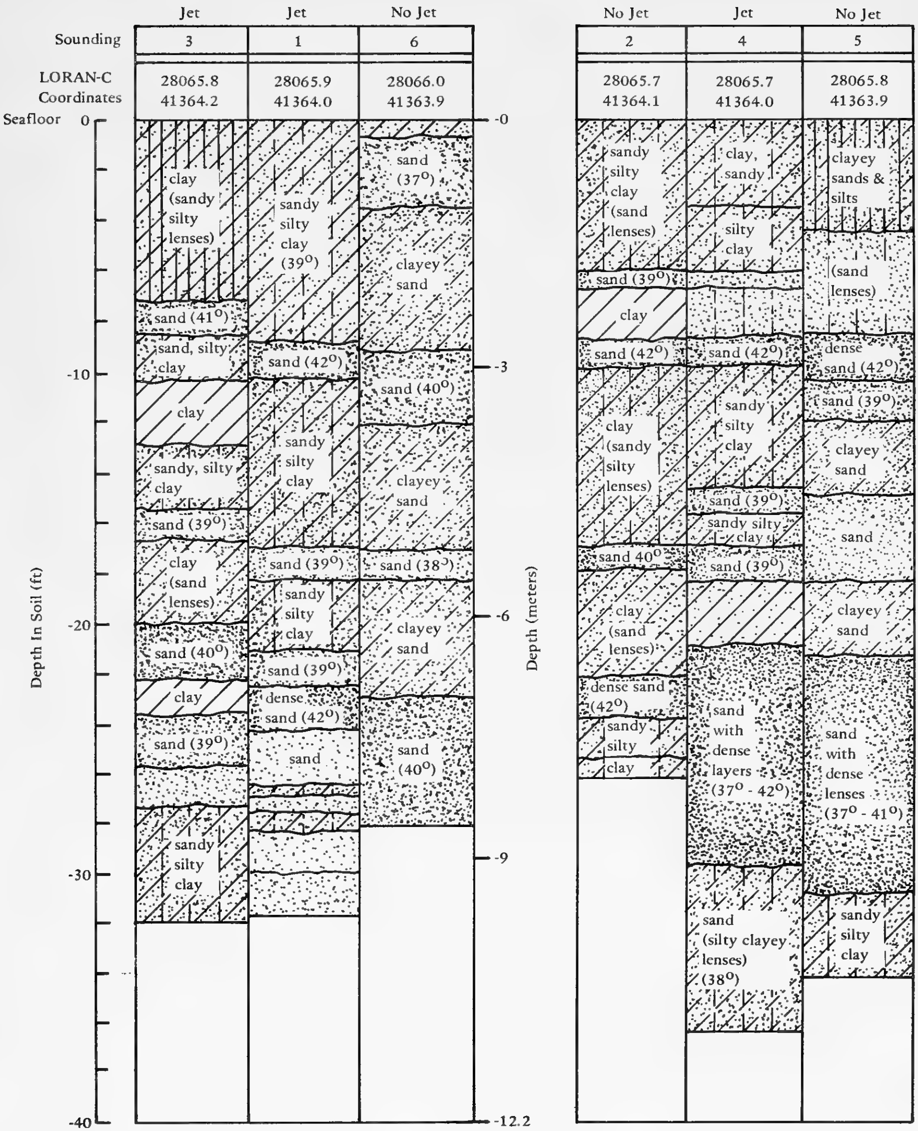


Figure 18. Stratigraphy test site developed from XSP data taken at Port Hueneme on 19 February 1982.

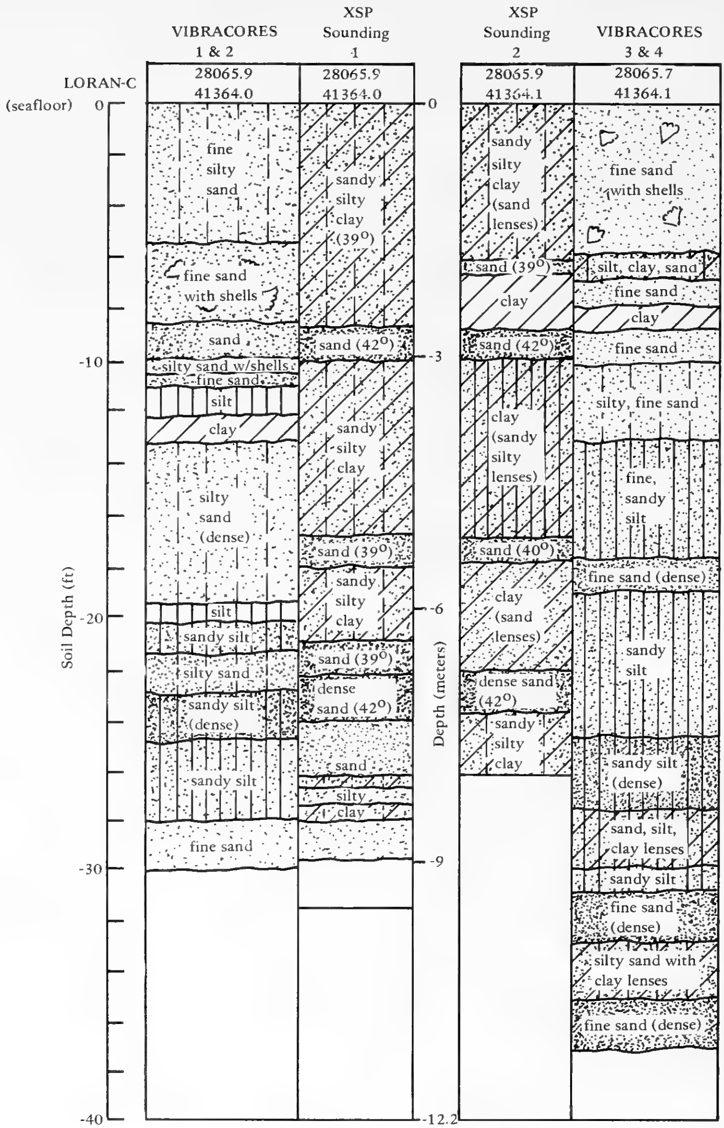


Figure 19. Vibracore stratigraphy from Port Hueneme test site compared to sounding stratigraphy from the same location.

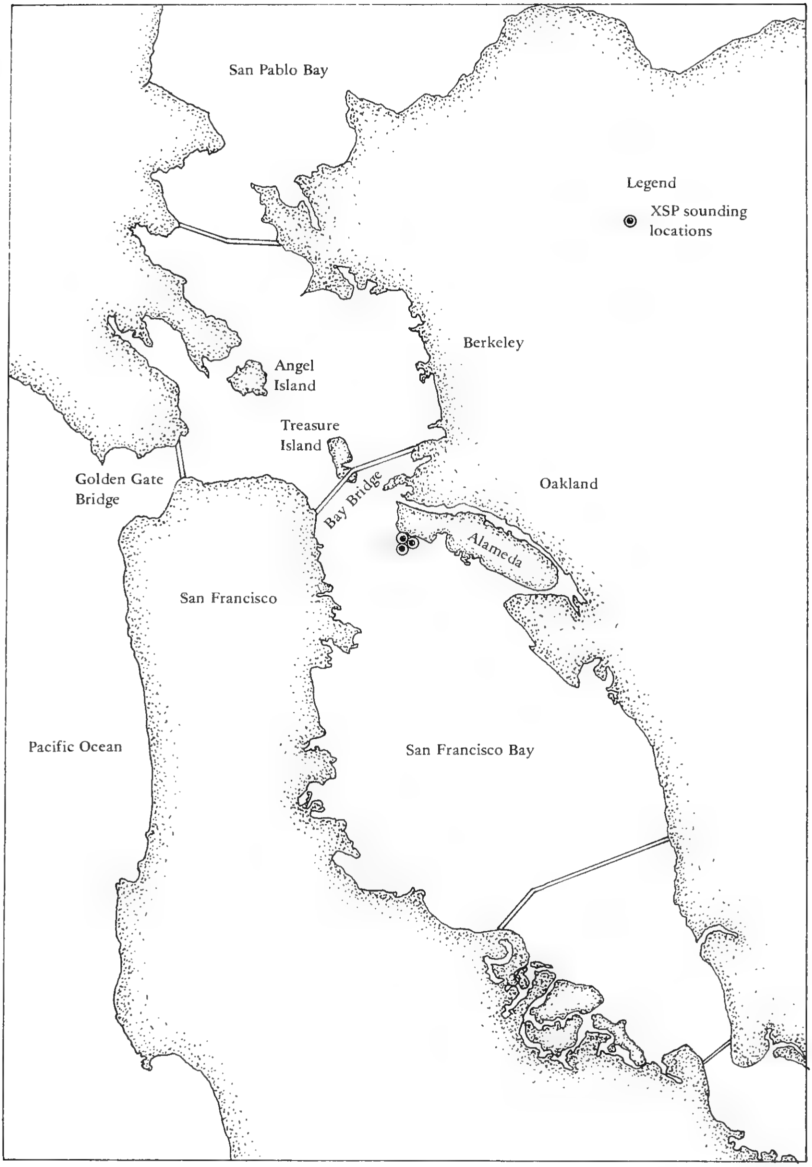


Figure 20. San Francisco Bay XSP test site, 40-foot mode, 16 June 1982.

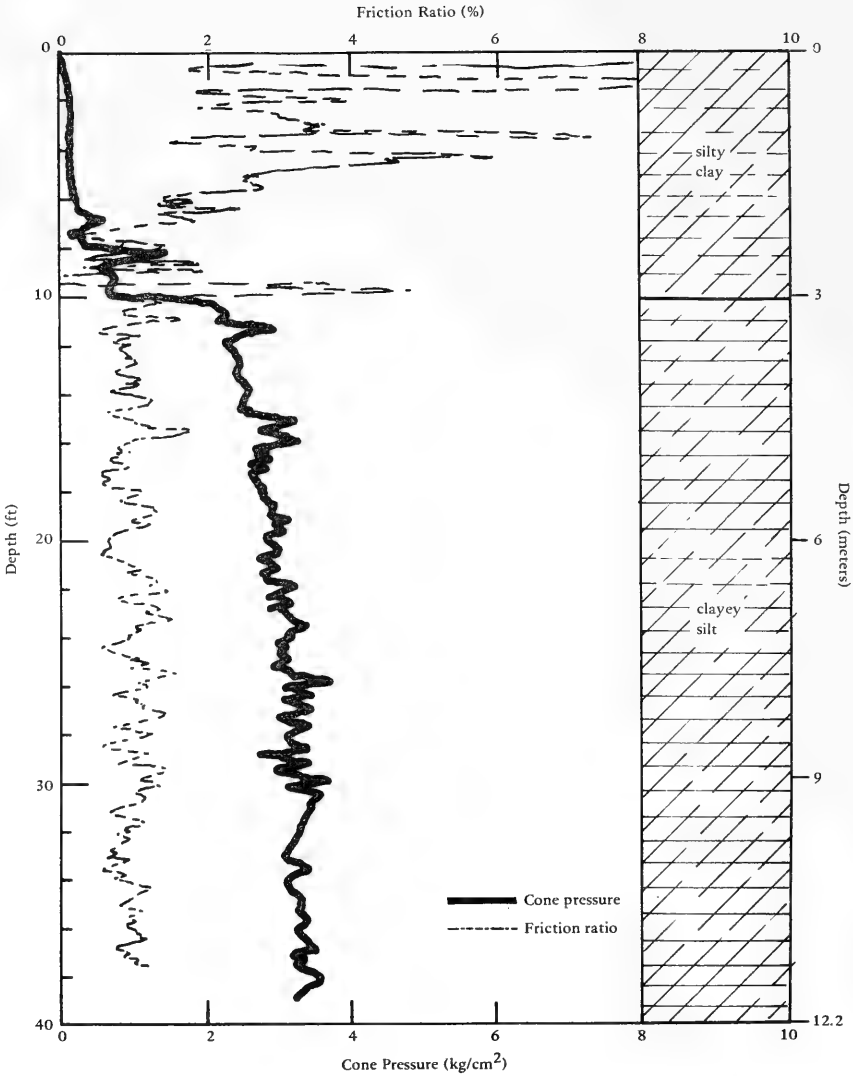


Figure 21. Cone pressure and friction ratio for XSP sounding 2 in San Francisco Bay.

Coronado, California

The XSP was tested at a site offshore from Coronado, Calif. (Figure 13d). A total of 15 soundings were made with the XSP in the 20-foot mode on 31 October 1982. The XSP was deployed from the deck of the NCEL Ocean Research Craft (ORC) (warping tug) with the crane (Figure 6). This site was used by the Offshore Bulk Fuel Supply (OBFS) to install a Single Point Mooring (SPM) buoy using four large drag-embedment anchors. The locations of these anchors were marked with bouys (Figure 22) and labeled North, South, East, and West. Three XSP soundings were conducted at each of the four anchor marker buoys; two of the three used jetting at a water flow rate of 50 gpm and a pressure of 50 psi. Three other soundings (numbers 13, 14, and 15) were taken in the area probing for the cobble layer detected with a jet probe in February 1981. The data were very consistent from sounding to sounding. An example is provided in Figure 23. The stratigraphy developed from the 15 soundings is shown in Figure 24. Those soundings conducted without jetting reached 7 to 8 feet in depth. For three of the sites, the soundings with jetting reached from 8-3/4 to 9-1/2 feet in depth. At the East Buoy, however, refusal was met on one sounding at 5-1/4 feet with jetting and at 7-1/2 feet for the remaining two soundings (one with and one without jetting). Refusal may have been met at 5-1/4 feet due to hitting a cobble layer or rock since, at the end of the testing, the cone tip was found to be flattened. The last three soundings were done to probe and map the area. One sounding was shoreward of the buoys where refusal was met at 6-1/2 feet, and the other two were seaward of the buoys where refusal was met at 9-1/2 feet. The results of these soundings are similar to those of jet probing done at the same sites (Figure 25).

No problems were encountered in conducting these soundings. However, it is apparent that very dense sands or cobble layers cannot be penetrated with the 10,000 pounds of thrust which can be developed by the XSP.

DISCUSSION

In general, the XSP has shown itself to be a reliable piece of equipment for gathering in-situ soil data. A total of 64 soundings were performed, and the only major problem encountered was on the very first sounding when the cone unit was broken off and lost. The XSP can be handled easily if the support vessel has the proper amount of space and lifting equipment. The easiest way to deploy the XSP is with a deck crane (Figure 6). However, it was demonstrated during the tests at Norton Sound that the XSP can be deployed from a horizontal position when held by davits over the side of the ship (Figure 7). Successful deployment and recovery of the XSP requires a stationary support vessel. Deployment has been made in sea state 2, and it is anticipated that sea state 3 is a limiting condition (depending on the support vessel and handling procedures).

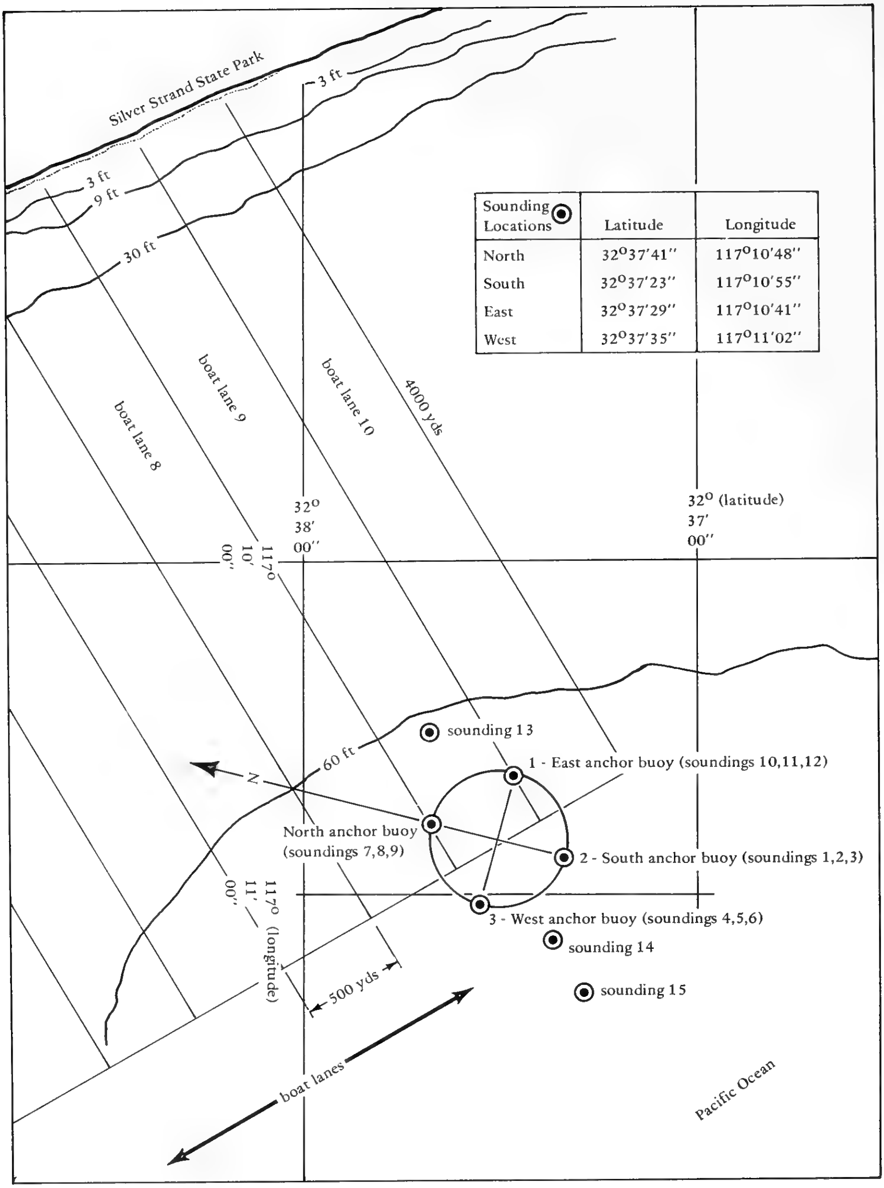


Figure 22. Coronado XSP test site, 20-foot mode, 31 October 1982.

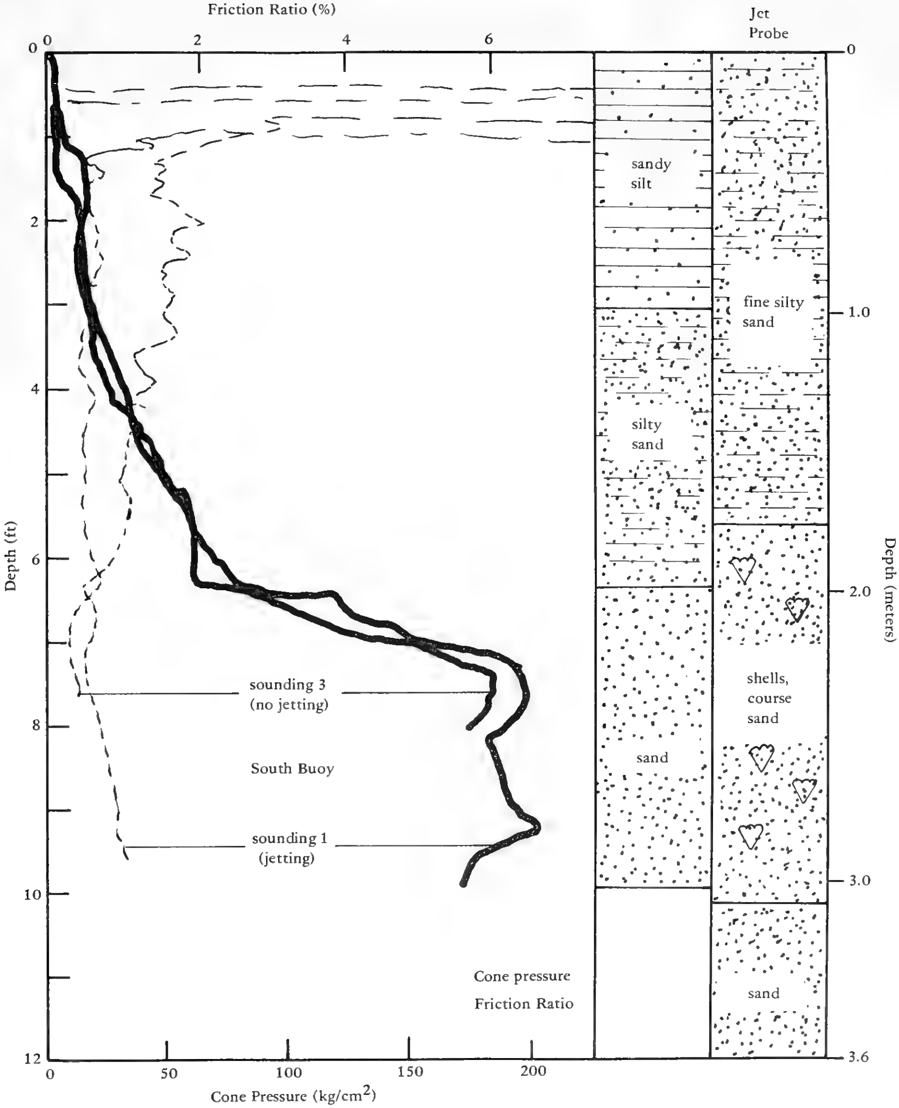


Figure 23. Cone pressure and friction ratio for Coronado XSP sounding 1 and 3 and resultant stratigraphy compared to jet probe data from same location.

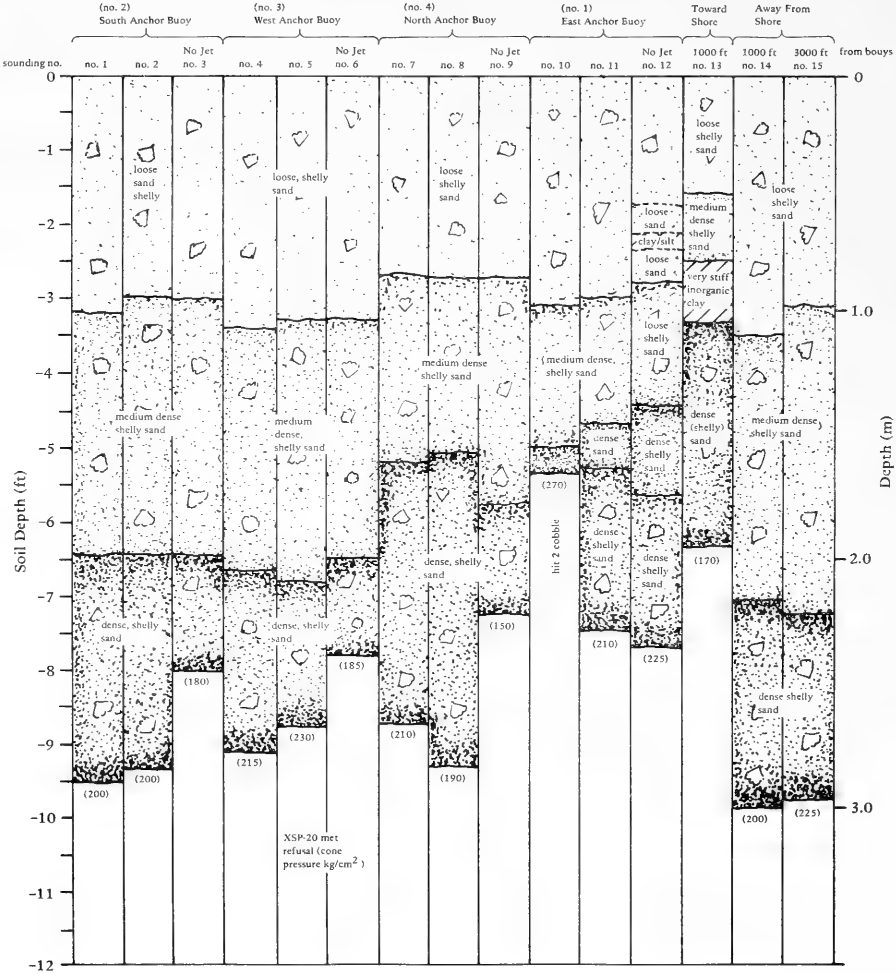


Figure 24. Stratigraphy developed from XSP data taken on 31 October 1982 at Coronado for soundings 1 through 15.

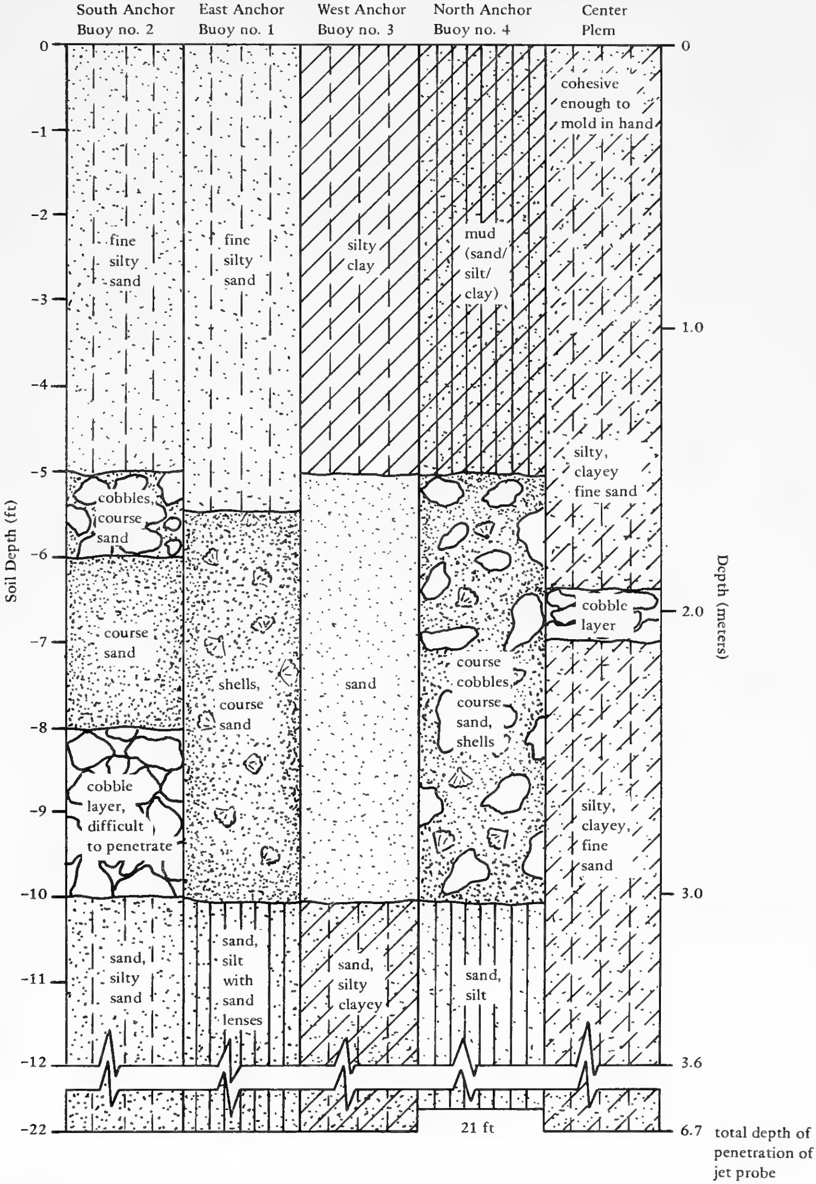


Figure 25. Jet probe data from Coronado sites (12 February 1981).

The electronics and electrical systems have performed well with the exception of the jetting sleeve. It failed several times during testing, and its use was eventually discontinued. Its purpose was to help evaluate the effect of the water jet on the data from the cone and friction sleeve as a sounding is in progress. The evaluation was done by performing side-by-side soundings with and without jetting and comparing the data.

The jetting did aid penetration. At the soundings near Port Hueneme full penetration was achieved when jetting was used (Figure 11). When it was not (Figure 10), refusal was met at a dense sand layer at 25 feet of penetration. At Coronado there was no significant difference in penetration with or without jetting (Figure 24). Refusal in all tests was essentially met at the cobble layer. However, with jetting, on the average, there was modest additional penetration. The jet was not used in Norton Sound or in San Francisco Bay.

The stratigraphies developed from the Port Hueneme (Figure 18) and the Coronado soundings (Figures 18 and 24, respectively) show that jetting does not influence the cone or friction sleeve data. In a highly layered seafloor at the Port Hueneme site, the stratigraphies developed with and without the jetting are in good agreement. For the Coronado sites, jetting and nonjetting stratigraphies are nearly identical.

Stratigraphies developed from the XSP data have been compared to historical core records and cores taken at the test sites. There are too many profiles from Norton Sound to present in this report, but in general the XSP stratigraphy compared well to the core stratigraphy. The data at Port Hueneme show good agreement to core records (Figure 19). No core was taken at the San Francisco Bay site, but the geology of the test area has been well-defined (Corps of Engineers, 1963). The test area was in young bay mud; the XSP identified the soil as a silty-clay to clayey silt. For this site, the data indicate an undrained shear strength of nearly zero at the soil surface, increasing to about 3 psi at a soil depth of 38 feet. Undrained shear strengths of these values are very indicative of young bay mud. At Coronado, no cores were taken, but the soil profile was determined with jet probings at the XSP sounding locations. The general agreement between the jet probe stratigraphy and the XSP stratigraphy is good (Figure 25).

The data acquired from replicate soundings showed good agreement, which is a recognized advantage of cone penetration testing.

CONCLUSIONS

1. The XSP cone penetrometer is a reliable piece of equipment for gathering in-situ soil data at subbottom depth of 40 feet in up to 200 feet of water.

2. The water-jet system aids penetration but does not allow penetration through very dense sands or cobble layers.

3. The water jet, when used as described in this report, does not affect the penetrometer data.

4. The sounding data (friction sleeve and cone pressure) are repeatable from test to test.

5. The sounding data can be interpreted readily to determine soil stratigraphy (using Figure 12). This stratigraphy compares well to core records and other data used for comparison.

6. The XSP is a reliable device for gathering marine soil data to assist the Navy in siting and designing facilities and structures in marine cohesionless sediments.

RECOMMENDATIONS

1. The XSP should be maintained ready for use on projects requiring geotechnical data. The device should be available to the Navy to assist in surveying underwater sites and in designing of seafloor facilities and structures.

2. Evaluation of the XSP cone penetrometer should be continued as it is used in various seafloors on these projects. Cores should also be taken at sounding locations to continue evaluation and perhaps for modifications of Figure 12.

3. The XSP should be updated to include a piezocone. This piezocone measures pore water pressure response, which has been shown to affect CPT data (ESOPT, 1982).

4. To increase the XSP's usefulness, the water-depth capability and soil penetration depth capability should be extended.

ACKNOWLEDGMENTS

The contributions of Woodward-Clyde Consultants, Plymouth Meeting, Penn., and Fugro-Gulf, Inc., Houston, Tex., are appreciated. The at-sea testing in Norton Sound was supported by the U.S. Geological Survey and the Bureau of Land Management through an interagency agreement with the National Oceanic and Atmospheric Administration. The 481st Transportation Company (Heavy Boat) U.S. Army Reserve provided ship support for the tests in San Francisco Bay. The contributions of Dr. Philip Valent (now of NORDA) in the early development is acknowledged. The capable assistance of Mr. P. Babineau and the Technical Services Division of NCEL in performing at-sea tests is appreciated.

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Appendix

INTERPRETATION AND USE OF CPT DATA

CORE PENETRATION TEST (CPT) PRACTICE

The CPT was introduced in Europe about 50 years ago and recently has gained acceptance in many countries. Its initial application was to pile design as the test resembles a model pile test. However, research has extended its utility to soil classification and determination of relative density, friction angle, settlement on sand, and clay impressibility. Also, methods for designing shallow foundations from CPT data have been developed. Many papers on CPT test equipment and data interpretation can be found in the proceedings of two Eurpoean Symposiums on Penetration Testing reported in ESOPT (1975) and (1982).

Soil Properties

Soil Classification. Efforts to classify soils with the CPT were first reported in Begemann (1965). His method was based on the ratio of sleeve friction to cone pressure (i.e., the friction ratio). In essence, he found that the friction ratio increased as median grain diameter decreased. Begemann's observations have been generally confirmed by others who developed soil classification charts. Common to classification charts is a dependence on the cone type used and the difficulty in classifying mixed (sand/silt/clay) soils. Of interest to the Navy are classification charts developed for electrical friction cones as this is the type of cone used in offshore investigations. Martin and Douglas (1981) published such a chart (Figure 12) which is perhaps the most comprehensive classification chart available. Work has also been done to extend CPT soil classification to carbonate soils (Beringen et al., 1981). In Beringen's chart, cone resistance is used to estimate the degree of cementation. Other parameters (e.g., gradation and microscopic examination) are used to further classify calcareous soils.

Relative Density. Relative density can be estimated from CPT cone pressure data but is confounded by lateral stresses, grain size, depth of overburden, and other parameters. Consequently, theoretical approaches to determining density have not proved as successful as empirical procedures. Caution, however, is warranted. Villet and Mitchell (1981) pointed out that these empirical relationships are not unique but vary according to the sand being penetrated. Schmertmann (1978) presented a plot of cone pressure versus vertical effective stress for different relative densities (Figure 26). These curves are for normally consolidated sands. For overconsolidated sands, he suggests a method of calculating an equivalent normal consolidated sand cone pressure from the measured overconsolidated sand cone pressure for use in Figure 26.

$$\frac{q_{cOC}}{q_{cNC}} \cong 1 + \frac{3}{4} \frac{K'_{oOC}}{K'_{oNC}} - 1 \qquad (1)$$

where: q_{cOC} = cone pressure for overconsolidated sand

q_{cNC} = cone pressure for normally consolidated sand

K'_{oOC} = coefficient of lateral pressure for overconsolidated sand

K'_{oNC} = coefficient of lateral pressure for normally consolidated sand

and $$\frac{K'_{oOC}}{K'_{oNC}} \cong (OCR)^{0.42} \qquad (2)$$

Villet and Mitchell have shown that this relationship can be improved by performing one or two triaxial tests to define the relative density-friction angle relationship. With this data a procedure by Durgunoglu and Mitchell (1975) can be used to develop a complete relative density, overburden pressure, cone pressure relationship. This procedure provides better accuracy but is more complex and, therefore, is not presented here. With this procedure, relative density relationships can be tailored for a particular sand.

Friction Angle. Friction angle for a given sand is proportional to relative density and can, therefore, be estimated from a relative density determined as described previously. An example of this type of relationship is shown in Figure 27 (Schmertmann, 1978). The relationship can be developed from one triaxial test by establishing a single point on the graph and drawing a line that follows the trend shown. Another method is shown in Figure 28 (Mitchell et al., 1978). With this graph, the friction angle can be estimated directly from the cone pressure and the overburden pressure. As with the relative density relationship of Figure 26, the curves given in Figures 27 and 28 must be applied with engineering judgment. In addition to these simple approaches for estimating friction angle, the Norwegian Institute of Technology (Senneset et al., 1982) has developed a method in which pore pressure is measured. The cone used must be a piezocone. The pore pressure information is used to convert total stress data to effective stress data, thereby eliminating some of the empiricism of previous methods.

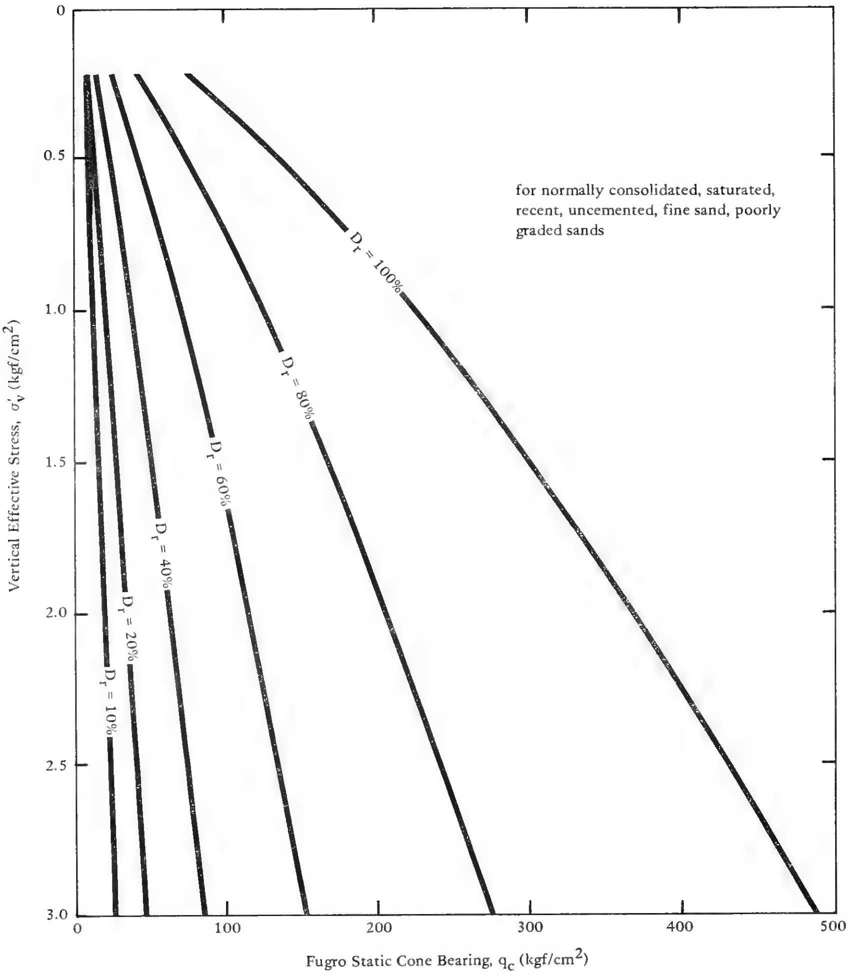


Figure 26. q_c D_r correlation.

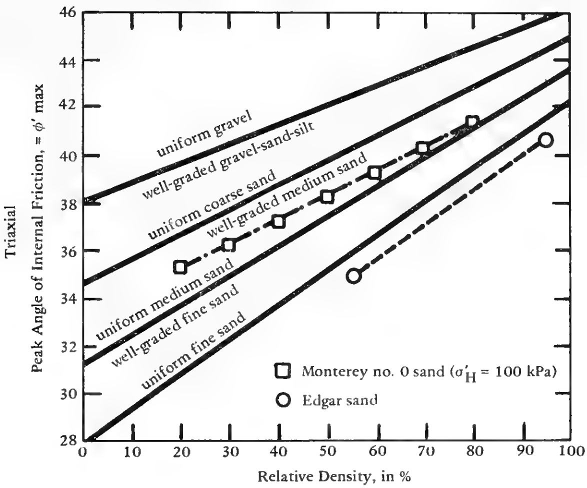


Figure 27. Triaxial cell friction angles for various sands as a function of relative density (Schmertmann, 1978).

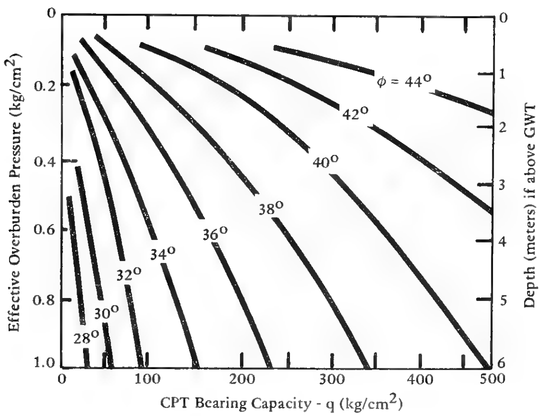


Figure 28. Method for estimating effective angle of friction (ϕ') from static cone bearing resistance (q_c) (from Mitchell et al., 1978).

Undrained Shear Strength of Clay. The undrained shear strength of clayey soils can be determined from the formula

$$s_u = \frac{q_c - \sigma_{vo}}{N_k} \qquad (3)$$

where: s_u = undrained shear strength

q_c = cone pressure

σ_{vo} = total overburden pressure

N_k = cone factor

The average value of N_k is 15 with a variation of about ±5. The recommendations on how to select the best N_k for reducing data are confusing. In his paper, deRuiter (1982) recommends a value of 10 to 15 for normally consolidated clays and 15 to 20 for overconsolidated clays. However, Schmertmann (1978) indicates the N_k varies according to cone type and clay strength. He says data suggest that weaker clays have higher N_k's and stronger clays have lower N_k's. However, each author suggests that caution be used and that a local correlation be made, preferably using a value backfigured from a failure. Further research with piezocone data may narrow the range of N_k. The work of Senneset et al. (1982) indicates that estimates of s_u can be made from cone data that include the effect of pore water pressures. By subtracting the pore water pressure piezocone data from the cone pressure some of the scatter can be reduced.

$$s_u = \frac{q_c - \mu_c}{N'_k} \qquad (4)$$

where: μ_c = pore pressure near the cone

N'_k = effective cone factor (9±3)

The likely variation of N'_k is ±3.

Other Properties

Estimates of the remolded strength and sensitivity of clays can be made when a friction cone is used. Schmertmann (1978) has presented these methods and states that they represent one measure of these properties. Schmertmann also indicates overconsolidation of clays can be estimated, but large errors may be involved. Compression moduli for sands and clays can be estimated with empirical correlations (Senneset et al., 1982). When a piezocone is used, the coefficient of consolidation of a clay can be roughly determined (Senneset et al., 1982).

However, the CPT test must be performed so that pore pressure dissipation curves can be determined, which means the cone must be stopped periodically. As a result, the time required to perform a cone sounding is very significantly increased.

PILE DESIGN

Pile design from CPT data is separated into two parts: end bearing and side friction. Procedures for estimating end bearing resistance were first developed in The Netherlands several decades ago and have been under continuous development since then. Procedures for estimating frictional resistance followed the development of the friction sleeve on the CPT. The procedures that follow are for driven, straight-sided displacement piles as summarized by Schmertmann (1978).

End Bearing Resistance

For all soils, the pile tip is assumed to be supported by a zone soil from 0.7d to 4d (d = pile diameter) below the tip to 8d above the tip. The lowest below-tip end bearing contribution is found using the procedures shown in Figure 29. However, the cone record is searched to 10d below the pile tip to check for a weaker layer of significance. If such a weaker layer is found, this governs the weakest path rule in Figure 29. For the above tip contribution, if there are a few abrupt cone pressure reductions and recoveries, they can be ignored. Because of uncertainties involved in developing cone tip pressures, a cutoff of 300 kg/cm^2 is usually applied to the cone pressure. Also, pile tip pressures are limited to 150 kg/cm^2 in sands and 100 kg/cm^2 in very silty sands. In clays, these procedures have been found applicable when undrained shear strengths are less than 7 psi. For higher strength clays, Schmertmann recommends reducing end bearing according to the adhesion factors given in Figure 30. These factors will reduce tip capacity by a larger percentage as soil strength increases.

Side Friction

Nottingham (1975) developed an empirical procedure for estimating pile friction that can be applied to both sands and clay soils. This procedure has the advantage that a direct measure of soil adhesion is used in the design. The formula used is:

$$Q_s = K_{s,c} \left\{ \sum_{\ell=0}^{8d} \frac{\ell}{8d} f_s A'_s + \sum_{8d}^{L} f_s A'_s \right\} \qquad (5)$$

where: Q_s = total ultimate pile friction

$K_{s,c}$ = correction factors for sands and clays to be applied to f_s

ℓ = depth to which f_s is being considered

d = pile diameter

f_s = unit local friction sleeve resistance

A'_s = pile-soil contact area over f_s depth interval

L = total embedded pile length

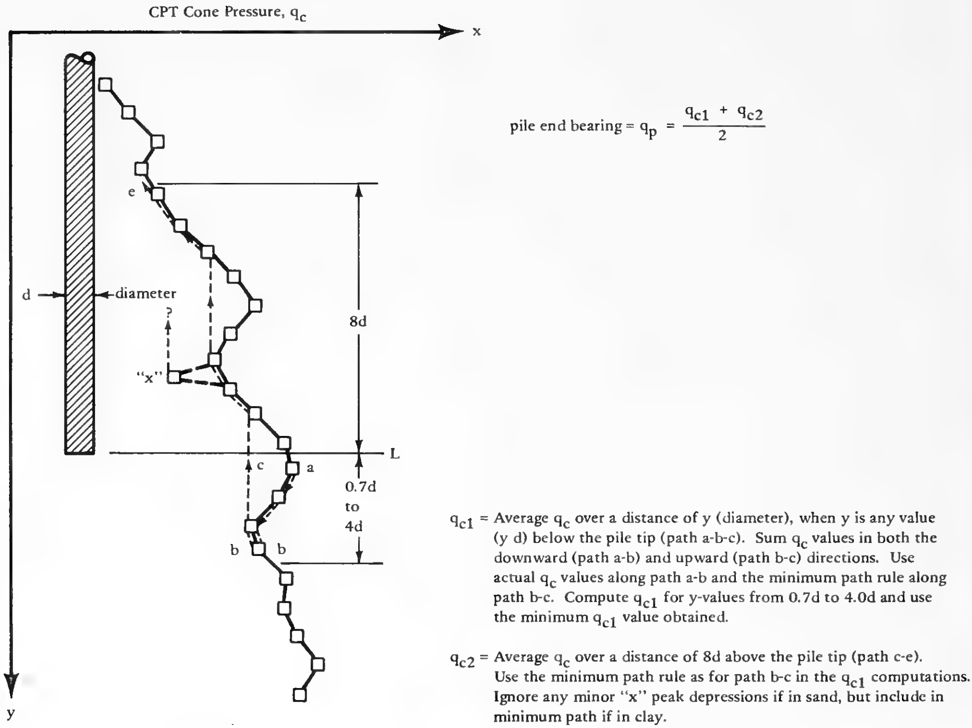


Figure 29. Dutch procedure for predicting pile tip capacity (from Schmertmann, 1978).

The correction factors K_s and K_c can be found in Figure 31. Other procedures are also available and are reported by Schmertmann (1978). One method involves estimating s_u and then reducing it by multiplying by the factor given in Figure 30. Another incorporates effective stress by including overburden pressure but still relies on an empirically derived factor. The method presented herein has been demonstrated by Schmertmann and has given good results; therefore it is recommended. Negative side friction caused by downloading on the pile by the soil is usually taken as two-thirds of the positive friction values.

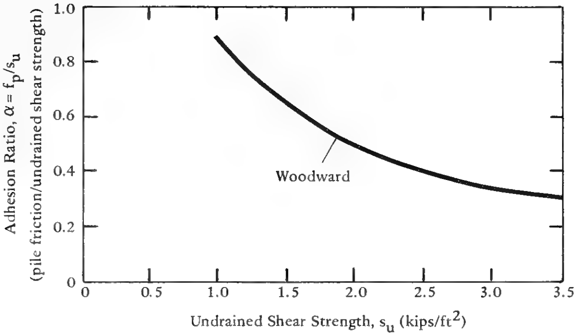


Figure 30. Correlations of clay adhesion-pile ratio with undrained shear strength (Schmertmann, 1978).

The factor of safety recommended by Schmertmann for piles designed from electrical friction cones is 2.25. This factor is applicable to tip bearing and side friction and should result in a factor of safety of at least 2.0 to the yield point.

Methods are also available for analyzing tapered piles, friction with no friction sleeve data, capacity variations due to pile shape, and insertion method. The reader is referred to Schmertmann (1978) for these details.

Bearing Capacity

Bearing capacity can be estimated by the Terzaghi bearing capacity equation or by graphs that convert cone pressure directly to bearing capacity. Schmertmann recommends using the Terzaghi equation, which is valid for footing embedment to D/B ≤1.5 (depth to width or diameter ratio).

$$q = \gamma \frac{B}{2} N_\gamma + DN_q + cN_c \tag{6}$$

where:

q = unit bearing capacity

γ = unit weight of soil

B = footing width or diameter

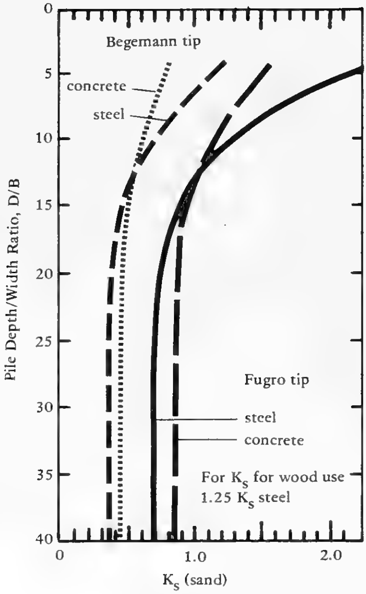
D = footing depth

c = soil cohesion

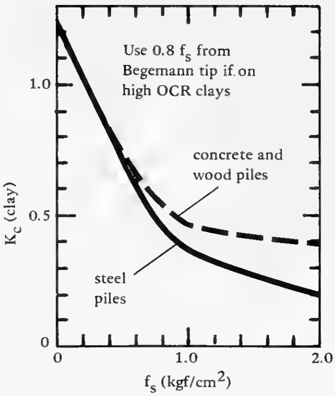
N_γ, N_q, N_c = bearing capacity factors

N_γ and N_q are estimated as

$$0.8\ N_\gamma \approx 0.8\ N_q \approx \bar{q}_{c_{0-1.5\ B}} \quad \text{in kgf/cm}^2 \tag{7}$$



(a) For sand.



(b) For clay.

Figure 31. Correction factors to be applied in Equation 5 (Nottingham, 1975) (from Schmertmann, 1978).

where $\bar{q}_c$ is the average cone pressure from the soil surface to a depth of 1.5 footing widths (kgf/cm²).

A factor of safety of 2 to 3 is then applied to obtain the allowable bearing pressure. This procedure will result in error if the cone pressure is being significantly affected by pore pressure effects.

Clay bearing capacity is more directly related to the cone pressure. For shallow footing, the cone pressure for 1.5 footing widths below the footing can be averaged to obtain the ultimate footing pressure. The allowable footing pressure is obtained by applying a factor of safety of 2 to 3.

Settlement

Settlement of footings on sand and clays can be estimated using CPT data. The results for sand are quite adequate for the static loading conditions for which the procedures apply. For clays, more uncertainty exists, and the results give more of a qualitative indication of settlement rather than a quantitative estimation. The procedures involved in making settlement estimates are too involved to present here. The reader is referred to Schmertmann (1978) for details regarding these procedures.

LIST OF SYMBOLS

A'_s	pile-soil contact area over f_s depth interval
B	footing width or diameter
c	soil cohesion
D	footing depth
D_r	relative density
d	pile diameter
f_s	unit local friction sleeve resistance
GWT	ground water table
$K_{s,c}$	correction factors for sands and clays to be applied to f_s
K'_{oOC}	coefficient of lateral pressure for overconsolidated sand
K'_{oNC}	coefficient of lateral pressure for normally consolidated sand
L	total embedded pile length
ℓ	depth f_s being considered
N'_k	effective cone factor (9 or ±3)
N, N_q, N_c	bearing capacity factors
N_k	cone factor
OCR	overconsolidation ratio
Q_s	total ultimate pile friction
$\bar{q}_c$	average cone pressure from the soil surface to a depth of 1.5 footing widths (kgf/cm^2)
q	unit bearing capacity
q_c	cone pressure
q_{cOC}	cone pressure for overconsolidated sand
q_{cNC}	cone pressure for normally consolidated sand
s_u	undrained shear strength

σ_{vo}	total overburden pressure (vertical)
μ_c	pore pressure near cone
γ	unit weight of soil
α	adhesion ratio
ϕ	angle of internal friction

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